Part III Port Facility Section

Chapter 1 General

In this Part, matters related to waterways and basins, protective facilities for harbors, mooring facilities, port transportation facilities, cargo sorting facilities, storage facilities, facilities for ship services, other port facilities and facilities subject to the Technical Standards are described

Chapter 2 Items Common to Facilities Subject to Technical Standards

1 Verification of Structural Members

[Ministerial Ordinance] (Performance Requirements for Structural Members Comprising the Facilities Subject to the Technical Standards)

Article 7

1 The performance requirements for the structural members comprising the facilities subject to the Technical Standards shall be such that the functions of the facilities are not impaired and the continuous use of the facilities is not affected by damage due to the actions of self-weight, earth pressure, water pressure, variable waves, water currents, Level 1 earthquake ground motions, collisions with floating objects, etc., in light of the conditions of the facilities during construction and in service.

2 Beyond what is provided for in the preceding paragraph, the performance requirements for the structural members comprising the facilities of which there is a risk that damage may seriously affect human lives, property or socioeconomic activity following a disasterare as prescribed respectively in those items:

(1) In the event that the functions of the facilities are impaired by damage due to design tsunamis, accidental waves, Level 2 earthquake ground motions, etc., the structural stability of the facilities shall not be affected significantly. Provided, however, that in the performance requirements for the structural members comprising the facilities in which further improvement of performance is necessary due to environmental conditions, social circumstances, etc., to which the facilities are subjected, the damage due to the actions shall not affect the restoration through minor repair work of the functions of the facilities.

(2) In the performance requirements for structural members comprising facilities which are required to protect the landward side of the facilities from design tsunamis, it is necessary that damage due to design tsunamis, Level 2 earthquake ground motions, etc., shall not affect the restoration of the function of the facilities through minor repair work.

3 In addition to the provisions of the paragraph (1), the performance requirements for the structural members comprising high earthquake-resistance facilities shall be such that the damage, etc., due to Level 2 earthquake ground motions, etc., shall not affect the restoration of the functions required of the facilities through minor repair work in the aftermath of the occurrence of Level 2 earthquake ground motions. Provided, however, that in the performance requirements for the structural members comprising the facilities in which higher earthquake-resistant performance is required due to environmental conditions, social circumstances, etc., to which the facilities are subjected, the functions required of the facilities in the aftermath of the occurrence of Level 2 earthquake ground motion shall be maintained for the continuous use of the facilities without impairing the functions.

4 In addition to the provisions of the preceding three paragraphs, necessary matters concerning the performance requirements for the structural members comprising facilities subject to the Technical Standards shall be provided by the Public Notice

[Public Notice] (Structural Members Comprising the Facilities Subject to the Technical Standards)

Article 21

The items to be specified by the Public Notice under Article 7, paragraph (4) of the Ministerial Ordinance concerning the performance requirements of the structural members comprising the facilities subject to the Technical Standards shall be as provided in the following Article through Article 28.

Article 22

1 The performance criteria common to the structural members comprising the facilities subject to the Technical Standards shall be as provided respectively in the following items:

(1) The structural members comprising the facilities damage of which may seriously affect human lives, property, or socioeconomic activity shall be such that the degree of the damage under tha accidental situation, in which the dominating actions are design tsunamis, accidental waves or Level 2 earthquake

ground motions, is equal to or less than the threshold level corresponding to the performance requirements.

(2) The structural members comprising the facilities which are required to protect the landward side from design tsunamis shall be such that the degree of damage under tha accidental situation, in which the dominating actions are design tsunamis, accidental waves or Level 2 earthquake ground motions, is equal to or less than the threshold level.

2 In addition to the provisions of the preceding paragraph, the performance criteria for the structural members comprising high earthquake-resistance facilities shall be such that the degree of damage under tha accidental situation, in which the dominating actions are Level 2 earthquake ground motions, is equal to or less than the threshold level corresponding to the performance requirements.

3 In cases where the effects of scouring and sand washing-out on the integrity of the structural members may impair the stability of the facilities, the appropriate countermeasures shall be taken.

[Interpretation]

8. Structural Members Comprising the Facilities Subject to the Technical Standards

(1) The performance criteria not specific to structural type but common to all structural members (hereinafter called "structural members") requiring integrity to ensure stability of the facilities subject to the Technical Standards are set forth.

① **Structural members of facilities prepared for accidental incident** (except protection facilities against tsunamis) (Paragraph 2, Item 1 of Article 7 of the Ministerial Ordinance and the interpretation related to Paragraph 1, Item 1 of Article 22 of the Public Notice)

(a) Facilities, the damage of which may seriously affect human lives, property or socioeconomic activity, are called "facilities prepared for accidental incident."

(b) The performance requirements for structural members of facilities prepared for accidental incident (except protection facilities against tsunamis) where dominating actions are against accidental situations of Level 2 earthquake ground motions, design tsunamis and accidental waves shall be safety or restorability. The performance verification items and the standard indices to determine the limit value for these actions are indicated in **Attached Table 8-1**. "Damage" in the verification item in **Attached Table 8-1** comprehensively represents the items while considering that the verification items differ according to the structural members of the facilities concerned. The proper indices shall be set determining the limit value when verifying their performance.

Attached Table 8-1 Performance Verification Items and Standard Indices to Determine the Limit Value Corresponding to Performance Requirements Common to Structural Members of Facilities Prepared for Accidental Incident (Except Protection Facilities against Tsunamis)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification items	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	2	1	22	1	1	Safety and restorability	Accidental	Level 2 earthquake ground motions [Design tsunami] [Accidental waves]	-	Damage	-

* [ ] indicates that dominating actions are replaced in the design situation.

② **Structural members of facilities prepared for accidental incident** (protection facilities against tsunamis) (Paragraph 2, Item 2 of Article 7 of the Ministerial Ordinance and the interpretation related to Paragraph 1, Item 2 of Article 22 of the Public Notice)

(a) Facilities required to protect the landward side from design tsunamis are called "protection facilities against tsunamis."

(b) The performance requirements for structural members of protection facilities against tsunamis where the dominating actions are against accidental situations of Level 2 earthquake ground motions and design tsunamis shall be restorability. The performance verification items and the standard indices to determine the limit value for these actions are indicated in **Attached Table 8-2**. "Damage" in the verification item in **Attached Table 8-2** comprehensively represents the items while considering that the verification items differ according to the structural members of the facilities concerned. The proper indices shall be set determining the limit value when verifying their performance.

Attached Table 8-2 Performance Verification Items and Standard Indices to Determine the Limit Value Corresponding to Performance Requirements Common to Structural Members of Facilities Prepared for Accidental Incident (Protection Facilities against Tsunamis)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification items	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	2	2	22	1	2	Restorability	Accidental	Design tsunami [Level 2 earthquake ground motion]	-	Damage	-

* [ ] indicates that dominating actions are replaced in the design situation.

③ **Structural members of high earthquake-resistance facilities** (Paragraph 3 of Article 7 of the Ministerial Ordinance and the interpretation related to Paragraph 2 of Article 22 of the Public Notice)

(a) The performance requirements for structural members of high earthquake-resistance facilities where dominating actions are against accidental situations of Level 2 earthquake ground motions shall be restorability or serviceability. The performance verification items and the standard indices to determine the limit value for these actions are indicated in **Attached Table 8-3**. "Damage" in the verification item in **Attached Table 8-3** comprehensively represents the items while considering that the verification items differ according to the structural members of the facilities concerned. The proper indices shall be set determining the limit values when verifying their performance.

(b) The serviceability in **Attached Table 8-3** is indicated as a limited performance to be delivered as a facility function necessary for the transportation of emergency supplies after an earthquake, and is not related to performance for normal cargo handling and other operations at the facilities concerned.

Attached Table 8-3 Performance Verification Items and Standard Indices to Determine the Limit Value for Accidental Situations Common to Structural Members of High Earthquake-Resistance Facilities

Ministerial Ordinance			Public Notice			Performance requirements*	Design situation			Verification items	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	3	-	22	2	-	Restorability and serviceability	Accidental	Level 2 earthquake ground motion	-	Damage	-

* Serviceability in this table is for “functions necessary after an earthquake (transportation of emergency supplies).”
* Restorability in this table is for “functions of the main body” or “functions necessary after an earthquake (transportation of emergency supplies).”

(2) Scouring and washout (Paragraph 1 of Article 7 of the Ministerial Ordinance and the interpretation related to Paragraph 3 of Article 22 of the Public Notice)

If the stability of facilities may be impaired by scouring of the foundation, ground and such of the facilities concerned and by washout of sediment in the hinterland ground of the structures, it is necessary to take the appropriate measures to prevent scouring and washout while considering the structural type of the facilities concerned.

1.1 General

(1) **Part III, Chapter 2, 1 Verification of Structural Members** describes the verification and other items concerning the structural performance of concrete members, steel members, composite structural members and other members comprising port facilities.

(2) **Part III, Chapter 2, 1 Verification of Structural Members** targets structural members that are constructed according to the specified method and precision using materials selected according to **Part II, Chapter 11 Materials**.

(3) The verification of structural members basically sets forth verification indices suitable for the performance of structural members based on the performance criteria specified by the performance required for the facilities.

(4) Actions to structural members may refer to descriptions for each facility according to **Part III, Chapter 4 Protective Facilities for Harbors**.

(5) Environmental actions acting on structural members are set appropriately while considering the surrounding environments of the structural members.

(6) Performance is basically verified using a mathematical model based on the dynamic mechanisms of the materials or structures, or is demonstrated with experiments and other means. Extensive achievements and experiences in the past may permit using quantitatively verified load-carrying capacity equations or experimental rules. A method based on the specifications which have been confirmed to meet the performance requirements as necessary may be considered performance verification.

(7) When verifying the performance of structural members, methods described in **Standard Specifications for Concrete Structures**,[1)] **Standard Specifications for Steel and Hybrid Structures**[2)] or **Standard Specifications for Composite Structures**[3)] may be referred to according to the type of structural members or material properties.

(8) When examining the performance of structural members using the limit state design method, the appropriate values need to be set for five partial factors (material factor, action factor [load factor], structural analysis factor, structural member factor and structure factor) while considering the properties of the facilities, materials and load.

(9) Loading tests or high precision analysis should be done on joints, corners, abrupt changes in cross sections, openings, steel material anchorage zones and other locations where the modeling of load carrying mechanisms is difficult.

(10) When verifying the performance of structural members, it is necessary to make sure that the performance of the structural members is not affected by deterioration of the materials during their design working life, etc. The concept of the maintenance level indicated in **Part I, Chapter 2 Construction, Improvement or Maintenance of Facilities Subject to the Technical Standards** needs to be complied with.

1.1.1 Verification Method of Structural Members

(1) Verification of Safety of the Structural Members

The verification of safety of the structural members shall determine the verification indices considering the type of structural members, material properties and other factors, and shall compare their responses to their limit values.

① **Verification of cross-sectional failure**

Cross-sectional failure shall be verified by confirming that the value obtained by multiplying the ratio of the design force resultant S_d to the design cross-sectional force R_d by the structure factor γ_i is 1.0 or less.

$$\gamma_i S_d / R_d \leq 1.0 \qquad \textbf{(1.1.1)}$$

The design force resultant S_d can be obtained by calculating the force resultant S (S is a function of F_d) using the design load F_d, and then summerizing values of multiplying S by the structural analysis factor γ_a.

$$S_d = \sum \gamma_a S(F_d) \qquad \textbf{(1.1.2)}$$

The design cross-sectional force R_d can be obtained by calculating the resistance R (R is a function of f_d) of the member cross section using the design strength f_d, and dividing by the member factor γ_b.

$$R_d = R(f_d)/\gamma_b \qquad \textbf{(1.1.3)}$$

② **Verification of fatigue failure**

Fatigue failure shall be verified by confirming that the value obtained by multiplying the ratio corresponding to the value dividing the design fatigue strength f_{rd} of the design variable stress σ_{rd} by the member factor γ_b by the structure factor γ_i is 1.0 or less.

$$\gamma_i \sigma_{rd} / (f_{rd} / \gamma_b) \leq 1.0 \qquad \textbf{(1.1.4)}$$

The design fatigue strength f_{rd} shall be the value obtained by dividing the characteristic value of the material's fatigue strength f_{rk} by the material factor γ_m.

The fatigue failure may also be verified by confirming that the value obtained by multiplying the ratio of the design fluctuating cross-sectional force S_{rd} to the design fatigue resistance R_{rd} by the structure factor γ_i is 1.0 or less.

$$\gamma_i S_{rd} / R_{rd} \leq 1.0 \qquad \textbf{(1.1.5)}$$

The design fluctuating cross-sectional force S_{rd} shall be the value obtained by multiplying the fluctuating cross-sectional force $S_r(F_{rd})$ obtained by using the design variable action F_{rd} by the structural analysis factor γ_a.

The design fatigue resistance R_{rd} shall be the value obtained by dividing the member's cross-sectional fatigue resistance $R_r(f_{rd})$ obtained by using the material's design fatigue strength f_{rd} by the member factor γ_b.

(2) Verification of Serviceability of the Structural Members

The verification of serviceability of the structural members shall determine the proper verification indices such as stress, cracks, displacement and deformations while considering the type of structural members, material properties and other factors, and shall compare their responses to their limit values.

When the serviceability concerning damage to the structural members is verified by examining their displacement and deformation, confirm that the value obtained by multiplying the ratio of the design response δ_d of displacement and deformation occurring to the structural member to the design limit value δ_a of displacement and deformation by the structure factor γ_i is 1.0 or less.

$$\gamma_i \delta_d / \delta_a \leq 1.0 \qquad \textbf{(1.1.6)}$$

The design response δ_d shall be obtained by calculating the response δ (δ is a function of F_d) using structural analysis and the design action F_d, and then summerize values of multiplying δ by the structural analysis factor γ_a.

The design limit value δ_a must be set according to the service objective or function of the structural members.

1.1.2 Examination of Changes in Performance over Time

(1) The performance of structural members must not fall below that which is required due to the degradation of materials during the design working life, etc. Thus, changes in the performance of members over time shall be examined as appropriate while considering the maintenance level set to the structural members.

(2) Changes in the performance of structural members over time shall be examined by confirming that no or slight damage over time occurs, such as the corrosion of steel materials and deterioration of concrete due to environmental actions.

(3) Changes in the performance of concrete members, steel members or composite members over time shall be basically examined according to the methods described in each section of this Chapter.

1.1.3 Partial Factors

The partial factors listed in **Table 1.1.1** can be used for the verification of structural members. This table presents standard values for the partial factors; other methods may be used when appropriate for determining the partial factors.

Table 1.1.1 List of Partial Factors[4) 5) 6)]

Partial factor		Cross-sectional failure	Fatigue failure	Other
Material factor γ_m	Concrete	1.3	1.3	1.0
	Reinforcing bars and PC steel members	1.0	1.05	1.0
	Other steel members	1.05	1.05	1.0
Load factor γ_f	Permanent actions	1.0–1.1 (0.9–1.0)	1.0	1.0
	Variable actions			
	Wave force	1.2	1.0	1.0
	Actions other than wave force	1.0–1.2 (0.8–1.0)	1.0	1.0
	Accidental actions	1.0	-	-
	Actions during construction	1.0	-	-
Structural analysis factor γ_a		1.0	1.0	1.0
Member factor γ_b		1.1–1.3	1.0–1.3	1.0
Structure factor γ_i		1.0–1.2	1.0–1.1	1.0

Note 1: The figures in parentheses in the table are applied when a smaller action results in a large risk.

Note 2: The values below may be used for the member factor when examining cross-sectional failure:

- When calculating bending and axial force: 1.1
- When calculating the maximum value of axial compressive force: 1.3
- When calculating shear capacity carried by concrete: 1.3
- When calculating shear capacitycarried by shear reinforcing bars: 1.1

Note 3: Since variations in the fatigue damage accumulated so far in the existing structural members need to be considered in designs for improvement, the member factor is set to an adequate value between 1.0 and 1.3 when examining the fatigue failure.

Note 4: When examining cross-sectional failure, the following values may be used as the structure factor:

		Permanent situation	Variable situation	Accidental situation
Superstructure of piled piers	Slab Beams	1.2 1.1	1.2 1.1	1.0 1.0
Breakwaters		1.0	1.1	1.0
Quay walls (caissons, etc.)		1.0	1.1 (only during earthquakes: 1.0)	1.0
Other (sheet pile superstructures, etc.)		1.0	1.0	1.0

1.2 Concrete

1.2.1 Basic Policy for Performance Verification

(1) The verification and other items regarding the structural performance of reinforced concrete members and prestressed concrete members comprising port facilities are described in this section. This section can also be applied to similar members such as non-reinforced concrete members while considering their characteristics.

(2) **Part III, Chapter 2, 1 Verification of Members** targets structural members and other components that is constructed according to the specified method and precision using materials selected according to **Part II, Chapter 11 Materials** and **Standard Specifications for Concrete Structures [Construction]**[7] and other documents.

(3) When verifying the performance of concrete members, **Standard Specifications for Concrete Structures [Design]**[1] may be complied with for methods not described in this section.

1.2.2 Setting of Basic Cross Sections and Characteristic Values

(1) The cross sections of structural members need to have specifications conforming to the performance criteria of the facilities concerned and shall follow the details of structures shown in **Part III, Chapter 2, 1.2.6 Details of Structures**.

(2) Characteristic values used for performance verification can be determined following the descriptions in **Part II, Chapter 11 Materials**. The standard design strength can be the characteristic value of the compressive strength of concrete. The lower limit of the JIS Standards can be the characteristic value of the tensile yield strength and tensile strength of steel members.

(3) Cross-sectional force (bending moment, torsional moment, shear, axial force) applied to structural members is generally calculated by elastic analysis.

1.2.3 Verification Methods of Members

(1) Verification of Safety

Safety of the concrete members shall be verified using cross-sectional failure and fatigue failure as indices.

① **Verification of cross-sectional failure**

(a) Design cross-sectional force for the bending moment and axial force can be calculated in accordance with **Standard Specifications for Concrete Structures [Design]**.[1]

(b) Design cross-sectional force for shear can be calculated in accordance with **Standard Specifications for Concrete Structures [Design]**[1] while considering the type of beam members, plane members and other members and shear properties.

- Since shear failure of a beam member can occur by diagonal tensile failure mechanism and shear compression failure mechanism, these conditions need to be taken into consideration when calculating the design cross-sectional force. Which failure node will be taken can generally be determined from the span and height ratio of a member.
- If a plane member is subject to out-of-plane shear, the shear force needs to be calculated in accordance with a beam member. Moreover, when a concentrated load partially acts, the punching shear force needs to be calculated.

(c) An examination of torsion may usually be omitted since structural members in general port facilities are often less affected by torsion moment or are acted on by deformation conforming torsion moment. In other cases, it is desirable to examine these stresses in accordance with **Standard Specifications for Concrete Structures [Design]**.[1)]

② **Verification of fatigue failure**

(a) The design fatigue strength f_{rd} and the design fatigue resistance R_{rd} can be calculated in accordance with **Standard Specifications for Concrete Structures [Design]**.[1)]

(b) Other points of attention

- When the rate and degree of variable actions among all actions are high, fatigue needs to be examined.
- In the verification of fatigue failure, properly rank the cyclic actions, calculate the influence to each fatigue failure and the total influence to all action ranks, and evaluate the safety for fatigue failure. Since not only the magnitude of actions but also the number of cyclic actions significantly influences the safety for fatigue failure, the latter needs to be properly determined. Any influence from actions of a rank that does not reach the fatigue limit even after the two-millionth cycle may be ignored.

(2) Verification of serviceability

The compressive stress and crack width of concrete can be an index for concrete structural members in general port facilities. However, when the response value of the crack width cannot be properly calculated, serviceability may be verified using the stress of a reinforcing bar. When other special functions are required, it is desirable to verify by setting an adequate index referring to the relevant guidelines.

① Verification of the compressive stress of concrete in a permanent situation can be performed using **equation (1.2.1)**:

$$\sigma'_c \leq 0.4 f'_{c_k} \qquad \textbf{(1.2.1)}$$

where

σ'_c : compressive stress generated in concrete by a permanent action (N/mm^2);

f'_{ck} : characteristic value of compressive strength of concrete (N/mm^2).

② When verifying with the crack width, confirm that the value, which is obtained by multiplying the ratio of the design response value w_d of the crack width generated in the structural member to the design limit value of the crack width w_a by the structure factor γ_i, is 1.0 or less.

$$\gamma_i w_d / w_a \leq 1.0 \qquad \textbf{(1.2.2)}$$

③ When verifying with the reinforcing bar stress, confirm that the value, which is obtained by multiplying the ratio of the reinforcing bar stress σ_d corresponding to the design response value of the crack width generated in the structural member to the reinforcing bar stress σ_a corresponding to the design limit value of the crack width by the structure factor γ_i, is 1.0 or less.

$$\gamma_i \sigma_d / \sigma_a \leq 1.0 \qquad \textbf{(1.2.3)}$$

④ **Calculation of the design response value**

(a) The design response value of the crack width caused by bending may be calculated using e**quation (1.2.4)**:

$$w = 1.1 k_1 k_2 k_3 \left[4c + 0.7(c_s - \phi) \right] \left(\frac{\sigma_{se}}{E_s} + \varepsilon'_{csd} \right) \qquad \textbf{(1.2.4)}$$

where

w : design response value of the crack width (mm);

k_1 : coefficient expressing the influence of the surface profile of reinforcing bars on crack width (when deformed bars = 1.0);

k_2 : coefficient expressing the influence of concrete quality on crack width;

$$k_2 = \frac{15}{f'_c + 20} + 0.7$$

f'_c : compressive strength of concrete (N/mm^2). It can normally be the design value of the compressive strength f'_{cd};

k_3 : coefficient expressing the influence of the number of layers on the tensile bars;

$$k_3 = \frac{5(n+2)}{7n+8}$$

n : number of layers of tension bars;

c : concrete cover (mm);

c_s : distance between the centers of the reinforcing bars (mm);

ϕ : diameter of the tension reinforcing bar; nominal diameter of the smallest reinforcing bar (mm);

E_s : Young's modulus of reinforcing bars (N/mm^2);

ε'_{csd} : value for considering the increase in crack width due to concrete shrinkage, creep, etc. In general cases, on the order of 100×10^{-6};

σ_{se} : stress increment of reinforcing bars near the surface (N/mm^2).

(b) The increment of reinforcing bar stress σ_{se} can be obtained using **equation (1.2.5)** assuming the cross section is in the elastic range.

$$\sigma_{se} = \frac{M_d}{A_s jd} \qquad \textbf{(1.2.5)}$$

where

M_d : design value of the bending moment (N·mm);

$j = 1 - k/3$

k : neutral axis ratio $\left(= \sqrt{2np_w + (np_w)^2} - np_w \right)$

n : Young's modulus ratio (= E_s/E_c);

p_w : reinforcing bar ratio (= $A_s/(b_w d)$);

d : effective height (mm);

b_w : width of the member (mm);

A_s : cross-sectional area of the reinforcing bars (mm^2).

(c) The design stress of a material can be calculated in accordance with **Standard Specifications for Concrete Structures [Design]**.[1)]

⑤ **Setting of a design limit value**

(a) The limit values of the crack width w_a shall generally be those listed in **Table 1.2.1**. However, this table is applicable only when the concrete cover is 100 mm or less.

Table 1.2.1 Limit Values of Crack Width w_a

Environmental classification	Deformed bar/normal round bar	PC steel member
Particularly severe corrosion environment	0.0035 c	-
Corrosion environment	0.004 c	-
Ordinary environment	0.005 c	0.004 c

(c refers to cover.)

Here, "particularly severe corrosion environment" is applied to regions and members that are exposed to severe marine environments such as parts in direct contact with seawater, or those that are washed with seawater or blown by severe sea breezes. "Corrosion environment" can be applied to other normal cases, but "ordinary environment" may also be applied to regions and members where previous performance confirms that the possibility of significant corrosion of reinforcing bar is extremely low.

When epoxy-coated reinforcing bars and stainless reinforcing bars are used, the limit values of crack width may be set as shown in **Table 1.2.2** according **to References 8)** and **9)**. However, this table is applicable only when the concrete cover is 100 mm or less.

Table 1.2.2 Limit Values of Crack Width w_a When Epoxy-Coated Reinforcing Bars and Stainless Reinforcing Bars Are Used

Type of reinforcing bar	Limit values of crack width
Epoxy-coated reinforcing bars	10% more than for non-coated reinforcing bars
SUS-304-SD	0.5 mm
SUS-316-SD	0.5 mm
SUS-410-SD	0.005c or 0.5 mm (whichever is smaller)

(c refers to concrete cover.)

(b) When an aesthetically pleasing appearance for a structure is required, the limit value of the maximum crack width of the concrete surface for appearance can be on the order of 0.3 mm.

(c) Cracks in the structure due to causes other than the acting load (e.g., cracks originating in initial defects, which do not close when the load is removed) are excluded from application of this method. Therefore, a separate examination is necessary.

(3) When water-tightness is required, verification can be performed using the crack width as an index. In this case, it is necessary to specify the limit value of the crack width appropriately while considering the service conditions of the facilities and the characteristics of the acting loads, etc. In general, the limit values presented in **Table 1.2.3** can be used.

Table 1.2.3 Limit Value of Crack Width w_a for Water-Tightness

Level of water-tightness requirement		When high water-tightness is to be ensured	When normal water-tightness is to be ensured
Predominant action cross-sectional force	Axial tension	-*1)	0.1 mm
	Bending moment*2)	0.1 mm	0.2 mm

*1) All cross sections are compressed and the minimum compressive stress shall be 0.5 N/mm^2.
If an examination is carried out with a detailed analysis, the value shall be determined separately.
*2) When subjected to reversed cyclic loadings, the limit crack width is determined according to cases where the axial tension is predominant.

(4) In cases where the action of cargo handling equipment is comparatively large and deflection to an extent that will hinder cargo handling can be expected, as in the superstructures of piled piers, verification is made using deflection as the index as necessary. The limit value of deflection in this case can be determined referring to the performance of the crane and **Specifications for Highway Bridges and Commentaries**.[10)]

1.2.4 Examination of Changes in Performance over Time

(1) It shall be basically verified that the performance of structural members is not deteriorated during the design working life. However, an examination of changes in performance over time can be omitted for existing facilities which have a design working life of about 50 years and which show no significant reduction of performance so far due to deterioration caused by chloride-induced deterioration during the design working life, provided that the facilities satisfy all of the following conditions:

① As the concrete cover for reinforcing bars, a value equal to or greater than the standard value specified in **Table 1.2.4** is set.

② A quality equal to or better than the concrete with the water-cement ratio specified in **Table 3.2.2** of **Part II, Chapter 11, 3 Concrete** shall be ensured.

③ Construction work shall be performed with care.

Table 1.2.4 Standard Values of Concrete cover

Environmental classification	Cover (mm)	Remarks
Particularly severe corrosion environment	70	Parts in direct contact with seawater Parts washed with seawater Parts exposed to severe sea breezes
Ordinary environment	50	Parts other than the above

(2) Measures to Suppress or Prevent Corrosion of Reinforcing Bar

① There are many examples so far where the performance of concrete structural members in ports is remarkably reduced during the design working life because of corrosion of reinforcing bar due to chloride-induced deterioration. Therefore, measures to suppress or prevent corrosion of reinforcing bar occurring during the design working life should be taken for structural members where performance is expected to be reduced due to chloride-induced deterioration while considering the maintenance level set to the members.

② Typical measures to suppress or prevent corrosion of reinforcing bar in concrete structural members that have been applied to port structures so far include the use of high durability reinforcement such as epoxy-coated reinforcing bars, stainless reinforcing bars and continuous fiber reinforcement; suppression of penetration by external deterioration factors such as chloride ions by surface coating, densification of cement matrix or other means; and suppression of the corrosion of steel members with cathodic protection. When applying these measures, refer to **References 8)**, **9)**, **11)** and **12)** for high durability reinforcement, **References 13)** to **16)** for preventing the penetration of deterioration factors and **Reference 17)** for cathodic protection. In addition, **Reference 18)** may also be referred to.

③ When examining the application of newly developed materials and construction methods in addition to these measures, it is necessary to fully understand their characteristics and consider their construction conditions and maintenance methods after construction.

④ **References 19)** and **20)**, which show proposals for methods to develop plans for performance verification and maintenance when applying measures to improve the durability of concrete structural members, can be referred to.

(3) Corrosion of reinforcing bar Due to the Penetration of Chloride Ions

① The verification of corrosion of reinforcing bar due to the penetration of chloride ions can generally be performed using **equation (1.2.6)**.

$$\gamma_i C_d / C_{lim} \leq 1.0 \qquad \textbf{(1.2.6)}$$

where

γ_i : structure factor. It may be 1.0 in general, but should be 1.1 for important structures;

C_d : design value of chloride ion concentration at the position of the reinforcing bars (kg/m^3);

C_{lim} : limit concentration for initiation of corrosion of reinforcing bar (kg/m^3).

Setting various limit values is possible for the verification of corrosion of reinforcing bar by the penetration of chloride ions; here, however, the limit state is defined as the point of time when corrosion of reinforcing bar initiates, while considering the availability of a safety assessment and the possibility of an assessment at the current technical level.

② The design value C_d of chloride ion concentration at the position of the reinforcing bars can be obtained using **equation (1.2.7)**.

$$C_d = \gamma_{Cl}\, C_0 \left(1 - erf\left(\frac{0.1c}{2\sqrt{D_d t}}\right)\right) + C_i \qquad \textbf{(1.2.7)}$$

where

γ_{Cl} : safety coefficient considering the design value C_d of chloride ion concentration at the position of the steel. It should be 1.3 in general, but may be 1.1 if high construction precision is ensured;

C_0 : chloride ion concentration at the surface of the concrete (kg/m^3);

c : design value of the concrete cover (mm);

D_d : design diffusion coefficient for chloride ions (cm^2/y);

t : design working life (y);

$$\left(erf(s) = \frac{2}{\sqrt{\pi}} \int_0^s e^{-\eta^2} d\eta \right)$$

erf : error function

C_i : initial chloride ion concentration (kg/m^3). It shall be determined using actual data or past performance data. When no actual data is available, it shall be set to 0.3 kg/m^3.

③ It is preferable to set the chloride ion concentration on the surface C_0 based on actual data measured under environmental conditions similar to those at the location where the structural member is to be installed. In cases where the distance between the water level (HWL) and the bottom surface of the members of the concrete superstructure of a piled pier is on the order of 0 to 2.0 m, C_0 can also be set using **equation (1.2.8)** based on the actual data in **Referneces 21)** and **22)**.

$$C_0 = -6.0x + 15.1 \qquad \textbf{(1.2.8)}$$

where

C_0 : chloride ion concentration on the surface (kg/m^3); it shall not be less than 6.0 kg/m^3;

x : distance between the HWL and the bottom surface of the member (m).

④ The design diffusion coefficient for chloride ions D_d can be obtained using **equation (1.2.9)**.

$$D_d = \gamma_c D_k + \lambda \left(\frac{w}{l}\right) D_0 \qquad \textbf{(1.2.9)}$$

where

γ_c : material factor of concrete. In general, it may be 1.0;

D_k : characteristic value of diffusion coefficient for chloride ions in concrete (cm^2/y);

λ : coefficient expressing the effect of crack on the diffusion coefficient. In general, it may be 1.5;

D_0 : constant expressing the effect of crack on the migration of chloride ions in concrete. In general, it may be 400 cm^2/y;

w/l : ratio of crack width to crack interval;

$$w/l = (\sigma_{se}/E_s + \varepsilon'_{csd})$$

σ_{se} : increment of reinforcing bar stress (N/mm^2);

E_s : Young's modulus of reinforcing bars (N/mm^2);

ε'_{csd} : value for considering an increase in crack width due to concrete shrinkage and creep, etc. It may be set in accordance with **equation (1.2.4)**.

⑤ When the concrete which will actually be used is known in advance, the characteristic value D_k of the diffusion coefficient for chloride ions in the concrete shall be set by the experiments [23) 24)] using specimens prepared from the concrete. In other cases, D_k may be set using **equations (1.2.10)** and **(1.2.11)**.[25)]

When using ordinary Portland cement (0.35 < W/C < 0.55)

$$\log_{10} D_k = 3.4\,(W/C) - 1.9 \qquad \textbf{(1.2.10)}$$

When using blast-furnace slag cement or silica fume (0.40 < W/C < 0.55)

$$\log_{10} D_k = 2.5\,(W/C) - 1.8 \qquad \textbf{(1.2.11)}$$

⑥ The limit concentration for the initiation of corrosion of reinforcing bar C_{lim} shall be set appropriately while considering the conditions of similar structures, etc. If port and harbor facilities are constructed in ordinary marine environments and the concrete cover specified in **Part III, Chapter 2, 1.2.6 Details of Structures** is ensured, C_{lim} can generally be set at 2.0 kg/m^3. This is the lower limit of the chloride ion amount to initiate corrosion of reinforcing bar based on the results of experiments at the Port and Airport Research Institute (PARI).[26)]

(4) Examination for Other Deterioration Factors

① **Corrosion of reinforcing bar due to carbonation**

(a) Corrosion of reinforcing bar due to carbonation shall be verified when performance deterioration of structural members due to carbonation is expected. There are not many cases so far where the performance of concrete structural members in ports is remarkably reduced during the design working life because of corrosion of reinforcing bar due to carbonation.

(b) Verification of corrosion of reinforcing bar due to carbonation may be performed using **equation (1.2.12)**.

$$\gamma_i\, y_d / y_{lim} \leq 1.0 \qquad \textbf{(1.2.12)}$$

where

γ_i : structure factor. It may be 1.0 in general, but should be 1.1 for important structures;

y_d : design value of carbonation depth (mm);

y_{lim} : limit depth for the initiation of corrosion of reinforcing bar (mm).

(c) **Standard Specifications for Concrete Structures [Design]**[1)] may be referred to for details on the verification of corrosion of reinforcing bar due to carbonation.

② **Freezing and thawing actions**

In cold regions and other similar environments, the performance of structural members must not be degraded by the deterioration of concrete due to freezing and thawing actions. In this case, verification may be replaced by using the concrete indicated in **Part II, Chapter 11, 3 Concrete**.

③ **Chemical Attack**

The performance required for structural members must not be degraded by the deterioration of concrete due to chemical attack. In this case, if the concrete used meets the chemical attack resistance indicated in **Part II, Chapter 11, 3 Concrete**, the performance of structural members is not considered to be lost with chemical attack, and it may replace the verification of the chemical attack.

④ **Alkali-aggregate reaction**

As indicated in **Part II, Chapter 11, 3 Concrete**, the verification of alkali-aggregate reaction may be replaced by ensuring deterioration resistance to the alkali-aggregate reaction of concrete.

1.2.5 Examination of Initial Cracks

(1) **Part III, Chapter 2, 1.2.3 Verification Methods of Members** and **Part III, Chapter 2, 1.2.4 Examination of Changes in Performance over Time** assume that any initial cracks that may affect the performance required for structural members do not occur in the construction stage. Therefore, it must be confirmed that the performance required for structural members is not affected by the initial cracks. However, the verification of initial cracks may be more reasonably performed in the construction stage, in addition to cases where it is required in both the design stage and the construction stage.

(2) The verification of settlement cracks and plastic shrinkage cracks can generally be omitted. Moreover, the verification of carefully constructed structural members which are known to have no problems from previous construction records may be omitted for cracks due to cement hydration.

(3) If cracks due to cement hydration are a problem, the performance required for structural members shall be judged to be maintained by confirming that no cracks occur or that the crack width does not exceed the limit value.

① Whether cracks have occurred can be verified using **equation (1.2.13)**.

$$I_{cr}(t) \geq \gamma_{cr} \tag{1.2.13}$$

where

$I_c(t)$: crack index;

$$I_{cr}(t) = f_{tk}(t) / \sigma_t(t)$$

$f_{tk}(t)$: concrete tensile strength on the tth day of material age;

$\sigma_t(t)$: concrete maximum main tensile stress on the tth day of material age;

γ_{cr} : safety factor concerning the probability of crack occurrence.

The crack index shall be obtained by either temperature stress analysis or a highly reliable simplified evaluation method with a definite application range.

② **Verification of the crack width**

The response value of crack width may be calculated according to **Standard Specifications for Concrete Structures [Design]**.[1)]

The limit value of crack width shall be set while considering the environmental conditions, dimensions and shapes of the structures, construction methods, concrete mix proportions and types of reinforcements. In general, the limit values indicated in **Table 1.2.1** may be used.

1.2.6 Details of Structures

(1) Concrete Cover

① The concrete cover ensures the adhesive strength of the reinforcing bars and concrete which is the prerequisite for the verification of concrete structural members and significantly affects their durability. Therefore, the concrete cover needs to be properly determined while considering the required durability, functions of the facility and construction errors.

② The cover of reinforced concrete members in marine environments shall generally not be less than the values in **Table 1.2.4**. However, control of the crack width needs to be fully noted when adopting concrete cover more than 100 mm. Moreover, provided that the concrete cover is properly managed and inspected in the construction stage, construction errors involving the concrete cover may not be considered in performance verification.

③ If the above does not apply, the concrete cover may be in accordance with **Standard Specifications for Concrete Structures [Design]**.[1)]

④ The concrete cover in ② and ③ can be reduced in the following cases:

(a) Fully examined product from a concrete factory

(b) Measures to prevent corrosion of reinforcing bar are taken

(c) Non-corrosive reinforcement is used

(2) Other details of structures may be in accordance with **Standard Specifications for Concrete Structures [Design]**.[1)]

1.3 Steel

1.3.1 Basic Policy for Performance Verification

(1) **Part III, Chapter 2, 1 Verification of Members** describes the verification and other items regarding the structural performance of steel members composing port facilities.

(2) **Part III, Chapter 2, 1 Verification of Members** targets structural members and other components that are constructed according to the specified method and precision using materials selected according to **Part II, Chapter 11 Materials** and **Standard Specifications for Steel and Compound Structures [Construction]**[2)] and other documents.

(3) When verifying the performance of steel members, **Standard Specifications for Steel and Compound Structures [Design]**[2)] and **Standard Specifications for Composite Structures [Design]**[3)] may be complied with for methods not described in **Part III, Chapter 2, 1 Verification of Members**.

1.3.2 Setting of Basic Cross Sections and Characteristic Values

(1) Cross sections of the structural member must have specifications conforming to the performance criteria of the facility concerned and shall follow the details of structures shown in **Part III, Chapter 2, 1.3.6 Details of Structures**.

(2) Characteristic values used for performance verification can be determined following the descriptions in **Part II, Chapter 11 Materials**. The lower limit of the JIS Standards can be the characteristic values of the tensile yield strength and tensile strength of steel materials.

1.3.3 Verification Methods for Members

Safety and serviceability of the steel members shall be verified against the limit state set for each item by setting the proper indices which can express performance.

1.3.4 Examination of Changes in Performance over Time

It shall be basically verified that the performance of the structural members is not deteriorated during the design working life. Since steel members used in facilities subject to the Technical Standards are generally installed under severe corrosion environmental conditions, they are properly corrosion controlled with the cathodic protection method, the coating method or other corrosion protection methods. As such, changes in the performance of steel members over time shall be basically examined for the corrosion protection design of the steel members.

1.3.5 Corrosion Protection Design of Steel Members

(1) General

① Corrosion protection methods for steel members shall be properly taken with the cathodic protection method, the protective coating method or other corrosion control methods according to the natural situations in which the steel members exist. In this case, the standard corrosion protection methods shall be the cathodic protection method for the portion below the mean low water level (M.L.W.L.), and the protective coating method for the portion higher than the mean monthly-lowest water level (L.W.L.) minus 1 m.

② Corrosion protection using a corrosion allowance shall not be performed in tidal zones or underwater since significant corrosion such as concentrated corrosion may occur depending on the corrosion environmental conditions. However, the concept of corrosion control using a corrosion allowance may be applied to temporary structures.

③ The backfilling side of steel sheet pile has a slower corrosion rate than that of the seaward side, and thus no corrosion protection is required in particular. It is desirable to investigate in advance and take the proper measures if backfilling soil is supposed to be highly corrosive because of the effects of the waste material.

④ Application of the protective coating method for portions higher than the L.W.L. minus 1 m and cathodic protection is used for submerged sections below M.L.W.L and for sections in the sea bottom soil, and their reliability has been verified. If the coating method is also used underwater, it is necessary to select a coating material with a particular focus on durability, and to take care of damage incurred in the construction stage and due to collisions with driftwood. In cases where the coating method is applied to a marine atmosphere and underwater and the cathodic protection to marine soil, any degraded or damaged coating portions can be supplemented with cathodic protection; provided that the performance verification of cathodic protection is set to allow ample margin for degradation or damage of the coating material.

⑤ The applicable corrosion protection methods differ depending on whether the target facility is newly built or already existing. In other words, some methods can be applied only to newly built facilities or may have some restrictions in their construction conditions when applied to existing facilities. The applicable methods also differ depending on whether the target region of corrosion protection is in a tidal zone or underwater. Reliable methods need to be selected considering the characteristics of each corrosion protection method in terms of the corrosion environmental conditions, construction conditions and working life in addition to these conditions.

⑥ Since the maintenance of corrosion protection method is indispensable during the working life in order to maintain the performance of corrosion protection for a long period of time, inspection and diagnosis of the corrosion protection method needs to be done at a proper frequency and at required times to evaluate its performance of corrosion protection and repair the corrosion protection method or steel members as appropriate.

⑦ In general, refer to the **Manual on Corrosion Prevention and Repair for Port and Harbor Steel Structures** (2009 edition).[27)]

(2) Corrosion Rate of Steel Members

① It is desirable to determine the corrosion rate of steel members by referring to past examples in the vicinity or to results of surveys under similar conditions, since it is largely affected by the environmental conditions of the water area such as the climate conditions, salinity density of the seawater, the degree of water pollution and the existence of river water flow.

② **Table 1.3.1**, which summarizes the results of surveys of existing steel structures, can be referred to in general for the corrosion rate of steel members. However, as **Table 1.3.1** only lists average figures, which may be exceeded in certain service conditions of the steel members, it is desirable to refer to the results of corrosion surveys under conditions as similar as possible when determining the corrosion rate of steel members. Since the figures in **Table 1.3.1** are corrosion rates for one side, use them together with figures for both sides while considering the conditions of both sides of the steel members.

③ The figure for “H.W.L. or higher” in **Table 1.3.1** is the corrosion rate immediately above H.W.L.. Moreover, it is desirable to determine the corrosion rate between the H.W.L. and the seawater section by referring to actual corrosion records in the water area concerned, because a field survey of corrosion has clarified that the corrosion rate varies depending on the water area and water depth. **Table 1.3.1** shows reference values as a range; in the water depth direction, it is desirable to correspond by distinguishing the tidal zone and the underwater area where the environmental conditions differ. L.W.L. − 1.0 m or so is suitable for the boundary in this case.

The figures in **Table 1.3.1** are not applicable to the concentrated corrosion rate since such rates greatly exceed the figures in the table.

Table 1.3.1 Standard Values of Corrosion Rates for Steel Members[27)]

Corrosive environment		Corrosion rate (mm/year)
Seaward side	H.W.L. or higher (Splash zone)	0.3
	H.W.L. to L.W.L. − 1 m (Tidal zone)	0.1 to 0.3
	Submerged zone	0.1 to 0.2
	Under seabed	0.03
Land side	Above ground and exposed to air	0.1
	Back side* in soil	
	Residual water level and above	0.03
	Residual water level and below	0.02

*Back side of sheet piles, etc.

(3) Cathodic Protection Method

① Range of application

(a) The range of application of the cathodic protection method shall be at or below the M.L.W.L.. The effect of the cathodic protection increases as the period of immersion of the steel members subject to corrosion protection in seawater is longer and decreases when it is shorter. Moreover, since a greater amount of protective current flows into tidal zones than underwater, more anode will be consumed. As a result, the cathodic protection method is generally applied to the M.L.W.L. or lower.

(b) The M.L.W.L. is the average of all tide levels at low water and is calculated by subtracting half of the tidal range (H_m) of the principal lunar semi-diurnal tide (M_2) from the mean water level (M.S.L.). It may be considered to be the mean value of the M.S.L. and the L.W.L. where there is no harmonic constant data.

(c) Corrosion protection with the coating method is necessary at or above the M.L.W.L.. In this case, since the period of immersion in seawater of a range between the M.L.W.L. and L.W.L. is shorter than at or lower than the L.W.L., the corrosion protection rate is somewhat inferior, and the portion immediately below the M.L.W.L. is easily corroded, it is desirable to apply coating corrosion control to some range below the M.L.W.L. and to combine it with the cathodic protection.

(e) In general, 90% is often used for the corrosion prevention percentage (generally defined in **equation [1.3.1]**) at or below the M.L.W.L.. However, the corrosion prevention percentage generally far exceeds 90% if properly maintained and is kept at or below the protective potential.[28)] [29)] [30)] The corrosion protection ratio may be set to a proper value based on the actual corrosion protection ratio in the target environment.

$$\text{Corrosion control rate} = \frac{\begin{array}{c}\text{Mass loss of}\\\text{nonprotected steel}\end{array} \times \begin{array}{c}\text{Mass loss of}\\\text{protected steel}\end{array}}{\begin{array}{c}\text{Mass loss of}\\\text{nonprotected steel}\end{array}} \times 100\ (\%) \qquad \textbf{(1.3.1)}$$

(f) In marine construction works, there may be a period without corrosion protection after the steel pipe piles or steel sheet piles are driven and before the superstructure is installed, or during the anode renewal period for cathodic protection. Since significant concentrated corrosion may occur during this period without corrosion protection, it is desirable to give this matter thoughtful consideration.

② Protective potential

(a) In general, the protective potential of port steel structures shall be -780 mV vs. the $Ag/AgCl_{(seaw)}$ electrode.

(b) When applying a protective current through a steel structure using the cathodic protection method, the potential of the steel structure gradually shifts to a low level. When it reaches a certain potential, corrosion is suppressed. This potential is known as the protective potential.

(c) To measure the potential, an electrode that indicates stable values even in different environmental conditions is used as a reference. The electrode is called the reference electrode. In seawater, in addition to the Ag/AgCl electrode, the saturated copper sulfate electrode and the zinc electrode are sometimes used. For the protective potential of each verification electrode, refer to **Reference 31)**.

(d) When combining the coating method and cathodic protection methods (particularly, the external power source method), care should be taken not to let the coating film deteriorated due to excessive current. The preferable potential in this case is -800 to -1,100 mV vs. the Ag/AgCl electrode.

③ **Protective current density**

(a) The protective current density shall be set to an appropriate value because it varies greatly depending on the water area environment.

(b) When applying cathodic protection, the current per unit surface area of the steel member which is needed to polarize the potential of the steel member to a more base value than the protective potential is called the protective current density. The value of protective current density decreases to a constant value with the elapse of time from the initial value at the start of cathodic protection. The constant value often decreases at or less than 50% of the initial value.

(c) The protective current density varies with water temperature, velocity, waves, water quality and other factors. Where there is an inflow of river water or various discharges, or the concentration of sulfides is high, the required protective current generally increases. Furthermore, where the velocity is high, the required protective current increases. When verifying performance, it is desirable to set a characteristic value by referring to the actual results of the existing facilities in the area concerned.

(d) The values listed in **Table 1.3.2** may be used as the protective current density at the start of cathodic protection for the bare steel member surface in normal water areas.

(e) The value of protective current density in soil has been reported to vary due to physical properties (grain size, water content, soil resistance rate, etc.) and chemical properties (pH, dissolved oxygen, activities of microorganisms, etc.) of the soil.[32) 33)] For example, the protective current density reduces if the soil resistance rate is very high.[32)] However, the protective current density in masonry having wide gaps with grain sizes of about 15 to 20 cm is on the order of 1/2 of that in seawater[34)], but about the same value of the protective current density as in seawater will be needed as the grain size becomes bigger.[35)]

(f) As the duration of protection goes on, the generated current weakens. Therefore, the average generated current for calculating the lifetime of the anode is often taken as the following depending on the duration of protection:

When protected for 5 years: 0.55 × initial current density
When protected for 10 years: 0.52 × initial current density
When protected for 15 years: 0.50 × initial current density

If the protection is intended to last for more than 15 years, the value for 15 years shall be applied.

(g) If there is a corrosion-resistant coated area within a cathodic protected area, the coefficient for density of protective current as in Table 1.3.3 shall be set assuming the conductivity (corrosion-resistant metal coating, etc.), deterioration and damage of the corrosion-resistant material.[28)] The protective current density flowing into the corrosion-resistant coated area can be obtained by multiplying the density value of protective current in **Table 1.3.2** by this coefficient. The area into which the current flows when calculating the protective current may be set at or below the H.W.L. or M.S.L..

Table 1.3.2 Protective Current Density at the Start of Cathodic Protection[28)] (mA/m^2)

	Clean sea area	Contaminated sea areas
In seawater	100	130–150
In rubble mound	50	65–75
In soil (below seabed)	20	26–30
Back side in soil	10	10

Table 1.3.3 Coefficient of Protective Current Density for Corrosion-Resistant Coating[28)]

Coating method		Coefficient of protective current density[*1]	Remarks
Painting		0.25	Damage rate is set in the early stages
Organic coating	Heavy duty plastic coating (steel pipe pile)	-[*2]	Small deterioration and damage rates
	Heavy duty plastic coating (steel sheet pile, steel pipe sheet pile)	0.10	Joint fitting part shall be considered[*3]
	Super high build coating	-[*2]	Small deterioration and damage rates
	Underwater coating (paint type)	0.25	Damage rate is set in the early stages
	Underwater coating (putty type)	-[*2]	Small deterioration and damage rates
Petrolatum coating		-[*2]	Small deterioration and damage rates
Inorganic coating	Mortar coating	0.10	Conductive[*4]
Metal coating		1.00	Conductive

*1 This coefficient shall be applied when [bare steel member area／coating area] > 1.
*2 This can be excluded from the cathodic protection area because of high insulation performance and resistance to deterioration and damage.
*3 Heavy corrosion-resistant coating steel sheet piles and steel pipe sheet piles have partially uncoated areas in joint fitting parts. Although the ratio of uncoated areas in joint fitting parts to the coated areas ranges from 8% to 13% depending on the type of steel sheet pile, 10% of the coated area is factored in the design here as the uncoated area.
*4 Conductivity of the mortar coating will be factored in the design as 10% of that of metal. If a high insulation material such as FRP is used as a protection form of mortar coating, the area of coating can be excluded from the cathodic protection area.

(4) Protective Coating Method

① General

(a) It is better to use the protective coating method because cathodic protection cannot be applied to the regions in port steel structures where the duration of seawater immersion is short.

(b) As described in (3), the range of application of the cathodic protection method is designated as at or below the M.L.W.L.. However, because concentrated corrosion is liable to occur in the vicinity of the M.L.W.L., and the duration of immersion in seawater may be shortened by the effects of waves and seasonal fluctuations in tide levels, the protective coating method shall generally be used in combination with cathodic protection to the region above the depth of 1 m below the L.W.L..

(c) In steel sheet pile revetments in shallow sea areas and the like, the coating method may be applied depthwise to the whole length of the structure. By combining the cathodic protection and protective coating methods in seawater sections, extended life of the galvanic anode may be expected.[27)]

(d) The details of each corrosion-resistant coating method are described in **Part II, Chapter 11, 2.4 Corrosion Protection of Steel Members**.

② Selection of protective coating methods[36)]

As each coating method has its own features, the method most suitable to the target structure must be selected by fully examining (a) through (d) below.

(a) Conditions of the target steel structure

As the protective coating method to be applied to the target steel structure may differ depending on whether it was coated at a factory or at the site, or depending on the situation surrounding the steel structure, it is necessary to examine the following items after adequately understanding the situation of the steel structure. The primary target of description here is newly built steel structures.

- Environmental conditions: It is desirable to fully investigate the environment where the steel structure is built because it directly affects the durability of the protective coating method. Corrosion environment conditions include the constituents of seawater (salinity, etc.), pH, water temperature and velocity. Whether freshwater (rivers, etc.) or polluted water (industrial wastewater, etc.) flows into the seawater or mixes with warm water should be investigated as this influences corrosion. The possibility of damage due to the actions of waves or collisions with floating matter should also be investigated.

- Range of corrosion protection: The classification of the corrosion environment to which the coating method is applied ranges widely from marine atmosphere to marine soil. Therefore, the corrosion protection range of each protective coating method is determined by the shape of the steel structure and how it combines with the cathodic protection method and with multiple protective coating methods. Thus, it is necessary to select an appropriate coating method for the steel structures since each method has its own applicability.

  When trying to apply the coating down to the normal range (L.W.L. – 1 m) of an existing steel structure whose lower edge level of upper concrete is around the LWL, the construction becomes more difficult and expensive. If the cathodic protection method, which is guaranteed to be effective below the M.L.W.L., is adequately maintained, the coating method is not frequently required.[34)]

- Structural type of steel structures: The main steel structural types in ports are open-type piled piers, pipe sheet pile quay walls and sheet pile quay walls, and their materials are steel pipe piles, steel pipe sheet piles and steel sheet piles. Applicability of the protective coating method needs to be examined since the applicability to structural type differs depending on the type of protective coating methods. Items to keep in mind from the structural point of view, such as the height of the superstructure and existence of protrusions, also need to be examined.

(b) Required Performance for the protective coating method

One of the most important performance items required for the protective coating method is the effect of corrosion protection to the structures and the durability of the coating itself. Furthermore, it is necessary to apply the method while considering the expected working life and its expected economic purpose from among those that comply with the situation of the aforementioned target structures.

It is difficult to say whether the expected working life of the coating method is adequately understood, but one standard rule of thumb targeted primarily at steel pipe piles is indicated in **Reference 36)**.

- Effects of corrosion protection: Protective coating depending on the situation of the aforementioned target steel structures needs to be examined since the effect of corrosion protection varies by the type of protective coating method. Since the splash zones and tidal zones under extremely severe corrosion environments are difficult to maintain or repair, coating with a high corrosion protection effect needs to be applied.

- Durability: The marine atmosphere requires enough durability for factors such as direct sunlight and sea salt particles. Splash zones, tidal zones and seawater also require seawater resistance and physical strength against waves and collisions with floating matter, etc. When selecting coating, these factors and characteristics of deterioration of each coating must thoroughly be examined. **Referneces 37)** and **38)** examine the characteristics of deterioration, durability under actual marine environments and performance evaluation methods of several coating methods for a long period of 30 years.

(c) Factors in the application of protective coating methods

When selecting a coating method, consideration for construction is also needed since the construction quality of coating greatly affects the performance of corrosion protection, durability and the maintenance cost

- Constructability: The construction location of the protective coating may be restricted to factories or sites. When constructing at a site, conduct examinations while considering the applicability of the protective coating method, restrictions in working spaces and working hours due to weather, waves, tidal levels, structural types and difficulties in surface preparation since these conditions affect the construction. The effects on the surrounding environment during construction also needs to be examined.

- Construction period: Seasons and periods including the surrounding oceanographical phenomena and situations of working sites which permit construction must be considered.

(d) Actual results

Actual results under similar conditions shall be investigated since the evaluation of reliability of the coating method refers to these results. It is necessary to evaluate protective coating without past results by thoroughly investigating supportive experiment data or theories.

The protective coating methods applied to port steel structures are suitable for either coating at factories or at sites. Typical coating methods at factories include painting, heavy duty plastic coating, super high build coating and corrosion-resistant metal coating, while frequently applied coating methods at sites are underwater coating, petrolatum coating and mortar coating.

1.3.6 Details of Structures

(1) The details of structures of steel members shall be in accordance with those of each facility and each structural type indicated in **Part III, Chapter 3 Waterways and Basins** to **Chapter 11 Other Port Facilities**.

(2) Other details of structures may comply with **Standard Specifications for Steel and Compound Structures [Design]**[2] and **Standard Specifications for Composite Structures [Design]**.[3]

1.4 Composite Structure

1.4.1 Basic Policy for Performance Verification

(1) **Part III, Chapter 2, 1 Verification of Members** describes the structural performance verification and other items regarding composite structural members composed of steel, concrete and other materials composing port facilities.

(2) **Part III, Chapter 2, 1 Verification of Members** targets structural members and other components that are constructed according to the specified method and precision using materials selected according to **Part II, Chapter 11 Materials** and **Standard Specifications for Composite Structures [Construction]**[39] and other documents.

(3) When verifying the performance of composite structural members, **Standard Specifications for Composite Structures [Design]**[3] may be complied with for methods not described in **Part III, Chapter 2, 1 Verification of Members**.

1.4.2 Setting of Basic Cross Sections and Characteristic Values

(1) Cross sections of structural members must have specifications conforming to the performance criteria of the facility concerned and shall follow the details of structures shown in **Part III, Chapter 2, 1.4.5 Details of Structures**.

(2) Characteristic values used for performance verification can be determined following the descriptions in **Part II, Chapter 11 Materials**. The standard design strength can be the characteristic values of the compressive strength of concrete. The lower limit of the JIS Standards can be the characteristic values of the tensile yield strength and tensile strength of steel members.

1.4.3 Verification Methods of Members

(1) Verification of Safety

Safety of the composite structural members shall be verified using cross-sectional failure and fatigue failure as indices.

① **Verification of cross-sectional failure**

(a) The design cross-sectional force for the bending moment and axial force can be calculated in accordance with **Standard Specifications for Composite Structures [Design]**.[3]

(b) The safety for shear force must be verified considering the type of beam members and plane members, direction of shear force action and displacement of shear connectors. If the displacement of shear connectors does not affect the load carrying mechanism of the members, it can be calculated in accordance with the following items as well as **Standard Specifications for Composite Structures [Design]**.[3]

- For composite beam structural members, the yield and shear of reinforcing steel members, buckling, design shear resistance complying with failure conditions such as diagonal tensile failure and compression failuer of concrete shall be calculated and verified individually.

- If a plane member is subject to out-of-plane shear, the out-of-plane shear force shall be examined in accordance with a beam member. Moreover, when a concentrated load partially acts, punching shear failure shall be verified against the concentrated load.
- If the interface of different materials or plane members is subject to in-plane shear, the in-plane force shall be verified.
- If shear needs to be transmitted, the direct shear transmission on the shear plane shall be verified.

(c) An examination of torsion may usually be omitted since structural members in general port facilities are often less affected by torsion moment or are acted on by deformation conforming torsion moment. In other cases, it is desirable to conduct examinations in accordance with **Standard Specifications for Composite Structures [Design]**.[3)]

② **Verification of fatigue failure**

(a) The fatigue failure must be verified for members and shear connectors in the combined state after steel and concrete have been integrated considering the effects of the characteristics of actions whether variations or movements of actions exist or not.

(b) The verification of fatigue failure for concrete, reinforcing bars and steel members may be calculated in accordance with **Standard Specifications for Composite Structures [Design]**[3)] using the design variable stress that is calculated assuming that steel and concrete have been integrated.

(c) Other points of attention

- When the rate and degree of variable actions among all actions are high, fatigue needs to be examined.
- In the verification of fatigue failure, properly rank the cyclic actions, calculate the influence to each fatigue failure and the total influence to all action ranks, and evaluate the safety for fatigue failure. Since not only the magnitude of actions but also the number of cyclic actions significantly influence the safety for fatigue failure, the latter needs to be properly determined. Any influence from actions of a rank that does not reach the fatigue limit even after the two-millionth cycle may be ignored.

(2) The compressive stress and crack width of concrete can be an index for verification of the serviceability of composite structural members. However, when the response value of the crack width cannot be properly calculated, serviceability may be verified using the stress of a steel member. When other special functions are required, it is desirable to verify by setting an adequate index referring to **Standard Specifications for Composite Structures [Design]**[3)] and the relevant guidelines.

(3) Since the type and magnitude of actions and load-carrying mechanisms against actions for composite structural members vary before and after steel and concrete are integrated, verification must be properly performed before and after integration.

(4) Other examinations on limit states may be in accordance with **Standard Specifications for Composite Structures [Design]**[3)] and shall be in accordance with **Standard Specifications for Concrete Structures [Design]**[1)] and **Standard Specifications for Steel and Compound Structures**[2)], if necessary.

1.4.4 Examination of Changes in Performance over Time

(1) It must be confirmed that changes over time, such as the corrosion of steel members and deterioration of concrete due to environmental actions, do not occur or are restricted to small areas if they occur for composite structural members.

(2) Examination of the Corrosion of Steel Members

① It must be confirmed that corrosion does not occur or remains within a degree that does not affect the performance of members even if it occurs for externally exposed steel members. When corrosion protection measures are taken with a proper method considering the characteristics of the structures and environmental conditions, it may be assumed that the steel members shall not be corroded.

② Corrosion protection measures for steel members shall be properly set while considering the performance requirements, maintenance level and construction conditions. In this case, it is desirable to examine the proper construction method utilizing an actual result investigation of the existing port steel structures and corrosion-related data. Corrosion protection measures shall comply with the concept indicated in **Part III, Chapter 2, 1.3.5 Corrosion Protection Design of Steel Members** and **Part II, Chapter 11, 2.4 Corrosion Protection of**

Steel Members. For the selection of construction methods, refer to the **Manual on Corrosion Prevention and Repair for Port and Harbor Steel Structures.**[27]

③ For the corrosion of steel members covered with concrete, it must be confirmed that initial cracks or cracks in concrete due to external forces and corrosion of steel members accompanied by the carbonation of concrete and penetration of chloride ions do not occur or remain within a degree that does not affect the performance of the members even if they occur. Corrosion of steel members covered with concrete can be examined in **Part III, Chapter 2, 1.2.4 Examination of Changes in Performance over Time**.

④ It must be confirmed that steel members at the boundaries of the steel members and concrete are not corroded due to floods or other hazards, or remain within a degree that does not affect the performance of the members even if such events occur.

(3) The deterioration of concrete can be examined in **Part III, Chapter 2, 1.2.4 Examination of Changes in Performance over Time** for the effects of the corrosion of steel members due to the penetration of chloride ions, carbonation, freezing and thawing actions, chemical attack and alkali-aggregate reaction.

1.4.5 Details of Structures

Details of structures may be in accordance with **Part III, Chapter 2, 1.2.6 Details of Structures**, **Part III, Chapter 2, 1.3.6 Details of Structures**, and **Standard Specifications for Composite Structures [Design]**[3], and shall be in accordance with **Standard Specifications for Concrete Structures [Design]**[1] and **Standard Specifications for Steel and Compound Structures [Design]**[2], if necessary.

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2 Members of Structures

2.1 General

(1) **Part III**, **Chapter 2**, **1 Verification of Members** describes performance criteria of caisson structures, L-shaped block structures, cellular-block structures, upright wave-absorbing caisson structures and hybrid caisson structures that are composed of concrete members, steel members and/or hybrid members.

(2) For performance verification of members of respective structures, refer to **Part III**, **Chapter 2**, **1 Verification of Members**.

(3) Considerations for improved design of existing port facilities

① When designing and constructing a new facility by utilizing existing structures or members, it is necessary to verify the members in an appropriate way in consideration of design conditions, site conditions and other conditions of the facility. For the basic flow of designing a facility by utilizing an existing structure or existing members, refer to **Part I**, **Chapter 2**, **2.4 Improved Design of Existing Facilities Subject to the Technical Standards**.

② When designing a new facility by utilizing existing structures or members, it is preferable to clarify deterioration, damage and other changes in the states of concrete, steel and other materials through an on-site survey and conduct the verification of the members in consideration of the states of the materials.

③ For the method of verifying members to be utilized for a new facility, refer to **References 1)** and **2)**.

2.2 Caissons

[Public Notice] (Performance Criteria of Caissons)

Article 23

The performance criteria of a reinforced concrete caisson (hereinafter referred to as a “caisson” in this Article) shall be as prescribed respectively in the following items in consideration of the type of facility:

(1) Bottom slab and footings of a caisson shall be such that the risk of impairing the integrity of the bottom slab and footings of the caisson is equal to or less than the threshold level, under the permanent state in which the dominating action is self-weight, and under the variable situation in which the dominating actions are variable waves, water pressure during floating, and Level 1 earthquake ground motions.

(2) Outer walls of a caisson shall be such that the risk of impairing the integrity of the outer walls of the caisson is equal to or less than the threshold level, under the permanent state in which the dominating action is the internal earth pressure and under the variable situation in which the dominating actions are variable waves, water pressure during floating, and Level 1 earthquake ground motions.

(3) Partition walls of a caisson shall be such that the risk of impairing the integrity of the partition walls of the caisson is equal to or less than the threshold level under the variable situation in which the dominating action is water pressure during installation.

(4) A caisson which requires flotation shall be such that the risk of overturning of the floating body during flotation is equal to or less than the threshold level under the variable situation in which the dominating action is water pressure.

[Interpretation]

8. Members Composing Facilities Subject to the Technical Standards

(3) Performance Criteria of Caissons (Article 57, Paragraph 7 of the Ministerial Ordinance and the interpretation related to Article 23. paragraph 1 of the Public Notice)

Serviceability shall be the required performance for caissons under the permanent or variable situations in which the dominating actions are those shown below. Required performance verification items and indices for caissons under respective design situations shall be set appropriately depending on the type of facility in accordance with the performance criteria.

① **Bottom slab and footings**

Shown below are the performance verification items and indices for the bottom slab and footings of a caisson under respective design situations in accordance with the performance criteria.

(a) Permanent situation in which the dominating action is self-weight

Performance verification items for the bottom slab and footings of a caisson under the permanent situation in which the dominating action is self-weight and standard indices for setting limit values shall be in accordance with **Attached Tables 8-4**. Required performance verification items shall be set appropriately depending on the type of facility.

Attached Table 8-4 Performance Verification Items and Standard Indices for Setting Limit Values for Bottom Slab and Footings of Caisson
(permanent situation in which the dominating action is self-weight)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	−	23	−	1	Serviceability	Permanent	Self-weight	Water pressure, subgrade reaction, surcharge, earth pressure	Cross-sectional failure of bottom slab and footing	Design ultimate capacity
										Concrete stress in cross section of bottom slab and footing	Bending compressive stress
										Extrusion of bottom slab and footing from partition wall (yield of reinforcing bars)	Design yield stress

(b) Variable situation in which the dominating action is variable waves

Performance verification items for the bottom slab and footings of a caisson under the variable situation in which the dominating action is variable waves and standard indices for setting limit values shall be in accordance with **Attached Tables 8-5**. Required performance verification items shall be set appropriately depending on the type of facility.

Attached Table 8-5 Performance Verification Items and Standard Indices for Setting Limit Values for Bottom Slab and Footings of Caisson
(variable situation in which the dominating action is variable waves)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	23	-	1	Serviceability	Variable	Variable waves*1)	Self-weight, water pressure, subgrade reaction, surcharge, earth pressure	Cross-sectional failure of bottom slab and footing	Design ultimate capacity
										Extrusion of bottom slab from partition wall (yield of reinforcing bars)	Design yield stress
								Variable waves*2)		Cracking in cross section of bottom slab and footing	Crack widthcaused by bending
								Cyclic action of waves*3)		Fatigue failure of bottom slab and footing	Design fatigue strength

*1): Waves here shall be the waves that were defined in accordance with Item (1), Paragraph 1, Article 8 of the Public Notice and considered in performance verification of the structural stability of the facility of interest.
*2): In principle, waves here shall be the waves that were defined, in accordance with Item (2), Paragraph 1, Article 8 of the Public Notice, as the standard waves on the assumption that waves higher than the standard waves will strike the facility about 10,000 times during its design service life.
*3): Waves here shall be the waves that were defined, in accordance with Item (2), Paragraph 1, Article 8 of the Public Notice, as the waves having the height and period that were set appropriately depending on the frequency of occurrence during the design service life.

(c) Variable situations in which the dominating actions are the water pressure during flotation and Level 1 earthquake ground motions

Performance verification items for the bottom slab and footings of a caisson under the variable situation in which the dominating actions are the water pressure during flotation and Level 1 earthquake ground motions and standard indices for setting limit values shall be in accordance with **Attached Tables 8-6**. Required performance verification items shall be set appropriately depending on the type of facility.

Attached Table 8-6 Performance Verification Items and Standard Indices for Setting Limit Values for Bottom Slab and Footings of Caisson
(variable situation in which the dominating actions are the water pressure during flotation and Level 1 earthquake ground motions)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	23	-	1	Serviceability	Variable	Water pressure during flotation	Self-weight	Cross-sectional failure of bottom slab and footing	Design ultimate capacity
										Cracking in cross section of bottom slab and footing	Crack width caused by bending
								Level 1 earthquake ground motion	Self-weight, water pressure, subgrade reaction	Cross-sectional failure of bottom slab and footing	Design ultimate capacity
										Extrusion of bottom slab from partition wall (yield of reinforcing bars)	Design yield stress

② **Outer walls**

Performance verification items for outer walls of a caisson under the permanent situation in which the dominating action is the caisson internal earth pressure and under the variable situation in which the dominating actions are variable waves, Level 1 earthquake ground motions and the water pressure during flotation and standard indices for setting limit values shall be in accordance with **Attached Tables 8-7**.

Attached Table 8-7 Performance Verification Items and Standard Indices for Setting Limit Values for Outer Walls of Caisson

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	23	-	2	Serviceability	Permanent	Internal earth pressure	Internal water pressure	Concrete stress of cross section of outer wall	Bending compressive stress
										Extrusion of outer wall from partition wall (yielding of reinforcing bars)	Design yield stress
							Variable	Variable waves*1)	Internal water pressure, internal earth pressure	Cross-sectional failure of outer wall*2)	Design ultimate capacity
								Variable waves*3)		Cracking in outer wall	Crack width caused by bending
								Cyclic action of waves*4)		Fatigue failure of outer wall*2)	Design fatigue strength
								Level 1 earthquake ground motion	Internal water pressure, internal earth pressure	Cross-sectional failure of outer wall	Design ultimate capacity
								Water pressure during flotation	—	Cross-sectional failure of outer wall	Design ultimate capacity
										Cracking in outer wall	Crack width caused by bending

*1): Waves here shall be the waves that were defined in accordance with Item (1), Paragraph 1, Article 8 of the Public Notice and considered in performance verification of the structural stability of the facility of interest.
*2): Limited to outer walls affected by waves.
*3): In principle, waves here shall be the waves that were defined, in accordance with Item (2), Paragraph 1, Article 8 of the Public Notice, as the standard waves on the assumption that waves higher than the standard waves will strike the facility about 10,000 times during its design service life.
*4): Waves here shall be the waves that were defined, in accordance with Item (2), Paragraph 1, Article 8 of the Public Notice, as the waves having the height and period that were set appropriately depending on the frequency of occurrence during the design service life.

③ Partition walls

Performance verification items for partition walls of a caisson under the variable situation in which the dominating action is the water pressure during installation and standard indices for setting limit values shall be in accordance with **Attached Tables 8-8**.

Attached Table 8-8 Performance Verification Items and Standard Indices for Setting Limit Values for Partition Walls of Caisson

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	23	-	3	Serviceability	Variable	Water pressure during installation	—	Cross-sectional failure of partition wall	Design ultimate capacity
										Cracking in partition wall	Crack width caused by bending

④ **Caissons requiring flotation**

Performance verification items for caissons that require floatation under the variable situation in which the dominating action is the water pressure during flotation and standard indices for setting limit values shall be in accordance with **Attached Tables 8-9**. In verification of the performance of caissons that require flotation against overturning of the floating body, the standard index for setting the limit value shall be set appropriately.

Attached Table 8-9 Performance Verification Items and Standard Indices for Setting Limit Values for Caissons Requiring Flotation

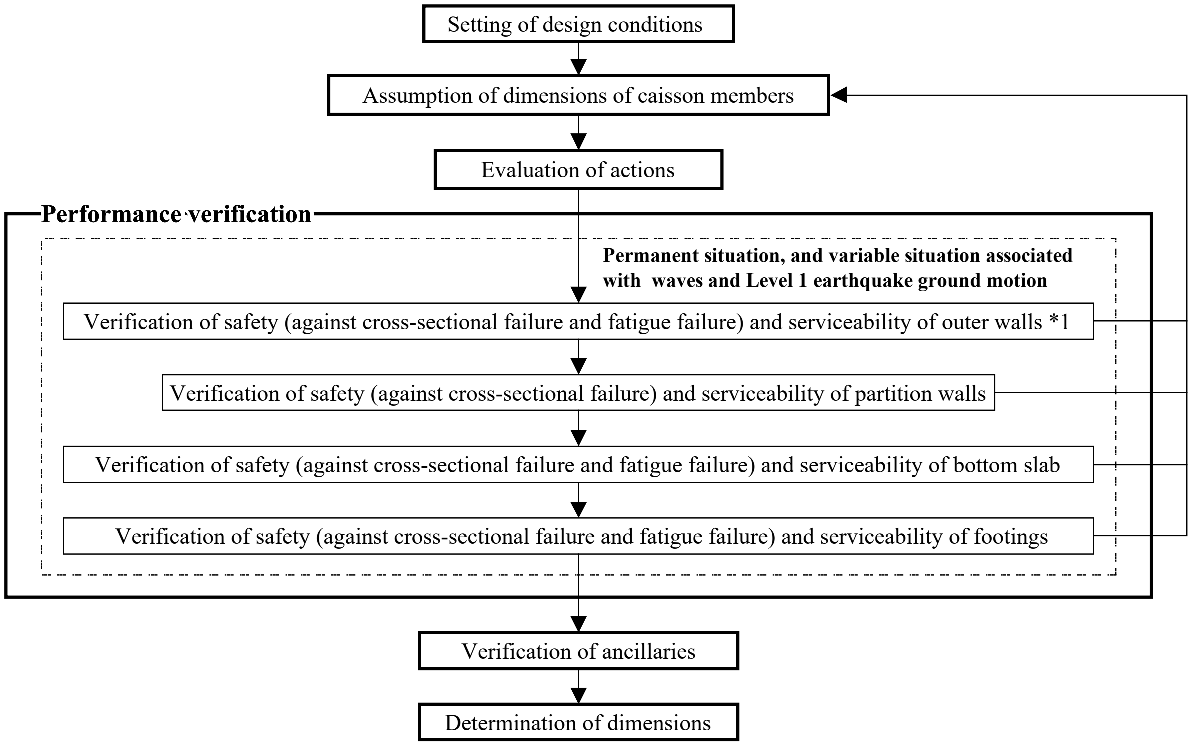
Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	23	-	4	Serviceability	Variable	Water pressure during flotation	Self-weight	Overturning of floating body	-

2.2.1 Fundamentals of Performance Verification

(1) The concept of verification described here may be applied to the performance verification of structural members of ordinary caissons.

(2) For the concept of verification of structural members, refer to **Part III, Chapter 2**, **1.1 General**.

(3) An example of the performance verification procedure for caissons is shown in **Fig. 2.2.1**.



*1 For outer walls which are not affected by waves, the safety verification may be omitted.

*2 For high earthquake-resistance facilities or the facilities to which damage might have a serious impact on human life, property, and social activity, it is preferable to verify the performance under accidental situations, as necessary. Verification of accidental situation associated with waves shall be performed in cases where damage to those facilities might have a serious impact on hazardous material handling facilities located just behind them.

Fig. 2.2.1 Example of Performance Verification Procedure for Caissons

2.2.2 Determination of Basic Cross Section and Characteristic Values

(1) The dimensions of members of a caisson shall be determined in view of the following factors:

① Capacity of caisson fabrication facilities
② Draft of the caisson and the water depth at the place of installation (depth above the crown of foundation mound)
③ Floating stability if the caisson is designed to float unassisted
④ Service conditions during towing and installation: tidal currents, waves, wind, and other conditions
⑤ Service conditions after installation of the caisson: filling and superstructure construction
⑥ Bending and torsion stresses acting on the caisson

(2) As a caisson becomes longer (mainly in the direction of the face line), it will be subjected to larger bending and torsion stresses caused by jack-up, uneven settlement and other factors. Therefore, it is necessary to examine the effects of those factors. It must be noted that towing and installing the caissons may be difficult in sea areas with high waves and/or strong currents and caissons may get damaged because it may take a long time to complete the filling work.

(3) There are many cases where caissons have outer walls with a thickness of 0.3 to 0.6 m, the bottom slab with a thickness of 0.4 to 0.8 m and partition walls with a thickness of 0.2 to 0.3 m.

(4) As the keel clearance of a caisson during installation, it is common to set the difference between the draft of the caisson and the mound crown to 0.5 m or more. This value allows for the inclination, rolling, pitching and yawing of the caisson and errors in the draft calculation. It is common to set the tide level during installation to the mean sea level (MSL) or so.

(5) For a caisson designed to float unassisted, the cross section that ensures the stability during flotation shall be determined.

① The stability of the caisson while floating may be examined using **equation (2.2.1)** (see **Fig. 2.2.2**). This equation can be applied to cases where the caisson cross section is nearly bilaterally symmetrical and its inclination is relatively small.

$$\frac{I}{V} - \overline{CG} = \overline{GM} > 0 \tag{2.2.1}$$

where

V : displacement volume (m^3)

I : geometrical moment of inertia with respect to long axis at water level (m^4)

C : center of buoyancy

G : center of gravity

M : metacenter

$\overline{CG}$: distance between center of gravity and center of buoyancy (m)

$\overline{GM}$: distance between metacenter and center of gravity (m)

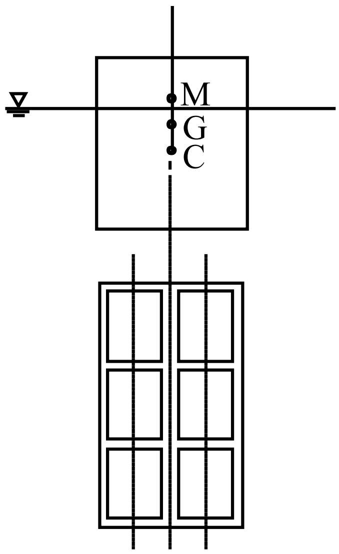


Fig. 2.2.2 Stability of Caisson

② The stability of the caisson while towed with a counter ballast placed may be examined using **equation (2.2.2)** or **(2.2.3)**.

(a) When using water as a counter ballast:

$$\frac{1}{V'}\left(I' - \sum i\right) - \overline{C'G'} > 0 \tag{2.2.2}$$

(b) When using sand, stone, concrete or the like as a counter ballast:

$$\frac{I'}{V'} - \overline{C'G'} > 0 \tag{2.2.3}$$

where

V' : displacement volume for caisson with counter ballast (m^3)

I' : geometrical moment of inertia with respect to long axis at water level for caisson with counter ballast (m^4)

C' : center of buoyancy for caisson with counter ballast

G' : center of gravity for caisson with counter ballast

$\overline{C'G'}$: distance between center of gravity and center of buoyance for caisson with counter ballast (m)

i : geometrical moment of inertia with respect to centerline parallel to axis of rotation of caisson at water level in each chamber (m^4)

③ **Equation (2.2.4)** shall be used in cases where a ballast is placed in a caisson with a footing on only one side in order to keep it in balance. (See **Fig. 2.2.3**.)

$$W_1 l_1 + W l_w = F l_f \tag{2.2.4}$$

where

W_1 : weight of ballast (kN)

W : weight of caisson (including weight of footing) (kN)

F : buoyancy acting on caisson (including footing) (kN)

l_1 : distance from outside of caisson outer wall to point where W_1 acts (m)

l_w : distance from outside of caisson outer wall to point where W acts (m)

l_f : distance from outside of caisson outer wall to point where F acts (m)

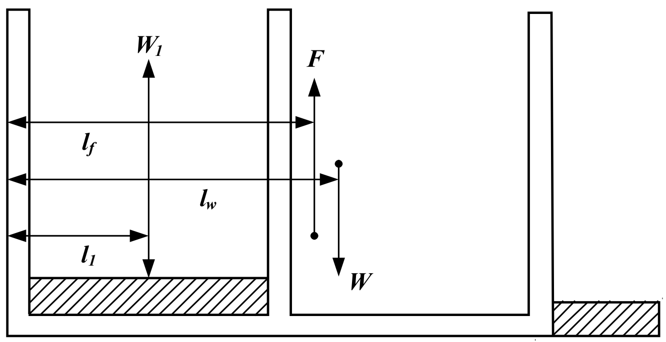


Fig. 2.2.3 Stability of Caisson with Counter Ballast

2.2.3 Actions

(1) The combinations of actions to be considered in performance verification and load factors shall be set appropriately for each facility.

(2) The combinations of actions to be considered in performance verification and the standard values of the load factors to be used for multiplying the characteristic values of actions are shown in **Table 2.2.1**. Here, the values used for the bottom slab can also be used for footings. The value in the top row in each cell of each table is the load factor to be used in examination of safety (against cross-sectional failure); the value shown in square brackets in the middle row is the load factor to be used in cases where the smaller the action, the larger the design load. These values were determined in consideration of the relationship with external stability and other factors based on reliability analysis.[3), 4)] The value shown in parentheses in the bottom row of each cell is the load factor to be used in examination of serviceability. For accidental situations, a load factor of 1.0 may be used.

If the leveling accuracy of a rubble mound is alleviated, a reaction greater than that in case of the normal leveling accuracy of ±5 cm will act on the caisson bottom slab, and in this case, the values shown in **Table 2.2.1** cannot be used. In the case where the leveling accuracy of the rubble mound is alleviated to the range of ±30 cm, the factors can be set by reference to **References 5)** and **6)**.

(3) For setting conditions of waves to be considered in the verification of serviceability under the variable situation associated with waves, refer to **Part II**, **Chapter 2**, **4.1.2 Setting of Wave Conditions to be Used for Verification of Serviceability of Structural Members**.

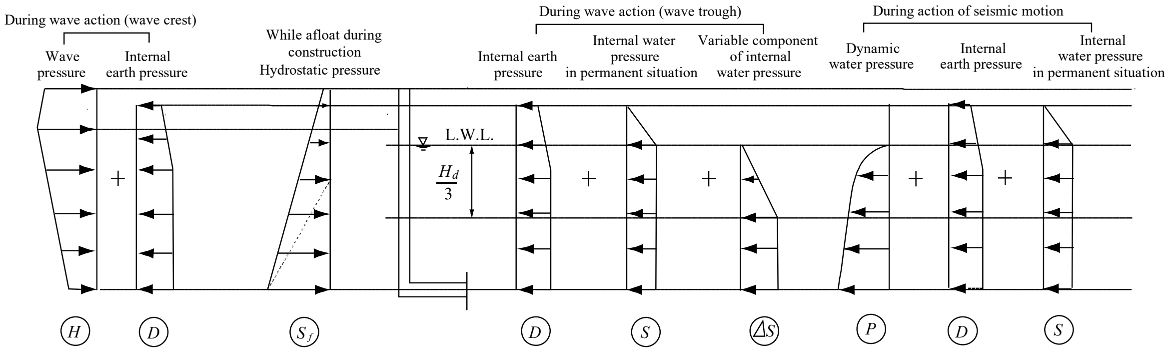
Table 2.2.1 Combinations of Actions and Load Factors
(a) Breakwaters

Situation	Design situation	Self-weight	Hydrostatic pressure	Internal earth pressure	Bottom slab reaction	Internal water pressure	Uplift	Variable component of bottom slab reaction	Variable component of internal water pressure	Wave force	Dynamic water pressure	Hydrostatic head difference between chambers	Remarks
In service	Permanent situation associated with self-weight	0.9 (1.0)	1.1 (1.0)		1.1 (1.0)								Bottom slab
	Permanent situation associated with internal earth pressure			1.1 (1.0)		1.1 (1.0)							Outer wall
	Variable situation associated with waves	1.1 [0.9] (1.0)	1.1 [0.9] (1.0)		1.1 [0.9] (1.0)		1.2 [0.8] (1.0)	1.2 [0.8] (1.0)					Bottom slab
				0.9 (1.0)						1.2 (1.0)			Outer wall
				1.1 (1.0)		1.1 (1.0)			1.2 (1.0)				
	Variable situation associated with Level 1 earthquake ground motion			1.0 (−)		1.0 (−)					1.0 (−)		Outer wall
During construction	Variable situation associated with water pressure while afloat	0.9 (0.5)	1.1 (0.5)										Bottom slab
			1.1 (0.5)										Outer wall
	Variable situation associated with water pressure during installation											1.1 (0.5)	Partition wall

(b) Quaywalls

Situation	Design situation	Self-weight	Hydrostatic pressure	Internal water pressure	Internal earth pressure	Bottom slab reaction	Surcharge	Dynamic water pressure	Bottom slab reaction during action of seismic motion	Loads during construction		Remarks
										Installation	Still water	
In service	Permanent situation associated with self-weight	0.9 (1.0)	1.1 (1.0)			1.1 (1.0)	0.8 (0.5)					Bottom slab (Surcharge is equivalent to bottom slab reaction component.)
	Permanent situation associated with internal earth pressure			1.1 (1.0)	1.1 (1.0)							Outer wall
	Variable situation associated with Level 1 earthquake ground motion	1.0 (−)	1.0 (−)				1.0 (−)		1.0 (−)			Bottom slab (Surcharge is that during action of seismic motion.)
				1.0 (−)	1.0 (−)			1.0 (−)				Outer wall
During construction	Variable situation associated with water pressure while afloat	0.9 (0.5)									1.1 (0.5)	Bottom slab while afloat
											1.1 (0.5)	Outer wall while afloat
	Variable situation associated with water pressure during installation									1.1 (0.5)		Partition wall during installation

① Actions to be considered in performance verification of outer walls of breakwater caissons are shown in **Figs. 2.2.4** to **2.2.6**. The standard values of the load factors are shown in **Tables 2.2.2** to **2.2.4**.



*In this figure, H_d stands for design wave height. In verification of the safety (against cross-sectional failure), H_d=H_{max} may be assumed.

Fig. 2.2.4 Actions on Front Wall (parallel to face line: seaward side) (Breakwater)

Table 2.2.2 Combinations of Actions and Load Factors for Front Wall (Breakwater)

Design situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with waves during action of wave crest	From outside of caisson	1.2H−0.9D	1.0H−1.0D
Variable situation associated with water pressure while afloat during construction		1.1S_f	0.5S_f
Variable situation associated with waves during action of wave trough	From inside of caisson	1.1D+1.1S+1.2ΔS	1.0D+1.0S+1.0ΔS
Variable situation associated with Level 1 earthquake ground motion		1.0D+1.0S+1.0P	Not examined

* For the symbols in the table, see **Fig. 2.2.4**.

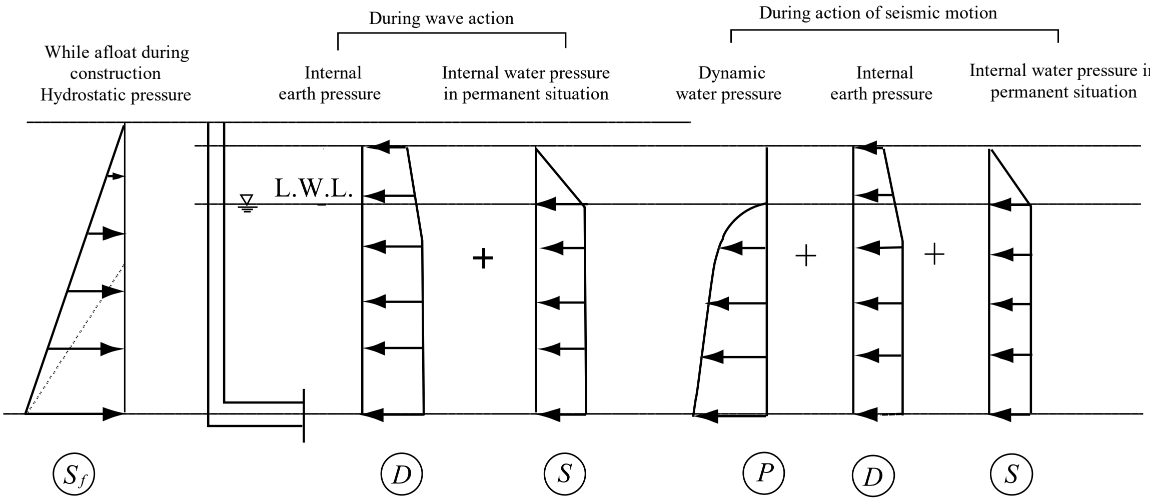
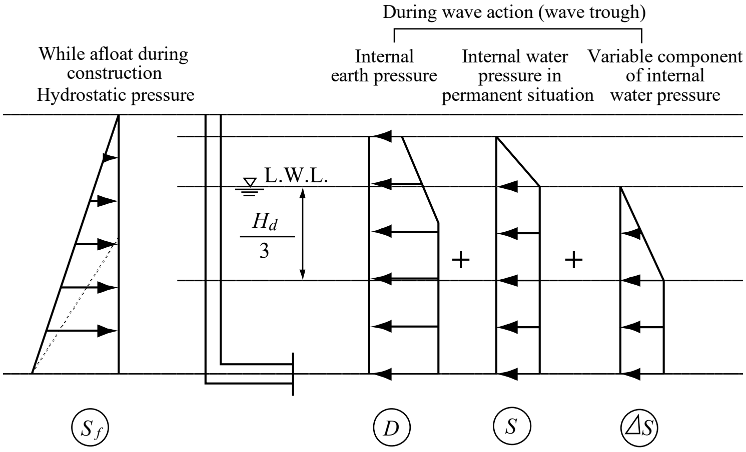


Fig. 2.2.5 Actions on Rear Wall (parallel to face line: landward side) (Breakwater)

Table 2.2.3 Combinations of Actions and Load Factors for Rear Wall (Breakwater)

Design situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure while afloat during construction	From outside of caisson	1.1S_f	0.5S_f
Permanent situation associated with internal earth pressure	From inside of caisson	1.1D+1.1S	1.0D+1.0S
Variable situation associated with Level 1 earthquake ground motion		1.0D+1.0S+1.0P	Not examined

* For the symbols in the table, see **Fig. 2.2.5**.



*In this figure, H_d stands for design wave height.

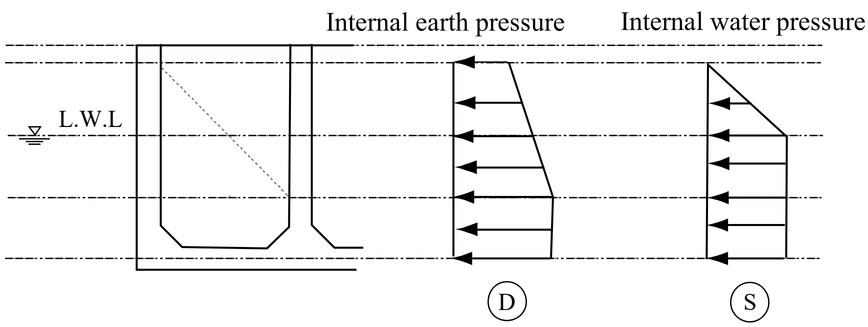
Fig. 2.2.6 Actions on Side Wall (perpendicular to face line) (Breakwater)

Table 2.2.4 Combinations of Actions and Load Factors for Side Walls (Breakwater)

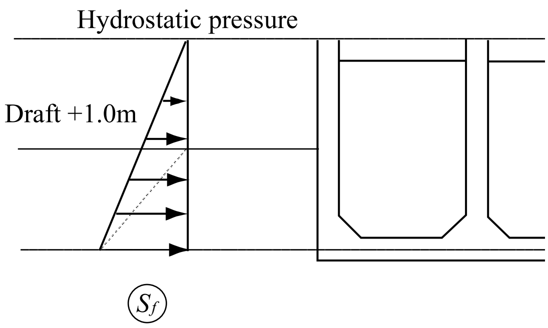
Design situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure while afloat during construction	From outside of caisson	$1.1S_f$	$0.5S_f$
Variable situation associated with waves during action of wave trough	From inside of caisson	$1.1D+1.1S+1.2\ \Delta S$	$1.0D+1.0S+1.0\ \Delta S$

* For the symbols in the table, see **Fig. 2.2.6**.

② Actions to be considered in performance verification of outer walls of quaywall caissons are shown in **Fig. 2.2.7**. The standard values of the load factors are shown in **Table 2.2.5**.

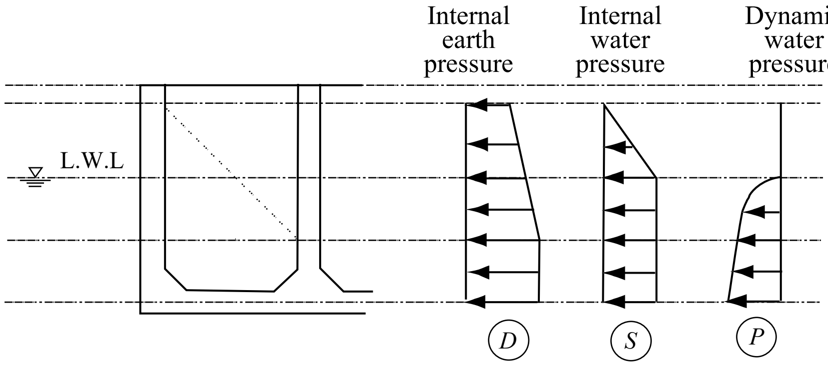


(a) Under calm conditions (actions from inside)



(b) While afloat (actions from outside)

Fig. 2.2.7 Actions on Outer Wall (Quaywall)



(c) During action of seismic motion (action to seaward side)

Fig. 2.2.7 Actions on Outer Wall (Quaywall)

Table 2.2.5 Combinations of Actions and Load Factors for Outer Wall (Quaywall)

Design situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure while afloat during construction	From outside of caisson	$1.1S_f$	$0.5S_f$
Permanent situation associated with internal earth pressure	From inside of caisson	$1.1D+1.1S$	$1.0D+1.0S$
Variable situation associated with Level 1 earthquake ground motion		$1.0D+1.0S+1.0P$	Not examined

* For the symbols in the table, see **Fig. 2.2.7**.

③ Actions to be considered in verification of the bottom slab of a breakwater caisson during construction can be determined by multiplying the characteristic values of the actions by the load factors shown in **Table 2.2.1**. In verification of a caisson in service, the composite load under calm conditions D_0, the variable component of bottom slab reaction ΔR and the uplift U shown in **Fig. 2.2.8** may be determined using the equations shown in **Table 2.2.7** in accordance with the classification of actions shown in **Table 2.2.6**.

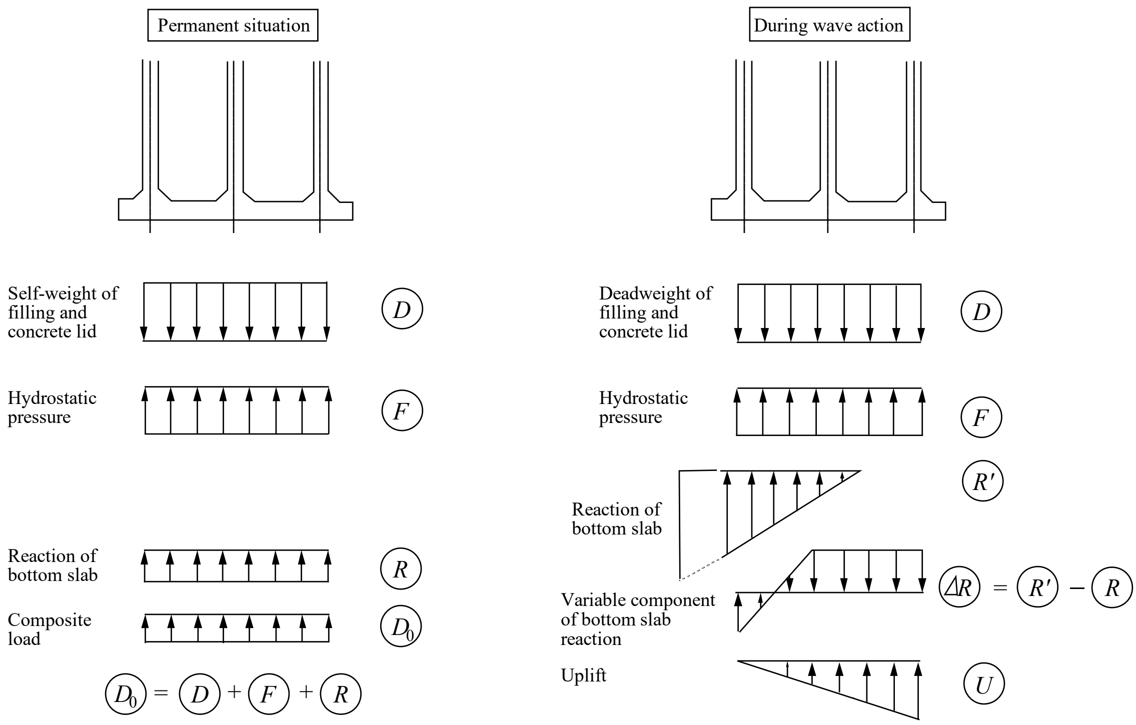


Fig. 2.2.8 Actions on Bottom Slab (Breakwater)

Table 2.2.6 Classifications of Actions during Wave Action (Breakwater)

Classification of action	Action
Permanent action	Composite load under calm conditions D_0
Variable action	Variable component of bottom slab reaction ΔR, Uplift U

Table 2.2.7 Combinations of Actions and Load Factors (Breakwater)
(a) Safety (against cross-sectional failure)

Design situation	Direction of ΔR and W				Combination of actions
Permanent situation	−				$0.9D_0+1.1F+1.1R$
Variable situation associated with water pressure while afloat during construction	−				$0.9D_0+1.1F$
Variable situation associated with waves during action of wave crest	ΔR	↑	W	↑	$1.1D_0+1.2\ \Delta R+1.2U$
	ΔR	↓	W	↑	$1.1D_0+0.8\ \Delta R+1.2U$
			W	↓	$0.9D_0+1.2\ \Delta R+0.8U$
Variable situation associated with waves during action of wave trough	ΔR	↑	W	↑	$1.1D_0+1.2\ \Delta R+0.8U$
			W	↓	$0.9D_0+0.8\ \Delta R+1.2U$
	ΔR	↓	W	↑	$1.1D_0+0.8\ \Delta R+0.8U$
			W	↓	$0.9D_0+1.2\ \Delta R+1.2U$

(b) Serviceability

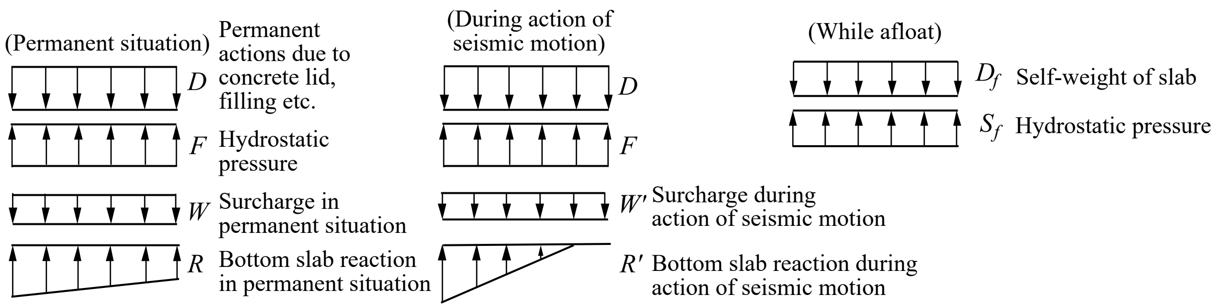
Design situation	Combination of actions
Permanent situation	$1.0D_0+1.0F+1.0R$
Variable situation associated with waves	$1.0D_0+1.0\ \Delta R+1.0U$

Note that $W=D_0+\Delta R+U$ is assumed to hold. Each action is represented as a signed value, which is positive for an action in the same direction as W or negative for an action in a direction opposite to W. For the symbols in the table, see **Fig. 2.2.8**.

Note: When the variable component of bottom slab reaction (ΔR) acts downwards and $1.2|\Delta R|>1.1|R|$ holds, the combination of actions shall be as follows:

$$0.9D_0+1.1|R|+0.8U \text{ or } 0.9D_0+1.1|R|+1.2U$$

④ Actions to be considered in verification of the stability of the bottom slab of a quaywall caisson during construction can be determined by multiplying the characteristic values of the actions by the load factors shown in **Table 2.2.1**. Actions to be considered in verification of the stability in service can be determined by using the equations shown in **Table 2.2.8** in consideration of the combinations of actions shown in **Fig. 2.2.9**.



Permanent situation: Situation in which surcharge is imposed.

Fig. 2.2.9 Actions on Bottom Slab (Quaywall)

Table 2.2.8 Combinations of Actions and Load Factors (Quaywall)

Design situation	Safety (against cross-sectional failure)	Serviceability
Permanent situation	$0.9D+1.1D_O+1.1F+0.8W$	$1.0D+1.0D_O+1.0F+0.5W$
Variable situation associated with Level 1 earthquake ground motion	$1.0D+1.0F+1.0R'+1.0W'$	Not examined
Variable situation associated with water pressure while afloat during construction	$0.9D_f+1.1S_f$	$0.5D_f+0.5S_f$

* For the symbols in the table, see **Fig. 2.2.9**.

⑤ As the action to be considered in verification of partition walls during construction, the hydrostatic head difference between chambers during construction (during installation) shall be used in principle.

⑥ As the action to be considered in verification of partition walls in service, the action in the state where extrusion force becomes the largest among the actions related to the bottom slab and actions related to the outer walls shall be used in principle.

(4) Actions to be considered in performance verification of caissons during fabrication may be set as follows.

① When a caisson is fabricated on a dry dock, floating dock or the like, it is unnecessary to examine the actions during fabrication. However, when the caisson is raised with jacks to move it on a slipway or caisson platform, or loaded on a launch truck, its self-weight acts as a concentrated load.

② When examination of a caisson during fabrication is necessary, it may be performed by assuming that the whole caisson is a beam.

(5) Actions to be considered in verification of caissons during launching and floating may be set as follows.

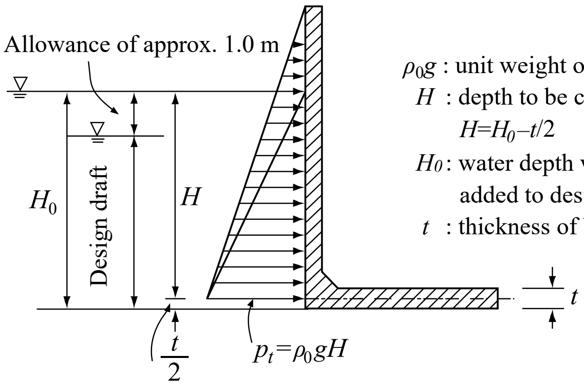
① In cases where a dry dock, floating dock or slipway is used, the hydrostatic pressure with an allowance added to the design draft may be used as the action during launching and floating. In cases where there is a danger that a greater hydrostatic pressure may act on the caisson temporarily during launching, separate examination is necessary.

② When a caisson is slid into water from a slipway or the like, not only the hydrostatic pressure but also the dynamic water pressure act on the caisson. When a caisson on a launch truck is put into water by using a winch or braking post, the speed is generally 3 to 5 m per minute and is not large enough to cause the dynamic water pressure to act on the caisson. However, depending on the inclination of the slipway, the front side of the caisson is subjected to the hydrostatic pressure equivalent to that at the water level deeper than the draft by 1 to 1.5 m, although the duration is very short. When a caisson is launched from a slipway, the dynamic water pressure acts on it, but this is a temporary action and there has been no study that provides measured values. In view of this, it is enough to add an allowance of about 1.0 m to the draft as measures against the dynamic water pressure. Note that the allowance of 1.0 m was determined in consideration of the facts described below in (a) and (b).

(a) When a caisson is launched at a dry dock, floating dock or the like, the extra hydrostatic pressure acts on the caisson at the moment when the bottom slab leaves the platform. This hydrostatic pressure is generally equivalent to that at the water level deeper than the draft by 0.1 to 0.4 m.

(b) The draft of a caisson increases by about 0.2 to 0.3 m due to bulge of the formwork during concrete placement. The draft also increases or decreases by about 0.2 to 0.3 m due to a difference between the calculated and actual unit weights of reinforced concrete.

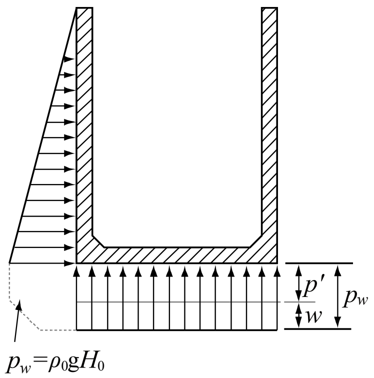
③ The water pressure acting on outer walls may be considered as a load with a triangular distribution in which the base is the distance to the crown and the height is the intensity of the hydrostatic pressure at the centerline of the bottom slab (p_t) as shown in **Fig. 2.2.10**.



$\rho_0 g$: unit weight of sea water (kN/m3)
H : depth to be considered in calculation of hydrostatic pressure (m)
$H=H_0-t/2$
H_0 : water depth with an allowance of approximately 1.0 m added to design draft (m)
t : thickness of bottom slab (m)

Fig. 2.2.10 Water Pressure Acting on Outer Wall

④ As the action on the bottom slab, the value obtained by subtracting the self-weight of the bottom slab from the intensity of the hydrostatic pressure at the bottom edge of the bottom slab (p_w) shall be used as shown in **Fig. 2.2.11**.



$p'=p_w-w=\rho_0 gH_0-w$
p' : action on bottom slab (kN/m^2)
p_w : hydrostatic pressure acting on bottom slab with an allowance of approximately 1.0 m added to design draft (kN/m^2)
w : self-weight of bottom slab (kN/m^2)
$\rho_0 g$: unit weight of sea water (kN/m^3)
H_0 : water depth with an allowance of approximately 1.0 m added to design draft (m)

Fig. 2.2.11 Actions on Bottom Slab

⑤ When a partition wall has a thickness of 0.2 m or more, it generally has sufficient bearing strength as a column. Thus, the examination of the bearing strength of the partition wall may be omitted.

⑥ When a caisson is launched from a steeply-sloped slipway, the whole caisson will sink under water. Therefore, it may be necessary to attach a temporary lid to the caisson.

⑦ When a caisson is craned for launching, its outer walls are subjected to different actions depending on whether lifting accessories are used. Therefore, it is necessary to examine the actions that can occur with or without lifting accessories.

(6) Actions to be considered in performance verification of caissons during towing may be set as follows.

① It is unnecessary to take account of the hydrostatic pressure, dynamic water pressure and wave pressure that act on caissons while they are towed.

② The tensile force during towing of caissons can be calculated using **equation (2.2.5)**. (See **Fig. 2.2.12**.)

$$T=\frac{1}{2}\rho_0 C_D V^2 A \tag{2.2.5}$$

where

T : design value of tensile force during towing (kN); this value may be calculated by assuming that the partial factor to be used for multiplying the action term is 1.0.

C_D : drag coefficient

V : towing speed (m/s)

A : wetted surface area on caisson front side (m^2), $A=a(D+\delta)$

a : width of caisson (m)

D : draft (m)

δ : water level on front side (m)

ρ_0 : density of sea water (t/m^3)

l : length of caisson (m)

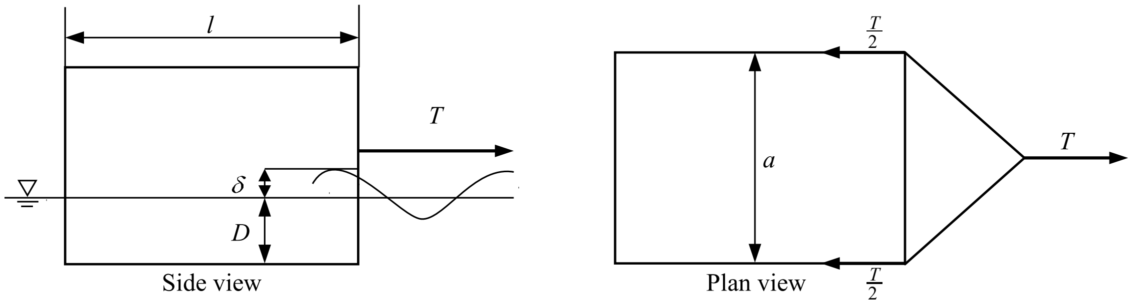


Fig. 2.2.12 Tensile Force during Towing

③ Since caissons have no superstructure like the ones of ships and towing will not take place in a strong wind, it is enough to consider only the fluid resistance by taking no account of the wind resistance.

④ Though the drag coefficient varies depending on the shape of the surface perpendicular to the current, the drag coefficient for the rectangular board that is given in **Table 7.2.1** in **Part II**, **Chapter 2**, **7.2 Fluid Force due to Current** may be used.

⑤ The towing speed is generally 2 to 3 knots.

⑥ Caissons being towed are generally subjected to the pressure resistance and the wave making resistance. However, in view of the fact that towing will not take place when waves are high, the verification of caissons under the water pressure during towing may be omitted provided that an allowance of 1.0 m is added to the draft.

(7) Actions to be considered in performance verification of caissons during installation may be set as follows.

① The water pressure caused by the hydrostatic head difference between chambers shall be set as the action on partition walls, considering construction conditions.

② Caissons may be put under water by filling them with water using a siphon, pump, valve or the like. When using a valve, it is enough to take account of the hydrostatic head difference of 1.0 m. When using a siphon or pump, it is desirable to keep the hydrostatic head difference within 1.0 m through supervision of construction work, for example, by moving the hose frequently.

③ A caisson shall be installed by pouring water into it first, and pouring a filling material into it after the water levels in all the chambers have reached the crest of the caisson. When pouring the filling material into the caisson, it is necessary to take care not to cause a difference in the earth pressure. The filling material is subjected to buoyancy, so it is unnecessary to consider the action of the filling material on partition walls during installation provided that the hydrostatic head difference caused by the filling material does not exceed about 1.6 times the hydrostatic head difference that occurs while water is poured into the caisson.

(8) Actions to be considered in performance verification of caissons in service may be set as follows.

① As actions on outer walls, the internal earth pressure and the internal water pressure shall be considered. For outer walls of breakwater caissons, the influence of the actions of waves shall also be considered. In addition to the actions of waves, breakwaters covered with wave-dissipating blocks are also affected by the impact of the wave-dissipating blocks against the front wall, and depending on the region, by the impact load of drift ice, driftwood and other drifting objects, ice formations, and other factors. Therefore, when these influences are remarkable, they must be considered as actions.

② **Internal earth pressure**

(a) It can be assumed that the internal earth pressure increases as the depth increases, but does not increase any more after the depth becomes larger than the inner width *b* of the wall as shown in **Fig. 2.2.13**.

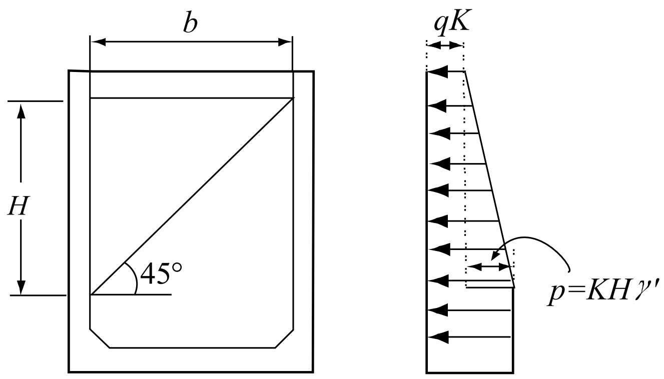


Fig. 2.2.13 Determination of Internal Earth Pressure

(b) In the case where sand or rubble is used as a filling, the coefficient of earth pressure at rest *K* can be generally set at 0.6. However, the internal earth pressure may be disregarded when the filling consists of blocks or concrete.

(c) In cases where strong cast-in-place concrete is located on top of caissons and it can be regarded that the effect of the surcharge does not reach the filling, the surcharge may be disregarded. However, the self-weight of the cast-in-place concrete shall be taken into account.

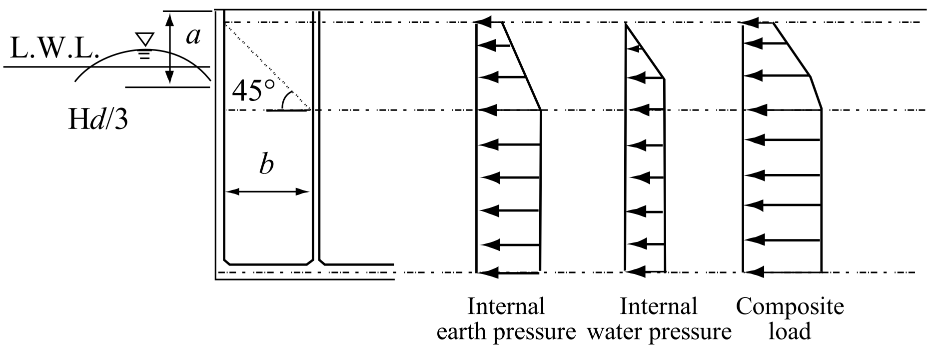
(d) The way of determining the internal earth pressure shown in **Fig. 2.2.13** was established for convenience based on past records and experiments, not based on measurements. For example, according to **Reference 7)**, the earth pressure distribution of filling sand increases almost monotonically as the depth increases, and the coefficient of earth pressure is about 0.3 to 0.35 or about 0.4 when the ratio of wall height *H* to wall width *B*, *H*/*B*, is 4 or more or 2 or less respectively. This can be used as a reference in performance verification of a caisson. However, when determining the internal earth pressure in this way, it is necessary to ensure that the caisson has sufficient stability against combinations of other actions.

③ **Internal water pressure**

The internal water pressure shall be considered as the head difference between the water level in the caisson and the lowest water level (LWL). In verification of the front wall of a breakwater caisson and its side walls perpendicular to the face line, the external water level may be considered as the difference between LWL and (H_{max})/3 when the wave troughs act on the surface of the front wall, as shown in **Fig. 2.2.14(a)**. The internal water pressure may be disregarded when the wave crests act on the surface of the front wall. For the rear wall, the external water level may be considered as LWL as shown in **Fig. 2.2.14(b)**.

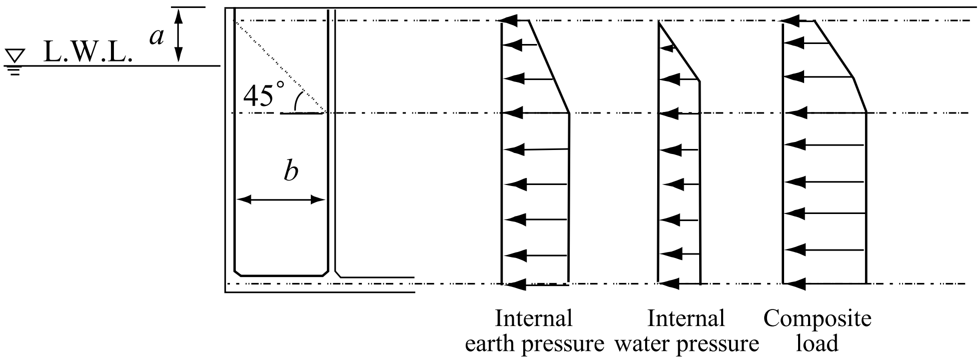
④ For the front wall of a breakwater caisson, the wave force shall be taken into account when wave crests act on the wall surface.[8), 9)]

⑤ Determination of the internal earth pressure and the internal water pressure in each structural member is as shown in **Fig. 2.2.14**.

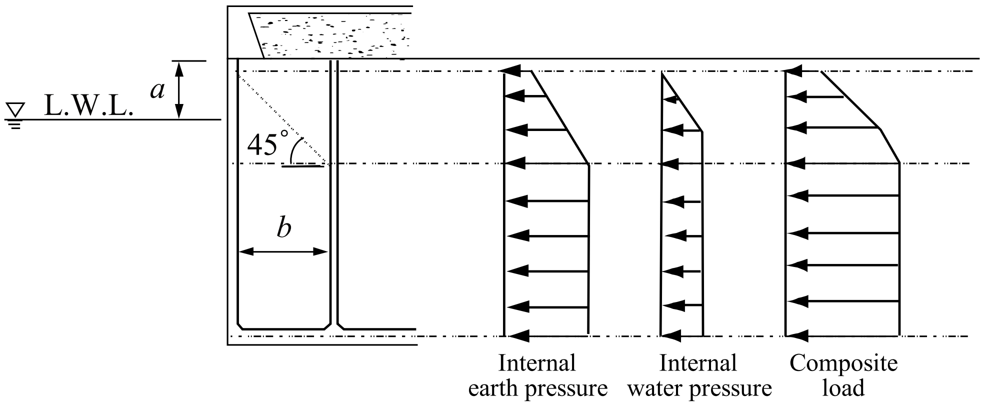


*In this figure, H_d stands for design wave height.
In verification of the safety (against cross-sectional failure), $H_d = H_{max}$ may be assumed.

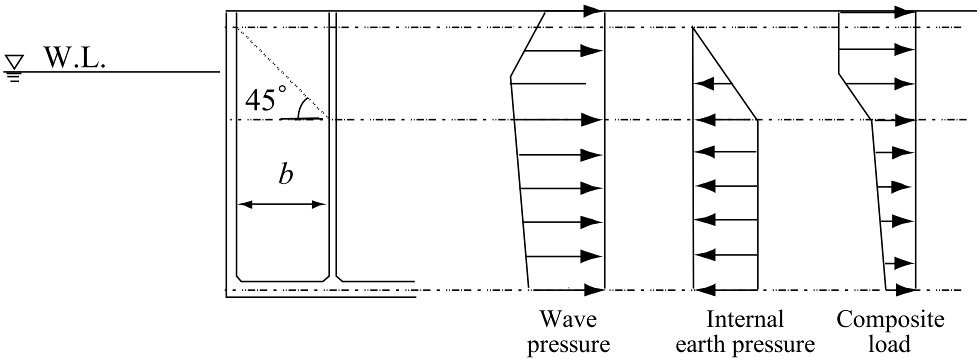
(a) Breakwaters (front wall and side walls perpendicular to face line)



(b) Breakwaters (rear wall)



(c) Quaywalls (front wall, rear wall, and side walls perpendicular to face line)



(d) Actions of waves

Fig. 2.2.14 Determination of Internal Earth Pressure and Internal Water Pressure and Actions of Waves

(9) Actions to be considered in performance verification of the bottom slab may be set as follows.

① For fixed parts surrounded by outer walls and partition walls, the bottom reaction, the hydrostatic pressure, the uplift, the weight of the filling material, the weight of the concrete lid, the weight of the bottom slab, and the surcharge shall be taken into account.

② The bottom reaction acting on a caisson or wall body can be calculated by using **equation (2.2.7)** or **(2.2.8)** in accordance with the relationship between the eccentricity of total resultant force e and the width of the bottom b calculated by using **equation (2.2.6)**, as shown in **Fig. 2.2.15**.

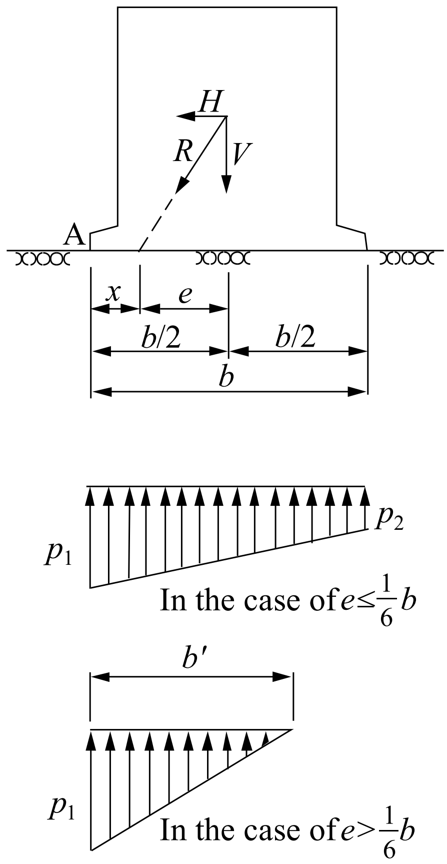


Fig. 2.2.15 Bottom Reaction

$$\left.\begin{aligned} e &= \frac{b}{2} - x \\ x &= \frac{M_w - M_h}{V} \end{aligned}\right\} \qquad \textbf{(2.2.6)}$$

where

e : eccentricity of total resultant force (m)

b : width of bottom (m)

V : characteristic value of vertical resultant force per unit length in direction of caisson face line (kN/m)

H : characteristic value of horizontal resultant force per unit length in direction of caisson face line (kN/m)

M_w : characteristic value of moment around point A due to vertical resultant force (kNm/m)

M_h : characteristic value of moment around point A due to horizontal resultant force (kNm/m)

(a) In the case of $e \leq \frac{1}{6}b$

$$\left.\begin{aligned} p_1 &= \left(1+\frac{6e}{b}\right)\frac{V}{b} \\ p_2 &= \left(1-\frac{6e}{b}\right)\frac{V}{b} \end{aligned}\right\} \tag{2.2.7}$$

(b) In the case of $e > \frac{1}{6}b$

$$\left.\begin{aligned} p_1 &= \frac{2}{3}\frac{V}{\left(\frac{b}{2}-e\right)} \\ b' &= 3\left(\frac{b}{2}-e\right) \end{aligned}\right\} \tag{2.2.8}$$

where

p_1 : characteristic value of reaction at front toe (kN/m^2)

p_2 : characteristic value of reaction at rear toe (kN/m^2)

b' : action width of bottom reaction in the case of $e > \frac{1}{6}b$

③ The hydrostatic pressure shall be the water pressure acting on the caisson bottom slab at the design tide level.

④ The uplift shall be taken into account in cases where waves act on a caisson or wall body. For calculating the uplift, refer to **Part II**, **Chapter 2, 6 Wave Force**.

⑤ The unit weight of the filling material is normally determined by testing the material to be used.

⑥ The weight of the concrete lid and bottom slab shall be the weight without consideration of buoyancy.

⑦ The surcharge acting on the bottom slab of a caisson includes the weight of soil on top of the caisson and the live load. However, the surcharge may be disregarded in the case where cast-in-place concrete is placed on top of the caisson and it can be regarded that the influence of the surcharge does not reach the bottom slab.

⑧ In performance verification of the bottom slab, the action on it can be considered to be linearly distributed as shown above. In reality, however, the bottom reaction is uneven and discrete due to roughness of the mound surface. According to results of various tests including a loading test, the degree of discretization of the bottom reaction varies depending on the design situation.[10)] Refer to **Reference 5)**, in which authors presented a reliability analysis conducted by using a stochastic model developed for distribution of bottom slab reaction based on the said test results, and proposed load factors that can be used in verification of the bottom slab in cases where the mound leveling accuracy deviates from the standard value.

(10) Actions to be considered in performance verification of footings may be set as follows.

① The bottom reaction, the weight of the footings, and the surcharge on the footings shall be taken into account. Actions may be set considering the distributions shown in **Fig. 2.2.16**.

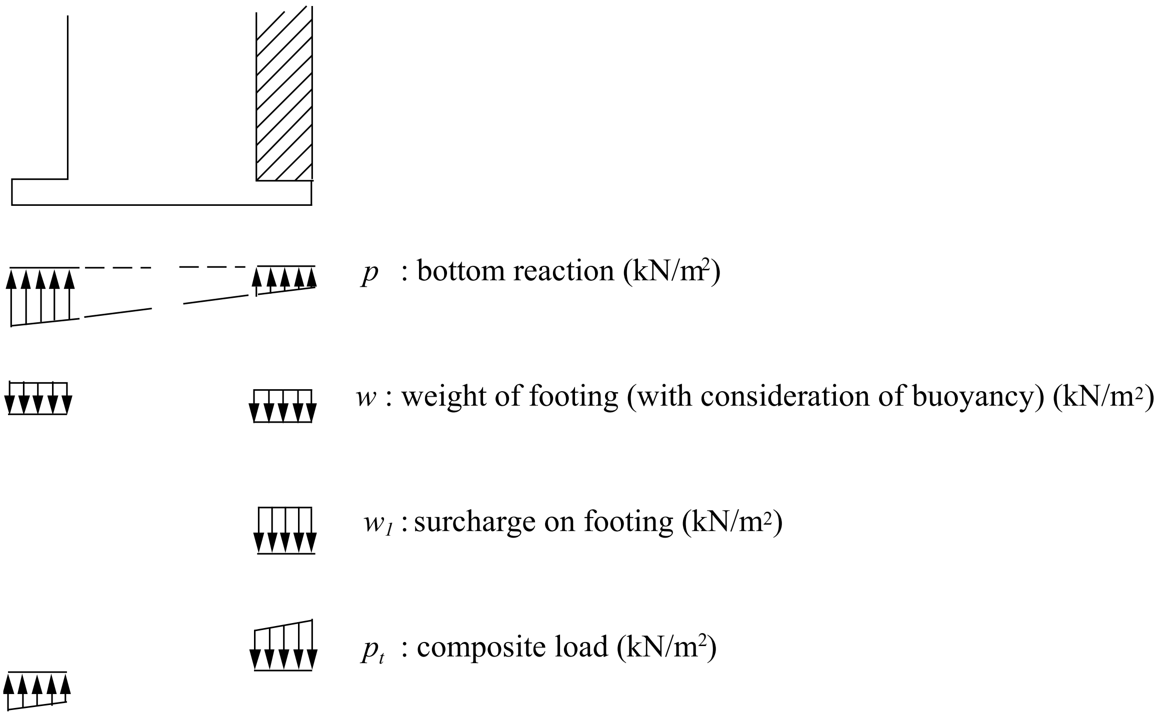


Fig. 2.2.16 Actions on Footings

② For the bottom reaction acting on footings, the values calculated using **equation (2.2.7)** or **(2.2.8)** can be used.

③ The weight of a footing shall be the submerged weight with consideration of buoyancy.

④ As the surcharge on footings, the weight of wave-dissipating blocks with consideration of buoyancy below the design water level, the weight of overburden soil on the land side of a quaywall, the live load, and other loads shall be considered, depending on the type of facility.

(11) Actions to be considered in performance verification of partition walls may be set as follows.

① In verification for extrusion of outer walls from partition walls, the internal earth pressure and internal water pressure acting on the outer walls shall be considered. It may be assumed that these act on the joints between the partition walls and the outer walls (see **Fig. 2.2.17**).

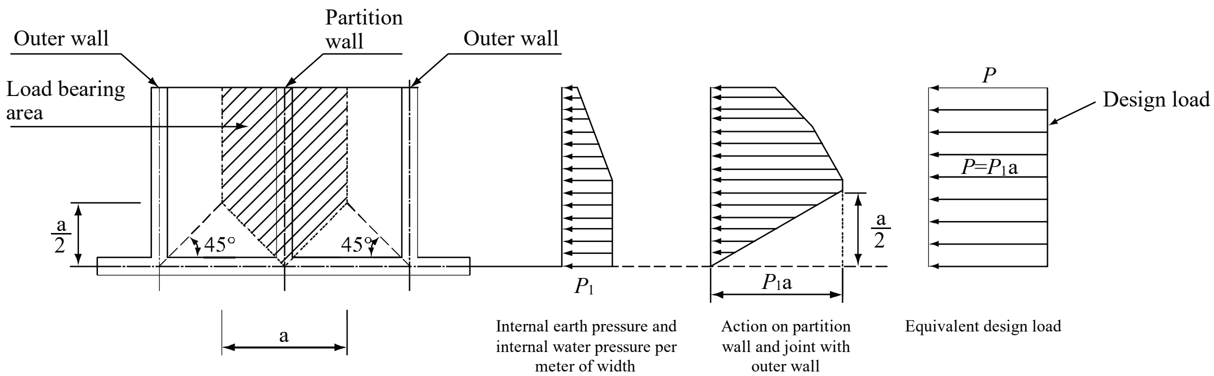


Fig. 2.2.17 Actions to be Considered in Examination of Extrusion of Outer Wall from Partition Wall

② In verification for extrusion of the bottom slab from partition walls, the weight of the filling material acting on the bottom slab, the surcharge, the weight of the bottom slab, the weight of the concrete lid, the bottom reaction, the uplift, and the hydrostatic pressure shall be taken into account. It may be assumed that these act on the joints between the partition walls and the bottom slab (see **Fig. 2.2.18**).

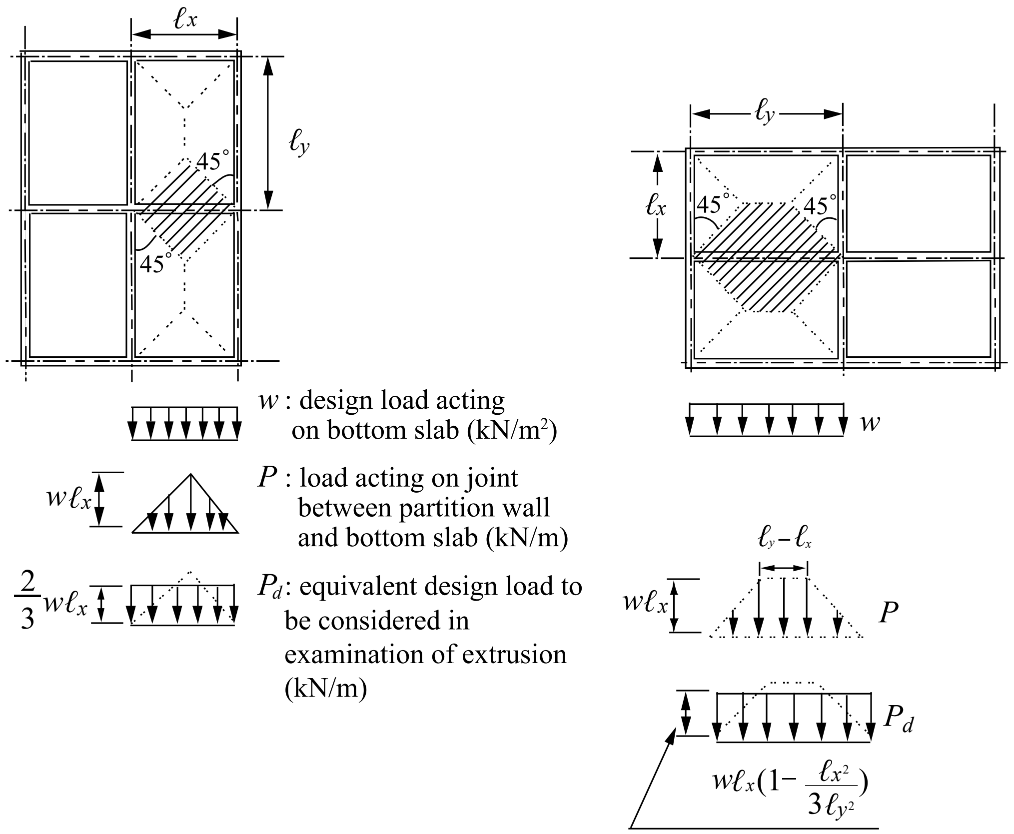


Fig. 2.2.18 Actions to be Considered in Examination of Extrusion of Bottom Slab from Partition Wall

③ If there is a possibility that a caisson might be subjected to an action caused by non-uniformity of the supporting soil layer, this action shall be examined. In this case, verification of the individual members of the caisson may be performed assuming that they are cantilevers with a span equivalent to 1/3 of the length or width of the caisson (see **Fig. 2.2.19**). Verification may also be performed using a structural analysis model in which only the parts of the ground which can be expected to have the bearing capacity are replaced with ground springs.

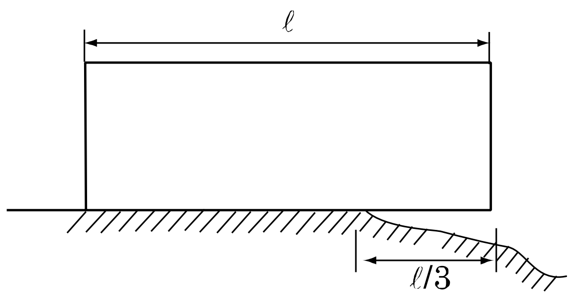


Fig. 2.2.19 Examination of Action due to Non-uniformity of Ground Bearing Capacity

④ The standard load factors for actions to be considered in verification of partition walls are shown in **Table 2.2.9**.

Table 2.2.9 Combinations of Actions and Load Factors

Design situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure during installation during construction	Direction of action due to hydrostatic head difference between chambers	$1.1S_f$	$0.5S_f$
Permanent situation associated with internal earth pressure	Direction of extrusion of outer wall from partition wall	Maximum outward design load that acts on outer wall	Not examined
Permanent situation associated with self-weight Variable situation associated with waves Variable situation associated with Level 1 earthquake ground motion	Direction of extrusion of bottom slab from partition wall	Maximum downward design load that acts on bottom slab	Not examined

2.2.4 Performance Verification

(1) Performance verification of structural members shall be performed based on **Part III**, **Chapter 2**, **1.1 General**.

① Performance verification of structural members shall be performed by setting the verification indices for the corresponding limit states for the actions on the members calculated using the methods described in **Chapter 2**, **2.2.3 Actions**. The settings of the verification indices shall be based on **Part III**, **Chapter 2**, **2 Structural Members**. The partial factors to be used in the performance verification may generally be set based on **Table 1.1.1** in **Chapter 2**, **1.1.3 Partial Factors**.

② The cover of reinforcing bars for caissons is set to a value equal to or larger than the standard value shown in **Table 1.2.5**. Examination of changes in performance over time may be omitted in cases where caissons are constructed carefully using concrete with the water-cement ratio shown in **Table 3.2.2** in **Part II**, **Chapter 11**, **3.2 Concrete Quality and Performance Characteristics** and the design service life is set to about 50 years.

(2) In performance verification of structural members, sectional forces may be determined by modeling the structural members as slabs fixed on three sides and free on one side or slabs fixed on four sides according to constraint conditions and making calculations based on **References (Part III)**, **Chapter 4**, **2 Tables for Calculating Bending Moments in Slabs**. The sectional forces may also be calculated by using the finite element method or other structural analysis techniques, regardless of the descriptions below in (3) through (7).

(3) Performance verification of outer walls can be performed as follows:

① An outer wall can be assumed as a slab fixed on three sides and free on one side. It can also be assumed as a slab fiexed on four sides in cases where sufficient reinforcing bars are placed at joints and sectional forces can be smoothly transmitted between the outer wall and the concrete lid.

② When the ratio of longer to shorter span of an outer wall is 5 or more, sectional forces can be calculated by using the values for a slab where the ratio of longer to shorter span is 5.

③ The values of unbalanced moments between outer walls and the bottom slab can be used directly without distribution.

④ The span to be considered in calculations shall be a center-to-center distance in principle. According to test results given in **Reference 11)**, fixed points of members of a caisson outer wall are located inside the haunch and within the inner width of the outer wall.

⑤ In cases where extremely large unbalanced moments occur at points that are regarded as fixed points between outer walls, the bending moments at the edges of the outer walls may be distributed based on the slab stiffness ratio and the span moments may be corrected by adding one half of a distributed moment. For internal supporting points and spans except the first span, it is unnecessary to distribute unbalanced moments because the effect of distribution is small (see **Fig. 2.2.20**).

⑥ **Fig. 2.2.20** shows an example of distributing unbalanced moments that occur between outer walls, and **equation (2.2.9)** expresses the moments after distribution.

$$\left.\begin{aligned}
M'_{BA} &= M_{BA} - \left(M_{BA} - M_{BC}\right)\frac{K_a}{K_a + K_b} \\
M'_{BC} &= M_{BC} + \left(M_{BA} - M_{BC}\right)\frac{K_b}{K_a + K_b} \\
M'_a &= M_a - \frac{1}{2}\left(M_{BA} - M_{BC}\right)\frac{K_a}{K_a + K_b} \\
M'_b &= M_b + \frac{1}{2}\left(M_{BA} - M_{BC}\right)\frac{K_b}{K_a + K_b} \\
M'_{AB} &= M_{AB} \\
M'_{CB} &= M_{CB}
\end{aligned}\right\} \tag{2.2.9}$$

where

M'_{AB}, M'_{BA}, M'_{BC}, M'_{CB}, M'_a, M'_b :bending moments after distribution of unbalanced moments (kN・m)

M_{AB}, M_{BA}, M_{BC}, M_{CB}, M_a, M_b :bending moments before distribution of unbalanced moments (kN・m)

K_a, K_b : relative stiffness of outer wall

It should be noted that moments are positive or negative signed values.

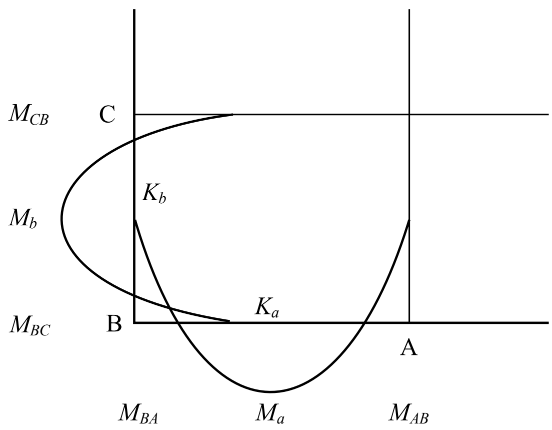


Fig. 2.2.20 Example of Distribution of Unbalanced Moments

(4) Performance verification of partition walls can be performed as follows.

① During installation, a partition wall can be regarded as a slab supported on three sides and free on one side.

② The span to be considered in calculations shall be the distance between the centerlines of walls.

(5) Performance verification of the bottom slab and footings can be performed as follows.

① The part of the bottom slab surrounded by outer walls and partition walls can be regarded as a slab fixed on four sides. Footings can be regarded as cantilever slabs.

② The span to be considered in calculation of a slab fixed on four sides shall be a center-to-center distance in principle.

③ The cross section to be considered in calculations in connection with bending and shearing of a footing shall be the front surface of the wall. However, the cross section to be considered in examination of diagonal tensile shear failure may be assumed to be the cross section at the base of the front face of the wall. In this case, the part of the haunch where the gradient is shallower than 1:3 shall be considered effective in calculations of the height of members at the front face of the wall.

④ In the case of reinforced concrete footings of normal dimensions, the caisson body is assumed to be rigid; therefore, it may be considered that the moments occurring in the footings do not reach the caisson body.

(6) Other Structural Members

The performance verification methods described in **Part III**, **Chapter 2**, **2 Structural Members** shall be applied correspondingly to slit members of slit caissons and other structural members that are not covered in the said section, considering the dimensions of the structural member to be verified, the characteristics of the actions on it, and other factors.

(7) Others

① In the case of quaywall caissons, verification of the safety (against fatigue failure) may be omitted in principle.

② In cases where a caisson is to be lifted with a jack or other device for transportation or there is a possibility that uneven settlement might occur after installation of a caisson, verification may be performed considering the entire caisson as a beam. In this case, verification for punching shear of the bottom slab is necessary.

③ Outer walls of breakwaters covered with wave-dissipating blocks might exhibit local failure due to repeated collisions of wave-dissipating blocks. Refer to **Reference 12)**, which provides methods for design and verification against local failures of caisson outer walls.

2.2.5 Verification of Suspension Hooks during Lifting

(1) The load to be caused by one suspension hook shall be determined appropriately in consideration of the weight of the caisson to be lifted, the adhesion acting on the bottom surface of the caisson, and other conditions.

① The action on one suspension hook can be determined by using **equation (2.2.10)**.

$$P_d = \frac{W + W' + F}{N \sin\theta} k \qquad \textbf{(2.2.10)}$$

where

P_d : design value of action on one suspension hook (kN); this value may be calculated by assuming that the partial factor to be used for multiplying the action term is 1.0.

W : characteristic value of weight of caisson (kN)

W' : characteristic value of additional weight of caisson (kN)
$W'=0.05W$

F : characteristic value of bottom friction of caisson (kN)
$F=3.0A$

A : bottom area of caisson (m^2)

k : imbalance coefficient

N : number of suspension hooks

θ : angle formed by rope and top surface of caisson (º); this angle may be assumed to be 90º in cases where it will be used for calculating the embedded length of suspension hooks or a suspension frame will be installed.

② The imbalance coefficient k may be generally set to 1.8. According to results of measuring the actions that occurred during lifting of actual caissons, the maximum imbalance coefficient was 1.24 and 1.56 for 3,300-kN caissons (8-point lifting) and 9,800-kN caissons (16-point lifting) respectively, and the overall average was 1.36. The general value of 1.8 means that the probability of exceeding 1.8 is about 0.3%. The imbalance coefficient is set to a larger value for lifting with a smaller number of suspension hooks, and this value may be reduced when it is considered appropriate to do so based on results of a lifting test or the like. According to results of measuring the actions that occurred during lifting of an actual caisson, the weight of the caisson was slightly larger than the design value due to bulge and stagnant water in the caisson, and the bottom adhesion was slightly smaller than the design value. As a whole, the measurement results of actions were almost equal to design values.

③ The load factor may be assumed to be 1.0 on condition that the imbalance coefficient has been set appropriately.

④ When the unit weight of a caisson was assumed to be 24.0 kN/m^3 and stagnant water in the caisson was taken into consideration, the bulge of the caisson by weight was 4.5% and 5.8% on average for 3,300-kN and 9,800-kN caissons respectively and the standard deviation was 2.2%. The overall average was 5.5% and the standard deviation was 2.1%. From these results, the additional weight of a caisson was set to 5% of the design value of the weight of the caisson.

⑤ When sand mat and geotextile fabric were laid underneath a caisson, the bottom adhesion was in a range from 0 to 1.2 kN/m^2 with the average of 0.45 kN/m^2 and the standard deviation of 0.47 kN/m^2. However, it is expected that the actual bottom adhesion will be significantly affected by the shape of the bottom surface and by sand mat and other underlaid materials. In view of this, the design value of the bottom adhesion per unit area was set to 3.0 kN/m^2.

(2) Verification in cases where plain bars are used for suspension hooks

① Verification of suspension hooks may be performed by using **equation (2.2.11)** and using the shear yield resistance of the suspension hooks or the bonding and fixing resistance of the embedded parts of the suspension hooks, whichever is smaller (see **Fig. 2.2.21**).

$$\gamma_i \frac{P_d}{\min(T_{1_d}, T_{2_d})} \leq 1.0$$

$$T_{1_d} = \frac{\pi D^2 f_{vy_d}}{2 \cdot 10^3 \gamma_b} \qquad \textbf{(2.2.11)}$$

$$T_{2_d} = \frac{2\pi D f_{bo_d} m \alpha l}{10^3 \gamma_b}$$

where

γ_i : structure factor

T_{1d} : design value of shear yield resistance of suspension hook (kN)

T_{2d} : design value of bonding and fixing resistance of embedded part of suspension hook (kN)

P_d : design value of action on one suspension hook (kN); this value may be calculated by using **equation (2.2.10)**.

D : diameter of suspension hook (mm)

f_{vyd} : design value of shear yield strength of suspension hook (N/mm^2); this value may be calculated by assuming that the material factor γ_c is 1.0.

f_{bod} : design value of bond strength of concrete (N/mm^2); this value may be calculated by assuming that the material factor γ_c is 1.0.

m : effect of hook; this may be generally assumed to be 1.5.

α : factor to be used for taking account of bond strength of plain reinforcing bars; this value may be generally set to 1.1.

l : embedded length of suspension hook (mm)

γ_b : member factor (= 1.1)

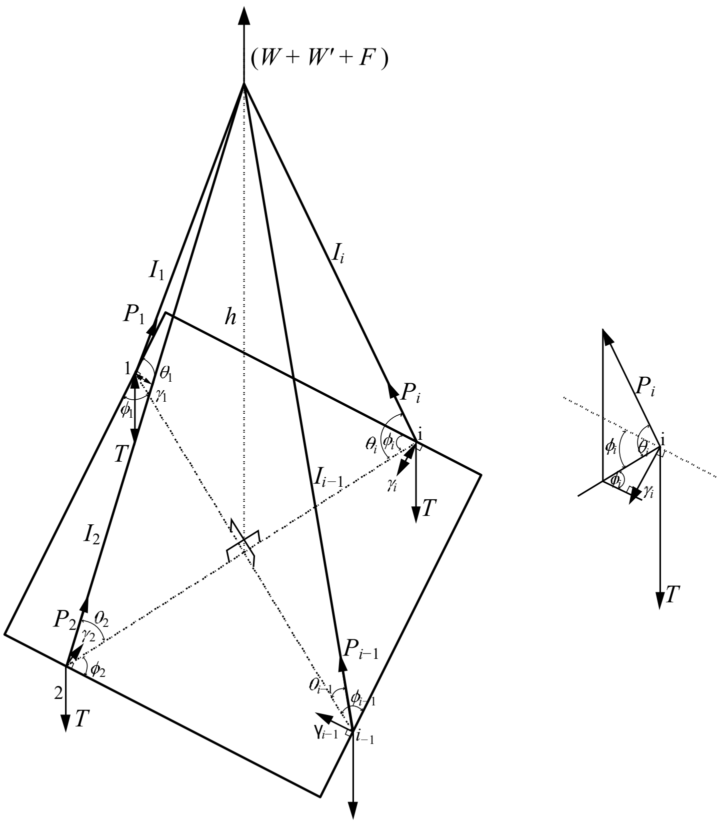


Fig. 2.2.21 Actions on Suspension Hooks during Direct Lifting

② It was found in full-scale failure tests of caisson suspension hooks [11), 13)] that caisson walls crack in different ways before the failure load is reached. However, it was also proved that caissons are sufficiently safe when they are subjected to loads within the range of the design load, so it is allowable to omit examination on failure of caisson walls.

③ Actions on suspension hooks are transmitted by the adhesion between concrete and straight parts of suspension hooks and by the hook fixing effect. According to test results, the action on suspension hooks that was 3 times the action on the hook starting point was almost equivalent to the design load, and it was about 40 to 60% of the ultimate failure load. The test results also indicated that the failure load was governed by the tensile strength of concrete in parts close to the hooks, not by the bond between suspension hooks and concrete. Based on the mechanism of these eventual failures and results of measuring failure loads, it can be considered that hooks have sufficient strength even when they bear 1/3 of the overall load.

④ According to test results concerning the bond strength during lifting of a caisson, the maximum bond strength of concrete with compressive strength of 24.0 N/mm^2 was 1.2 to 1.4 N/mm^2 when the age of concrete was 11 days.

⑤ According to test results, the strength of suspension hooks was significantly affected by not only the tensile force but also the bending moment and the shearing force, and the suspension hooks reached the yield point when they were subjected to a very small action. However, suspension hooks are temporary tools and it was proved that they would not fracture under loads in the range of the design load. In view of this, it was decided that verification should be performed in terms of the tensile yield or the shear yield. In general, the design value of the shear yield strength is smaller than the design value of the tensile yield strength, so performance verification can be performed by using the shear yield strength.

⑥ To prevent cracking between embedded parts of suspension hooks used in combination with a suspension frame, it is effective to shape the suspension hooks in such a way as to reduce the horizontal force that occurs due to the structure (see **Fig. 2.2.22**) and to provide reinforcing bars between suspension hooks to reduce the crack width.

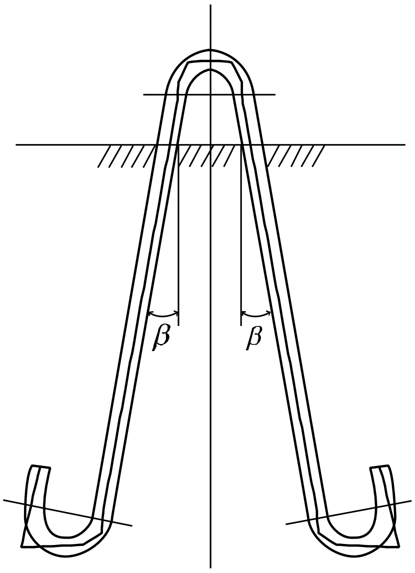


Fig. 2.2.22 Shape of Suspension Hook

⑦ For direct lifting, performance verification shall be performed against shear failure at points where suspension hooks are embedded on top of an outer wall. On the assumption that a fracture on top of the outer wall is shaped as shown in **Fig. 2.2.23**, verification of performance against shear failures on top of the outer wall can be performed by using **equation (2.2.12)**.

$$\gamma_i \frac{R_d}{V_{cd}} \leq 1.0$$

$$R_d = P_d \cos\theta \sin\phi \qquad \textbf{(2.2.12)}$$

$$V_{cd} = \frac{f_{V_{cd}} A_\tau}{10^3}$$

where

γ_i : structure factor

R_d : design value of horizontal force acting on suspension hook in direction perpendicular to outer wall (kN)

V_{cd} : design value of shear resistance (kN)

θ : angle formed by rope and top surface of caisson (º)

ϕ : angle formed by outer wall and projection on top surface of caisson (º)

$$f_{V_{cd}} = \frac{0.20\beta_d \beta_p \beta_r \sqrt{f'_{cd}}}{\gamma_b}$$

$$\beta_d = \sqrt[4]{1000/d} \leq 1.5$$

$$\beta_p = \sqrt[3]{100/p} \leq 1.5$$

$$\beta_r = 1.0$$

f'_{cd} : design value of compressive strength of concrete (N/mm^2); this value may be calculated by assuming that the material factor γ_c is 1.0.

d : distance from center of suspension hooks to horizontal reinforcing bar (mm)

$$p = \frac{A_s}{b\sqrt{2}d}$$

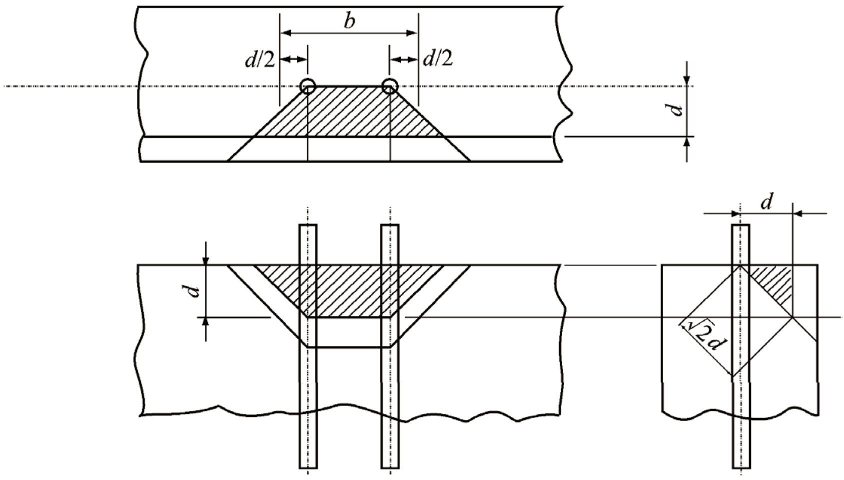
A_s : amount of horizontal reinforcing bars in shear plane (mm^2)

b : length obtained by adding d to spacing of suspension hooks (mm); spacing of suspension hooks shall not exceed $5D$ in principle.

D : diameter of suspension hook (mm)

A_τ : shear resistance area (mm^2), $A_\tau = b\sqrt{2}d$

γ_b : member factor (= 1.3)



Shaded areas in the figure show the broken part.

Fig. 2.2.23 Fracture of Outer Wall

⑧ If the design shear resistance calculated by using **equation (2.2.12)** is lower than the design horizontal force, the required shear resistance shall be secured by increasing the amount of reinforcement on top of the outer wall or by using the reinforcing method shown in **Fig. 2.2.24**. Test results indicated that, when a suspension hook was subjected to a horizontal force, a bending strain occurred in the suspension hook in the part from the top surface of the caisson wall down to a depth 3 to 4 times the diameter of the suspension hook. Therefore, two or three layers of reinforcing bars shall be arranged in this part.

According to results of a tension test of improved suspension hooks shaped like a nut (see **Fig. 2.2.25**) or a hairpin and secured not relying on bonding, cracking occurred under a load in the range from 780 to 1,200 kN and the maximum load was 2,200 to 2,800 kN when the test wall was 0.4 meter thick, the compressive strength of concrete was 24 N/mm^2, the suspension hooks were made of SV70, the diameter of suspension hooks was 80 mm and 70 mm, the concrete was 10 to 11 days old, and the embedded length of suspension hooks was 2 m. The test results also indicated that the suspension hooks were removable and reusable after testing.

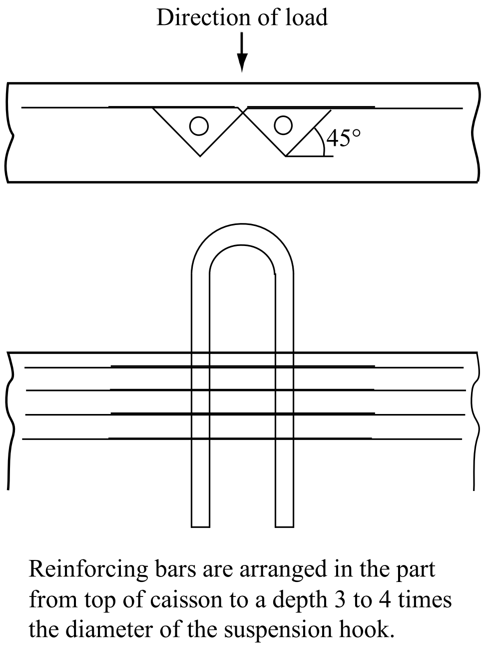


Fig. 2.2.24 Example of Reinforcing Method for Top of Outer Wall

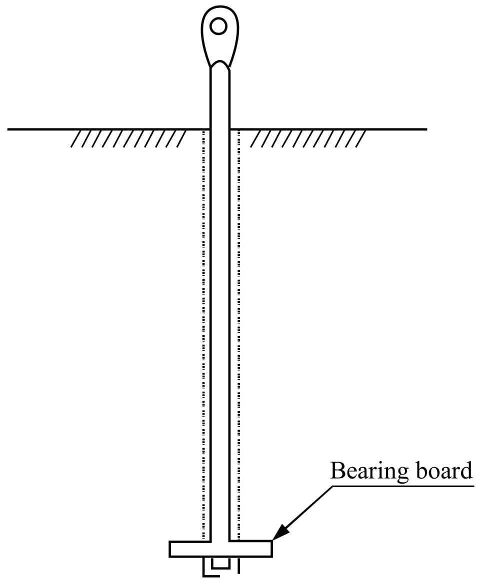


Fig. 2.2.25 Shape of Improved Suspension Hook (Nut-Shaped)

(3) Verification of Suspension Hooks Made of High Tensile Strength Deformed Steel Bars

① When suspension hooks for caissons or similar structures are made of high tensile strength deformed steel bars, performance verification of the suspension hooks can be performed in the following way.

② Performance verification of suspension hooks made of high tensile strength deformed steel bars can be performed by using **equation (2.2.13)** and using the tensile yield resistance of the suspension hooks or the bonding and fixing resistance of the embedded parts of the suspension hooks, whichever is smaller.

$$\gamma_i \frac{P_d}{\min(T_{1d}, T_{2d})} \leq 1.0$$

$$T_{1d} = \frac{\pi D^2 f_{yd}}{4 \cdot 10^3 \gamma_b} \qquad \textbf{(2.2.13)}$$

$$T_{2d} = \frac{\pi D f_{bod} \alpha l_1}{10^3 \gamma_b}$$

where

γ_i : structure factor

P_d : design value of action on one suspension hook (kN)

T_{1d} : design value of tensile yield resistance of suspension hook (kN)

T_{2d} : design value of bonding and fixing resistance of embedded part of suspension hook (kN)

D : diameter of suspension hook (mm)

f_{yd} : design value of tensile yield strength of suspension hook (N/mm^2); this value may be calculated by assuming that the material factor γ_c is 1.0.

f_{bod} : design value of bond strength of concrete (N/mm^2); this value may be calculated by assuming that the material factor γ_c is 1.0.

α : factor to be used for taking account of bond strength of deformed bars; this value may be generally set to 0.9.

l_1 : length of bonded part of suspension hook (mm)

γ_b : member factor (= 1.1)

③ According to results of a test of suspension hooks made of high tensile strength deformed steel bars, the suspension hooks satisfied performance requirements with no need to take special reinforcement measures under the condition that the concrete strength is 24 N/mm^2 or higher, the wall into which the suspension hooks were embedded was 0.4 meter thick, and the action on one suspension hook was 1,600 kN or less and on the assumption that the required bonded length of suspension hooks l_1 is 30D.

④ In principle, suspension hooks shall be longer than the length calculated by using **equation (2.2.14)** (see **Fig. 2.2.26**).

$$L = l_1 + l_2 + l_3 \qquad \textbf{(2.2.14)}$$

where

l_1 : length of bonded part of suspension hook (mm)

l_2 : length of unbonded part (6D or more)

l_3 : length of protruding part (2D to 200 mm)

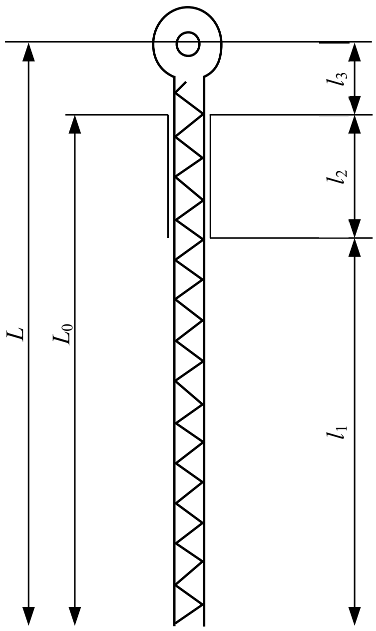


Fig. 2.2.26 Embedded Length of Suspension Hook

⑤ Performance verification of suspension hooks regarding tensile yield resistance and shear resistance at their hinges shall be performed based on **equation (2.2.15)** (see **Fig. 2.2.27**).

$$\gamma_i \frac{P_d}{T_{3d}} \leq 1.0\gamma_i$$

$$T_{3d} = \frac{f_{yd}(2R - dH)t}{10^3 \gamma_b}$$

$$\frac{P_d}{V_{1d}} \leq 1.0 \tag{2.2.15}$$

$$V_{1_d} = \frac{2 f_{vy_d} \left\{ 10 + \sqrt{R^2 - \left(\frac{dH}{2}\right)^2} \right\} t}{10^3 \gamma_b}$$

where

γ_i : structure factor

P_d : design value of action on one suspension hook (kN)

T_{3d} : design value of tensile yield resistance at hinge of suspension hook (kN)

V_{1d} : design value of shear resistance at hinge of suspension hook (kN)

R : diameter of ring (mm)

dH : diameter of ring hole (mm)

t : thickness of ring (mm)

f_{yd} : design value of tensile yield strength of suspension hook (N/mm^2); this value may be calculated by assuming that the material factor γ_c is 1.0.

f_{vyd} : design value of shear yield strength of suspension hook (N/mm^2); this value may be calculated by assuming that the material factor γ_c is 1.0.

γ_b : member factor (= 1.1)

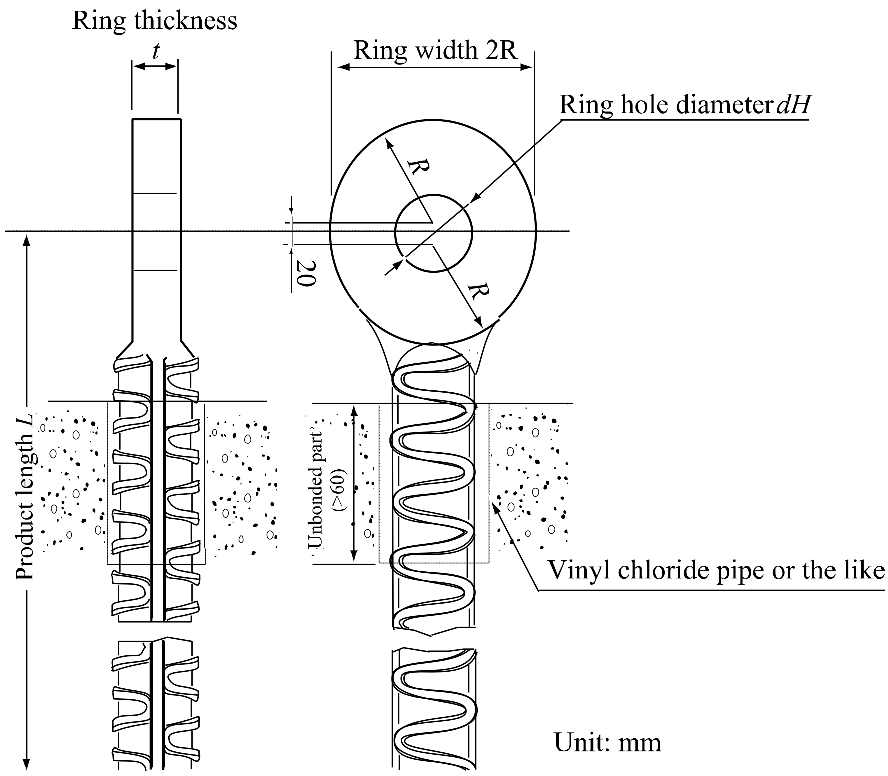


Fig. 2.2.27 Detailed Drawing of Suspension Hook

2.3 L-shaped Blocks

[Public Notice] (Performance Criteria of L-shaped blocks)

Article 24

The performance criteria of a reinforced concrete L-shaped block (hereinafter referred to as a "L-shaped block" in this Article) shall be such that the risk of impairing the integrity of the front wall, bottom slab, buttress, and footing of the L-shaped block is equal to or less than the threshold level, under the permanent state in which the dominating actions are self-weight and earth pressure, and under the variable situation in which the dominating actions are Level 1 earthquake ground motions and variable waves in consideration of the type of facility.

[Interpretation]

8. Members Composing Facilities Subject to the Technical Standards

(4) Performance Criteria of L-shaped Blocks (Article 7, paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 24 of the Public Notice)

① Performance criteria of caissons and their interpretation, except those related to flotation and installation, shall be applied correspondingly to L-shaped blocks, provided that the terms "outer wall," "partition wall" and "internal earth pressure" shall be replaced with "front wall," "buttress" and "earth pressure" respectively.

② Serviceability shall be the required performance for L-shaped blocks under the permanent situation in which the dominating action is earth pressure and under the variable situation in which the dominating actions are Level 1 earthquake ground motions and variable waves. Performance verification items for those actions and standard indices for setting limit values shall be in accordance with **Attached Table 8-10**.

Attached Table 8-10 Performance Verification Items and Standard Indices for Setting Limit Values for L-shaped Blocks

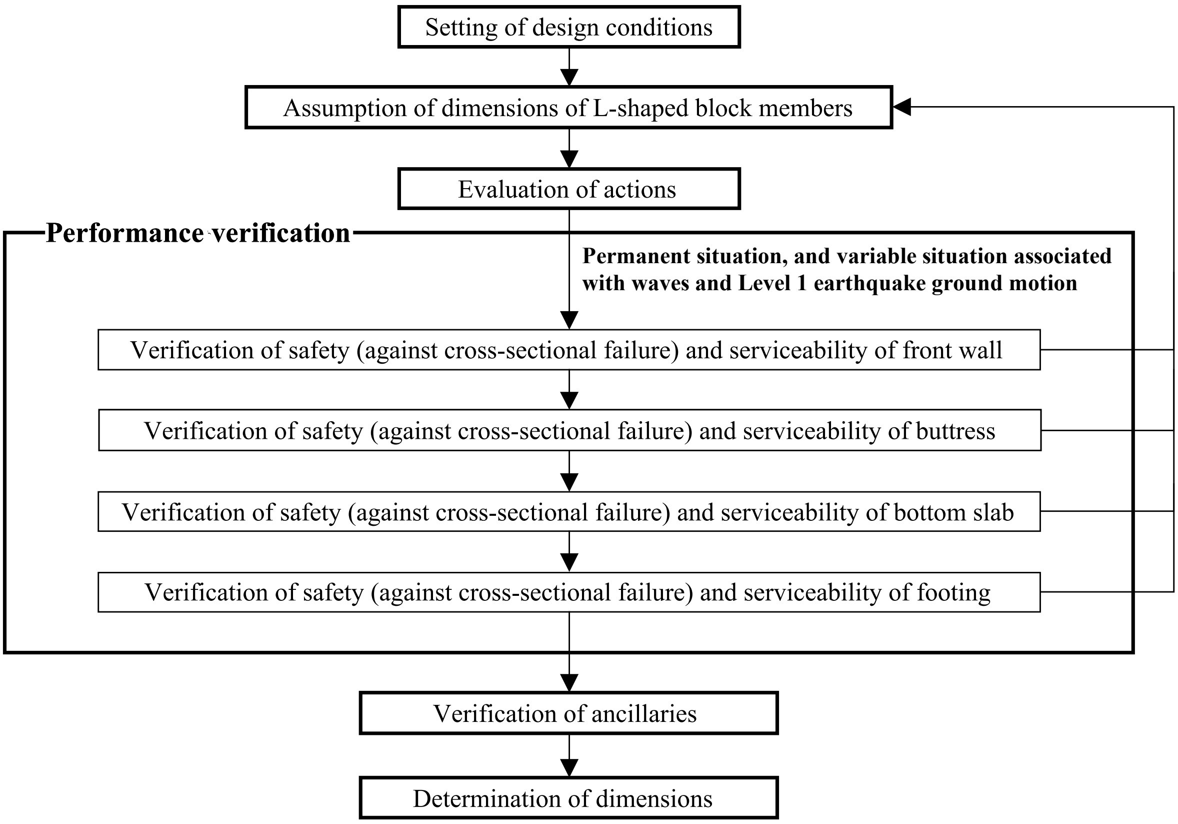
Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	24	-	-	Serviceability	Permanent	Earth pressure	Water pressure, reaction of front wall as bearing part, reaction of bottom slab as bearing part	Extrusion of front wall or bottom slab from buttress (yielding of reinforcing bars)	Design yield stress
							Variable	Level 1 earthquake ground motion [Variable waves]	Self-weight, earth pressure, water pressure, reaction of front wall as bearing part, reaction of bottom slab as bearing part	Extrusion of front wall or bottom slab from buttress (yielding of reinforcing bars)	Design yield stress

Note: The action shown in brackets is an alternative dominating action.

2.3.1 Fundamentals of Performance Verification

(1) An example of the performance verification procedure for L-shaped blocks is shown in **Fig. 2.3.1**.

(2) For performance verification of L-shaped blocks, refer to the **Technical Manual for L-shaped Block Quaywalls** [14] and **Part III**, **Chapter 2**, **2.2 Caissons**.



*1: For high earthquake-resistance facilities and the facilities to which damage might have a serious impact on human life, property, and social activity, it is preferable to verify the performance under accidental situations, as necessary.

Fig. 2.3.1 Example of Performance Verification Procedure for L-shaped Blocks

2.3.2 Determination of Basic Cross Section and Characteristic Values

(1) It is desirable that the dimensions of the members of L-shaped blocks be determined considering the following items:

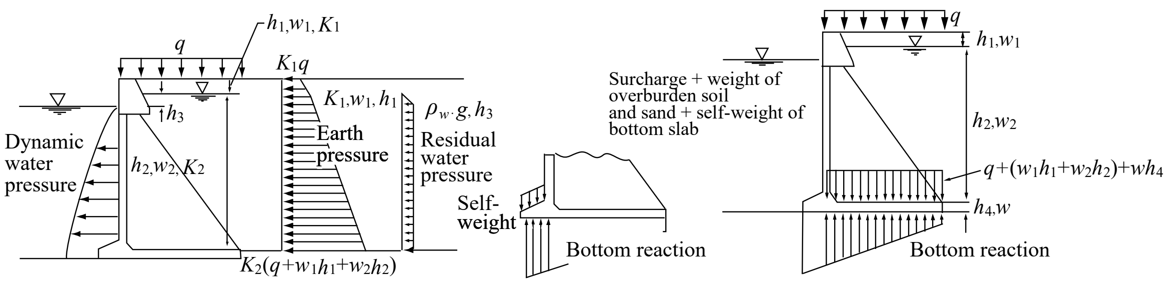
① Capacity of L-shaped block fabrication facilities
② Hoisting capacity of crane
③ Water depth at which L-shaped blocks are to be installed (mound water depth)
④ Service conditions after installation of L-shaped blocks (backfilling and superstructure construction)

(2) The wall height of L-shaped blocks should be determined so that the superstructure may be easily constructed, considering the water depth at the front face and the tidal range when the L-shaped blocks form the main body of a facility.

2.3.3 Actions

(1) For setting of actions, refer to **Part III**, **Chapter 2**, **2.2.3 Actions**.

(2) Actions on the members of L-shaped blocks can be considered as shown in **Fig. 2.3.2**.



where

q : surcharge (kN/m^2)

γ_1 : unit weight of soil above residual water level (kN/m^3)

γ_2 : unit weight of soil below residual water level (kN/m^3)

$\rho_w g$: unit weight of sea water (kN/m^3)

h_1 : thickness of layer of soil above residual water level (m)

h_2 : thickness of layer of soil below residual water level (m)

h_3 : tidal range (m)

h_4 : thickness of bottom slab (m)

K_1 : coefficient of earth pressure of soil above residual water level

K_2 : coefficient of earth pressure of soil below residual water level

w_1 : weight of soil above residual water level (kN/m^2)

w_2 : weight of soil below residual water level (kN/m^2)

w_4 : self-weight of bottom slab (kN/m^2)

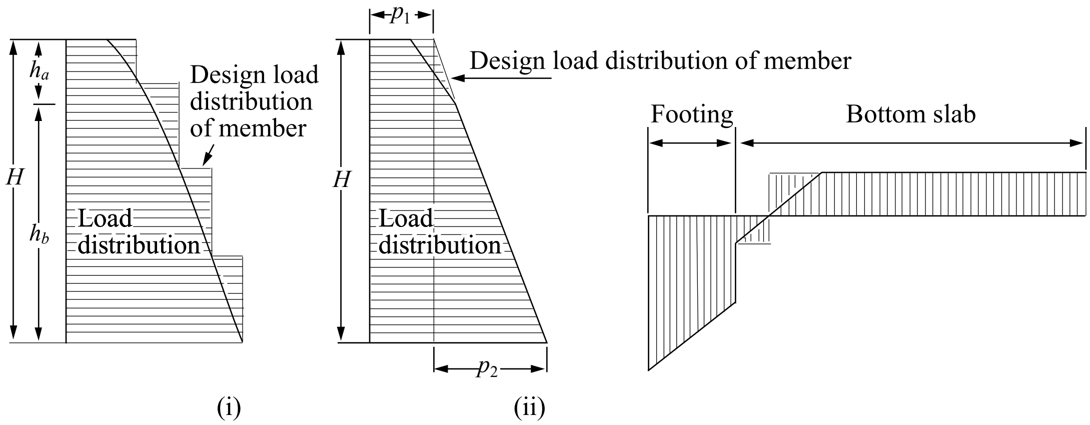
Fig. 2.3.2 Actions on L-shaped Blocks

(3) For calculating the earth pressure, refer to **Part II**, **Chapter 4**, **2 Earth Pressure** and the **Technical Manual for L-shaped Block Quaywalls** [14)].

(4) For calculating the bottom reaction, refer to **Part III**, **Chapter 2**, **2.2.3 Actions (9)**.

(5) In the fabrication process of an L-shaped block, its concrete wall may be constructed in the upright position or in the lying position. In cases where the wall is constructed in the lying position, the block needs to be raised before installation; therefore, in performance verifications, it is necessary to study the actions that occur when the block is raised.

(6) In general, the actions on L-shaped blocks are not distributed uniformly. However, these non-uniformly distributed actions may be considered to be a combination of appropriately divided, uniformly distributed loads. In this case, the combination of divided loads should not cause weak points in strengths of members. Examples of ways to divide loads are shown in **Fig. 2.3.3**.



(a) Earth pressure (b) External forces acting on footing and bottom slab

Fig. 2.3.3 Examples of Ways to Divide Loads

2.3.4 Performance Verification

(1) Front wall

① Performance verification of the front wall can generally be performed assuming it as a buttressed slab.

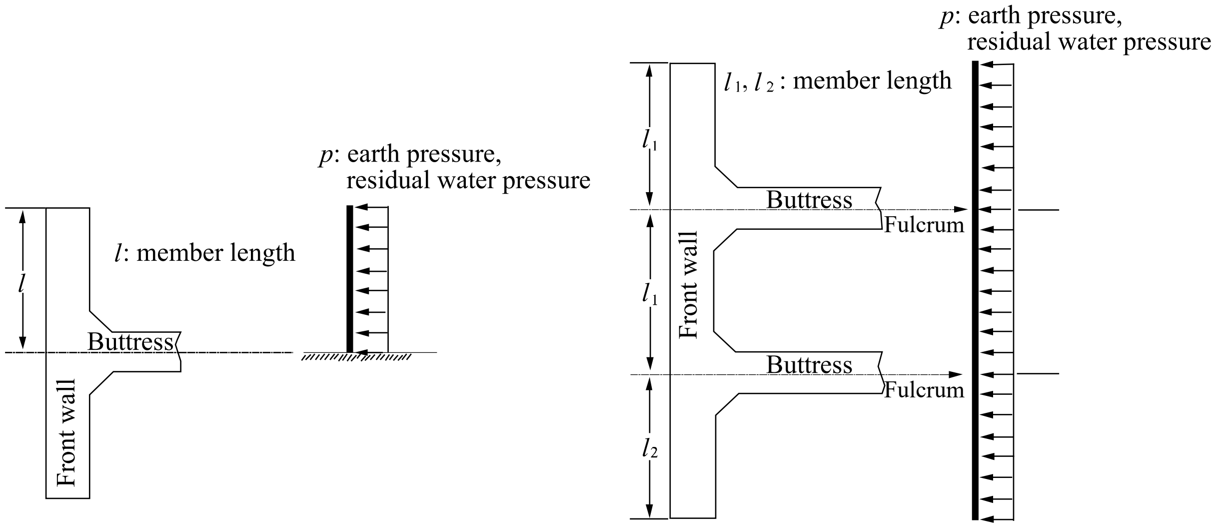
② In cases where the front wall is supported by a single buttress or by two or more buttresses, performance verification can be performed assuming the front wall as a buttressed cantilever slab or a continuous slab respectively.

③ The member length of the front wall shall be measured from the centerline of a buttress in principle.

④ Actions that work on the front wall from behind can be generally regarded as acting on the entire member length.

⑤ The member length of the front wall and the actions on it can be considered as shown in **Fig. 2.3.4**.

⑥ Structurally, the front wall is supported by the bottom slab as well as by one or more buttresses. Therefore, the front wall may be regarded as a slab which is supported on two or three sides. For an L-shaped block with a high front wall, it is generally possible to assume the front wall as a cantilever slab or continuous slab in the performance verification, giving consideration to complicatedly arranged reinforcing bars at the joint between the front wall and the bottom slab. It is also possible to assume the front wall as a slab supported on two or three sides, instead of a cantilever slab or continuous slab, if it is more reasonable to do so.



(a) When supported by one buttress (b) When supported by two buttresses

Fig. 2.3.4 Member Length of Front Wall and Actions on It

(2) Footing

① Performance verification of the footing can be performed assuming it as a cantilever slab supported by the front wall.

② The member length of the footing may be regarded as the distance between the front edge of the footing and the front face of the front wall.

③ The member length of the footing and the actions on it can be considered as shown in **Fig. 2.3.5**.

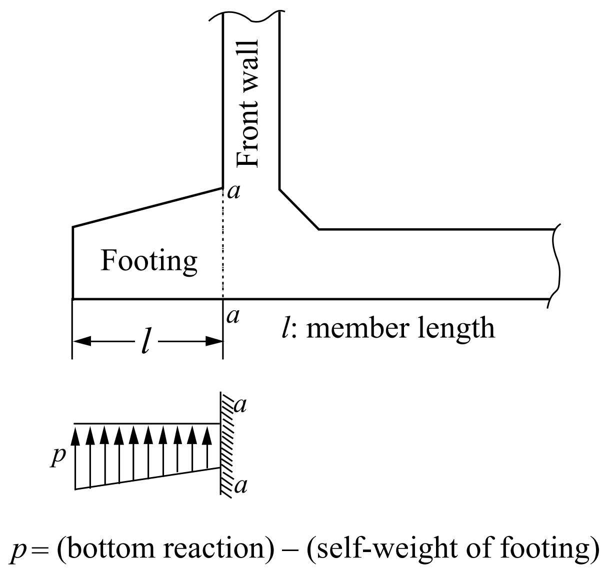


Fig. 2.3.5 Member Length of Footing and Actions on It

(3) Bottom Slab

① Performance verification of the bottom slab can generally be performed assuming it as a buttressed slab. When the bottom slab is supported by a single buttress or by two or more buttresses, it can be treated as a buttressed cantilever slab or a continuous slab respectively.

② The member length of the bottom slab may be regarded as the distance measured from the centerline of a buttress.

③ Actions from the upper surface of the bottom slab can generally be regarded as acting on the entire member length.

④ The bottom slab may be regarded as a structure supported by the front wall as well as by one or more buttresses. Therefore, performance verification of the bottom slab may be performed assuming it as a slab supported on two or three sides. However, for the same reason as stated in **(1)**, verification may generally be performed assuming that the bottom slab is a cantilever slab or a continuous slab. Accordingly, in the cases where it is advantageous in performance verification to assume the bottom slab as a slab supported on two or three sides, it is not necessary to assume the bottom slab as a cantilever slab or continuous slab as described in ①.

⑤ Among actions on the bottom slab, the bottom reaction acts on the entire member length, but the action transmitted by backfilling and coming from the upper surface of the bottom slab can be considered as acting on the net span of the bottom slab. However, considering the action from the upper surface of the bottom slab in this way requires troublesome calculations and does not have a large effect on performance verification. Therefore, the action transmitted by backfilling and coming from the upper surface of the bottom slab may generally be considered as acting on the entire member length.

⑥ In performance verification of the bottom slab, it is necessary to set the load factor considering the load under which the member is at the greatest risk.

(4) Buttresses

① Performance verification of a buttress can be performed assuming it as a T-beam integrated with the front wall.

② A buttress may be examined assuming it as a cantilever beam supported by the bottom slab against the reaction from the front wall.

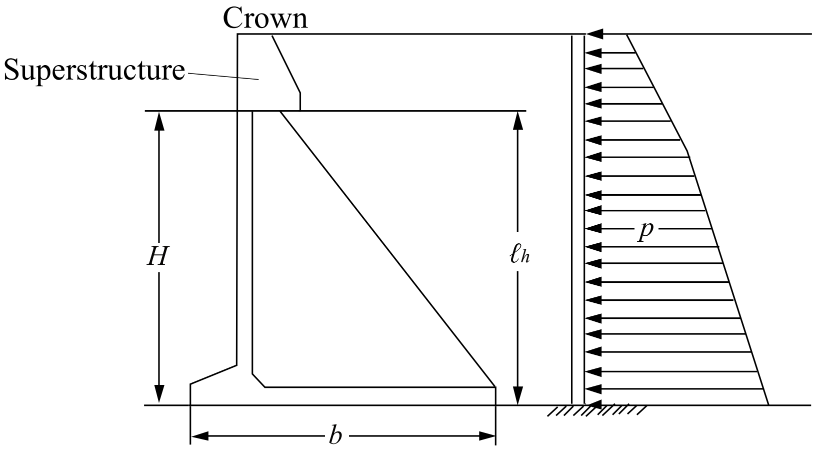
③ Performance verification of a buttress shall be performed for the cross sections parallel to the bottom slab.

④ The buttress(es), the front wall, and the bottom slab shall be ligdly connected. The amount of reinforcing bars required for this purpose shall be calculated independently from that of stirrups against shearing forces.

⑤ When performance verification of the front wall and the bottom slab is performed as described here, actions from behind the buttress(es) may be disregarded.

⑥ The member length of a buttress can be considered to be the total height including the bottom slab, as shown in **Fig. 2.3.6**. However, it is necessary to consider actions that work on the superstructure as well as the buttress.

⑦ When the cross section of a buttress is calculated assuming it as a T-beam, attention shall be paid to the position of the neutral axis which is located either in the front wall or in the buttress.



where

p	: sum of earth pressure and residual water pressure (kN/m^2)
l_h	: member length of buttress (m)
b	: width of block (m)
H	: height of block (m)

Fig. 2.3.6 Member Length of Buttress and Actions on It

2.3.5 Verification of Lifting Points

(1) For performance verification of lifting points, refer to **Part III**, **Chapter 2**, **2.2.5 Verification of Suspension Hooks during Lifting**.

(2) **Part III**, **Chapter 2**, **2.2.5 Verification of Suspension Hooks during Lifting** gives the imbalance coefficient of 1.8, and this is the value set for multipoint lifting. When the number of lifting points is small, the imbalance coefficient may be set to a value less than 1.8. It is common to set the imbalance coefficient to 1.8 for lifting at five points or more, 1.33 for lifting at four points or more, and about 1.2 for lifting at three or two points.

(3) In cases where an L-shaped block is lifted by a floating crane or is expected to be affected by waves during lifting, it is necessary to consider the impact load acting on the L-shaped block. The impact load acting on an L-shaped block lifted by a floating crane can be considered as about 20% of the weight of the block. It may be considered that the bottom adhesion and the impact load will not act on the block simultaneously.

2.4 Cellular Blocks

[Public Notice] (Performance Criteria for Cellular Blocks)

Article 25

The provisions of Article 23 apply mutatis mutandis to the performance criteria of cellular blocks made of reinforced concrete.

[Interpretation]

8. Members Composed of Facilities Subject to the Technical Standards

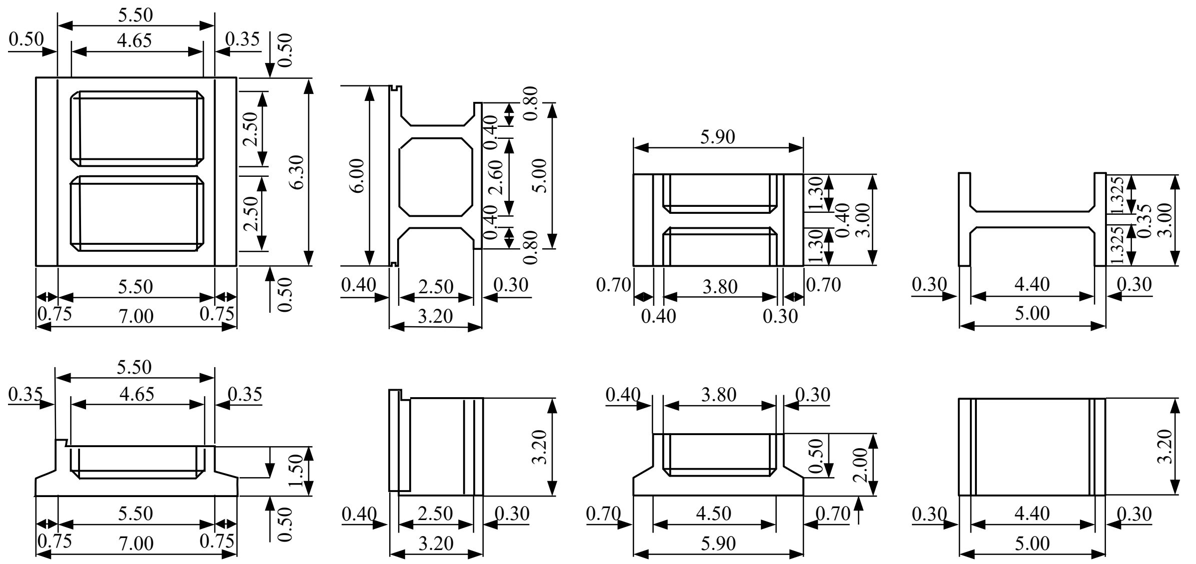
(5) Performance Criteria of Cellular Blocks (Article 7, paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 25 of the Public Notice)

The performance criteria of caissons and their interpretation shall be applied correspondingly to cellular blocks made of reinforced concrete.

2.4.1 Fundamentals of Performance Verification

(1) "Cellular blocks" generally refer to blocks that are composed of outer walls without a bottom slab. A single block or stacked blocks function as a wall body. As a special type, cellular blocks with a bottom slab are also used. It is necessary to adopt an appropriate performance verification method on the basis of an adequate understanding of the characteristics of the block shape.

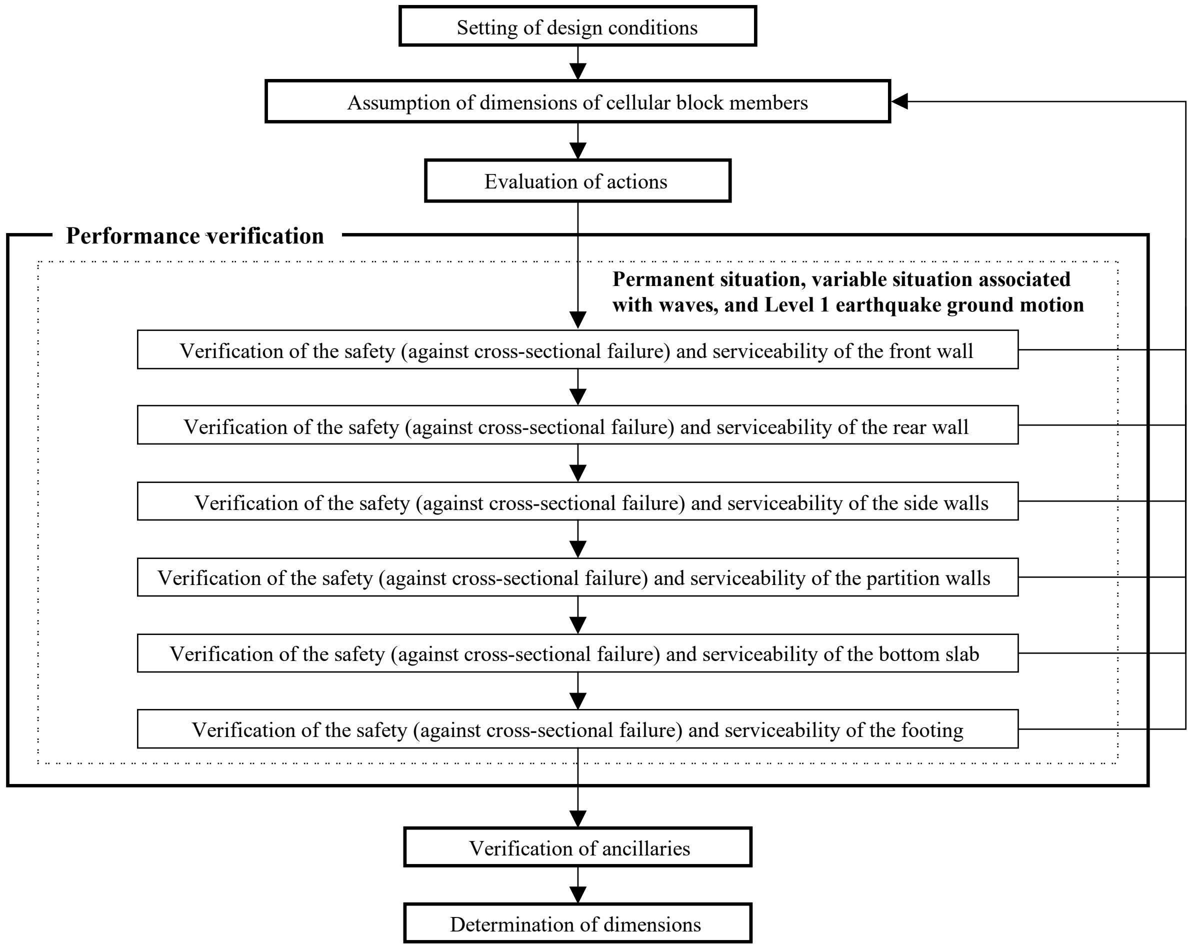
(2) Cellular blocks have various cross-sectional shapes. **Fig. 2.4.1** shows examples of cellular blocks.



(Unit: m)

Fig. 2.4.1 Examples of Cellular Blocks

(3) **Fig. 2.4.2** shows an example of the performance verification procedure for cellular blocks.



*1: For outer walls that are not exposed to waves, verification may be limited to serviceability.

*2: For high earthquake-resistance facilities or facilities to which damage might have a serious effect on human life, property, and social activity, it is preferable to verify the performance under accidental situations. The verification of accidental situations associated with waves shall be performed in cases wherein damage to those facilities might have a serious effect on hazardous material handling facilities located just behind them.

Fig. 2.4.2 Example of Performance Verification Procedure for Cellular Blocks

(4) For the performance verification of cellular blocks, refer to **Part III**, **Chapter 2**, **1 Verification of Members**.

(5) For the various types of cellular blocks, refer to **Part III**, **Chapter 2**, **2.2 Caissons** and **Part III**, **Chapter 2**, **2.3 L-Shaped Blocks** depending on structural type.

When cellular blocks will be used as members of breakwaters, revetments, or other structures subject to the action of the wave force, safety (against fatigue failure) should be studied separately.

2.4.2 Setting of the Basic Cross Section

(1) The dimensions of members of cellular blocks shall be set by considering the following items:

① Capability of cellular block fabrication facilities
② Hoisting capacity of the crane
③ Water depth at the location where cellular blocks will be installed
④ Work conditions after the installation of cellular blocks (backfilling and superstructure construction)
⑤ Formation of a mutually integrated block structure when blocks are stacked in tiers

2.4.3 Actions

(1) The rear wall is subject to the backfill earth pressure, the residual water pressure, and other external forces. However, the examination of these external forces may generally be omitted because they are canceled out by the internal earth pressure.

(2) The internal earth pressure and residual water pressure acting on cellular blocks can generally be considered as shown in **Fig. 2.4.3**. In cases wherein the wall body composed of cellular blocks is backfilled, the stresses on the side walls and rear walls due to the filling in the cellular blocks are reduced by the active earth pressure, the residual water pressure, and other forces after backfilling is completed. However, in view of the fact there are many cases wherein filling precedes backfilling in the process of construction, performance verification should be performed for members in this state.

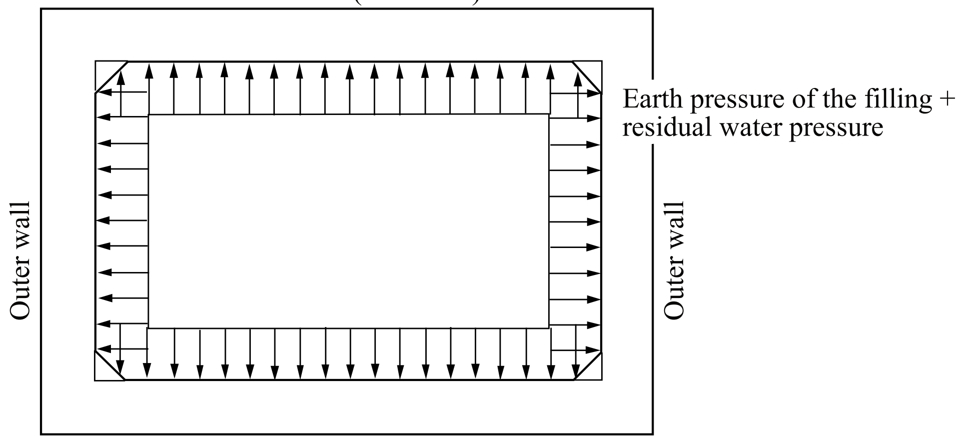


Fig. 2.4.3 Actions on the Cellular Block]

(3) Actions on the front wall, rear wall, and side walls

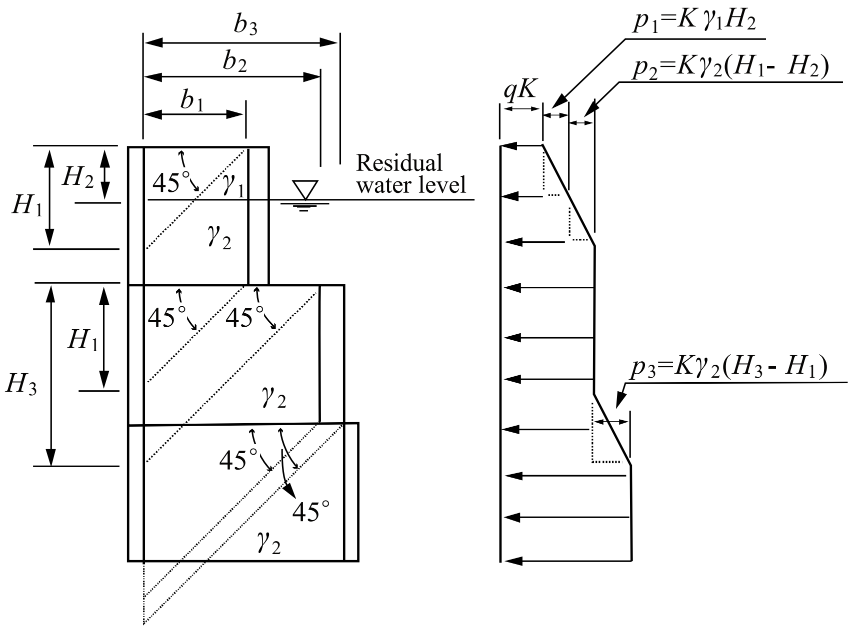
① As actions on the front wall, rear wall, and side walls of a cellular block, the internal earth pressure and residual water pressure shall be taken into account. However, in cases wherein cast-in-place concrete is placed on top of the cellular block to a degree such that the surcharge may not affect the interior of the cellular block, it is generally not necessary to consider the surcharge imposed on the cast-in-place concrete.

② Internal earth pressure

(a) The coefficient of earth pressure for the internal earth pressure may be set as 0.6. However, it is not necessary to consider the internal earth pressure when the filling consists of blocks or concrete.

(b) It may be considered that the internal earth pressure increases as the depth from the crown of the wall increases but does not increase any more after the depth becomes larger than the inner width b_1 of the wall.

(c) The earth pressure acting on cellular blocks stacked in tiers may be considered as shown in **Fig. 2.4.4**. However, when the inner width of the lower cellular blocks is smaller than that of the upper blocks (in the case of cellular blocks partitioned by partition walls), the earth pressure obtained for the upper blocks may be extended to the lower blocks without increasing its value.



where

q : surcharge (kN/m^2);

γ_1 : unit weight of the filling material above the residual water level (kN/m^3);

γ_2 : unit weight of the filling material below the residual water level (kN/m^3);

K : coefficient of internal earth pressure $K = 0.6$;

b_1 : inner width of the wall (m) ($b_1 = H_1$).

Fig. 2.4.4 Method of Calculating the Internal Earth Pressure

(d) For the internal earth pressure in cellular blocks, refer to **Part III**, **Chapter 2**, **2.2 Caissons.**

③ **Residual water pressure**

(a) For breakwaters

The residual water pressure (internal water pressure in a cellular block) is generally obtained by calculating the hydraulic head difference between the water level inside the block and LWL. However, when the wave trough acts on the front of a breakwater, the resultant increase in the internal water pressure shall be considered on the basis of the circumstances.

(b) For quaywalls

The residual water pressure is generally obtained by calculating the hydraulic head difference between the residual water level and LWL.

④ When the wave trough acts on the front of a cellular block used for a breakwater, revetment, or similar facility, the resultant increase in the residual water level difference needs to be examined. For the action that occurs in this state, refer to **Part II**, **Chapter 2, 6.2 Wave Force on Upright Walls**.

(4) Actions on partition walls

Partition walls shall be designed to ensure that outer walls will not fall forward, i.e., outer walls will not be extruded from partition walls, due to the internal earth pressure and residual water pressure. As an action that might cause the extrusion of outer walls from partition walls, the earth pressure acting on the shaded areas in **Fig. 2.4.5.** shall be taken into consideration.

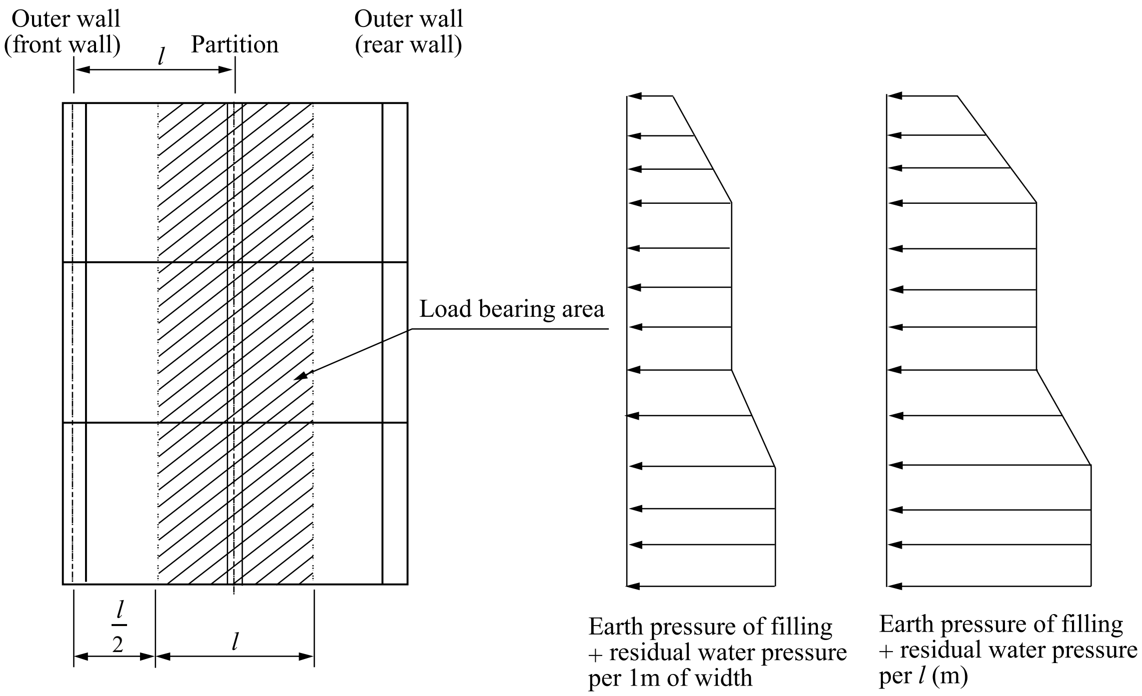


Fig. 2.4.5 Determination of Actions That Affect the Extrusion of the Outer Walls from the Partition Wall

(5) Wave forces are generally not considered. However, in cases wherein a particularly strong impact wave pressure acts on the wall, it is necessary to consider this action.

(6) For actions during construction, refer to **Part III**, **Chapter 2**, **2.3 L-Shaped Blocks**.

(7) For the combinations of the general actions to be considered in the performance verifications and the load factors to be used for multiplying the characteristic values of the respective actions, refer to the combinations of actions and the load factors shown in **Part III**, **Chapter 2**, **2.3.3 Actions**.

(8) In the cases wherein the actions on the members of cellular blocks are divided for calculation convenience, refer to **Part III**, **Chapter 2**, **2.3.3 Actions**.

2.4.4 Performance Verification

(1) Rectangular Cellular Blocks

① **Outer walls**

(a) Actions on a rectangular cellular block may be divided into stages and calculated for the unit width of the wall surface by assuming that the cellular block is a rigid frame. The methods for analyzing rigid frames include the slope-deflection method and the moment distribution method.

(b) The span to be considered in calculations shall be a center-to-center distance in principle. According to the results of a model test, the fixed points of members of a caisson outer wall are located inside the haunch and within the inner width of the outer wall.[11)]

(c) For a rigid frame shown in **Fig. 2.4.6**, the moment about the axis can be calculated by using **equation (2.4.1)**.

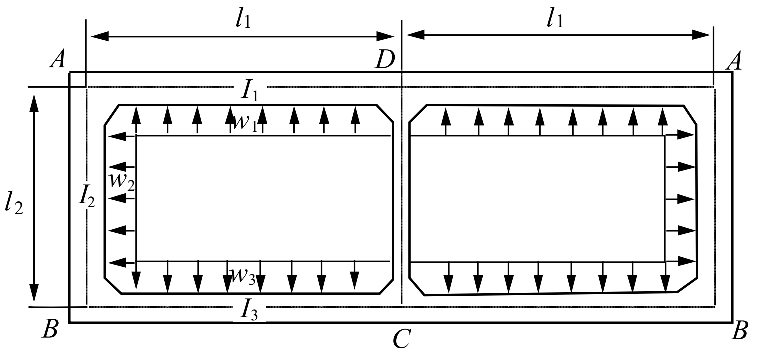
$$\left.\begin{aligned}
M_{CB} &= 2EK_3\theta_B + \frac{w_3 l_1^2}{12}\\
M_{BC} &= 2EK_3 2\theta_B - \frac{w_3 l_1^2}{12}\\
M_{BA} &= 2EK_2\left(2\theta_B + \theta_A\right) + \frac{w_2 l_2^2}{12}\\
M_{AB} &= 2EK_2\left(\theta_B + 2\theta_A\right) - \frac{w_2 l_2^2}{12}\\
M_{AD} &= 2EK_1 2\theta_A + \frac{w_1 l_1^2}{12}\\
M_{DA} &= 2EK_1\theta_A - \frac{w_1 l_1^2}{12}\\
K_1 &= \frac{I_1}{l_1}\\
K_2 &= \frac{I_2}{l_2}\\
K_3 &= \frac{I_3}{l_1}
\end{aligned}\right\} \tag{2.4.1}$$

where θ_A and θ_B shall be calculated by using **equation (2.4.2)**.

$$\left.\begin{aligned}
M_{BC} + M_{BA} &= 0\\
M_{AB} + M_{AD} &= 0
\end{aligned}\right\} \tag{2.4.2}$$

The end shearing force shall be calculated by using **equation (2.4.3)**.

$$\left.\begin{aligned}
S_{AD} &= -\frac{w_1 l_1}{2} - \frac{M_{AD} + M_{DA}}{l_1}\\
S_{AB} &= \frac{w_2 l_2}{2} - \frac{M_{AB} + M_{BA}}{l_2}\\
S_{BA} &= -\frac{w_2 l_2}{2} - \frac{M_{BA} + M_{AB}}{l_2}\\
S_{BC} &= \frac{w_3 l_1}{2} - \frac{M_{BC} + M_{CB}}{l_1}\\
S_{CB} &= -\frac{w_3 l_1}{2} - \frac{M_{CB} + M_{BC}}{l_1}\\
S_{DA} &= \frac{w_1 l_1}{2} - \frac{M_{DA} + M_{AD}}{l_1}
\end{aligned}\right\} \tag{2.4.3}$$



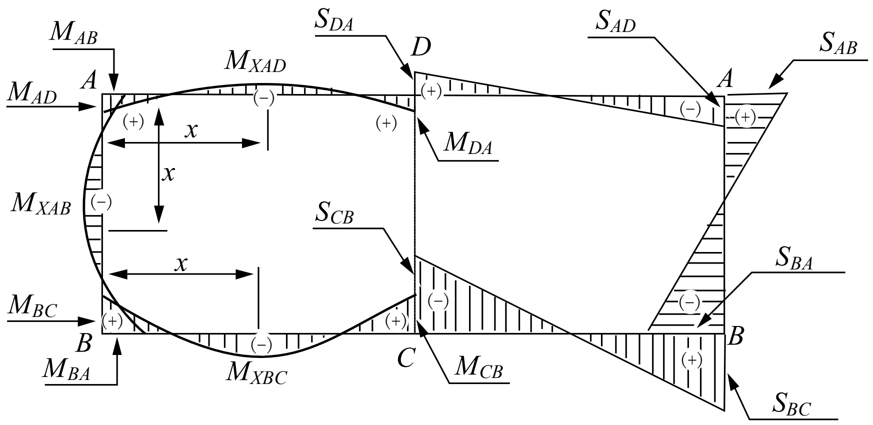


Fig. 2.4.6 Actions and Stresses on Rigid Frame

Bending moments at the given points of members shall be calculated by using **equation (2.4.4)**.

$$\left.\begin{aligned}
&\text{Bending moment of member BC}\\
&\qquad M_{x_{BC}} = M_{BC} + S_{BC}x + \frac{w_3}{2}x^2\\
&\text{Bending moment of member AB}\\
&\qquad M_{x_{AB}} = M_{AB} + S_{AB}x + \frac{w_2}{2}x^2\\
&\text{Bending moment of member AD}\\
&\qquad M_{x_{AD}} = M_{AD} + S_{AD}x + \frac{w_1}{2}x^2
\end{aligned}\right\} \qquad \mathbf{(2.4.4)}$$

Symbols in **Fig. 2.4.6** and **equations (2.4.1)** to **(2.4.4)** stand for the followings:

M_i : end moment (kN・m);

S_i : end shearing force (kN);

M_{xi} : bending moment at point x between supporting points (kN・m);

E : modulus of elasticity (kN/m^2);

K_i : relative stiffness (m^3);

I_i : geometrical moment of inertia (m^4);

l_i : span (m);

w_i : load intensity (kN/m);

θ_I : end deflection angle (rad).

② **Partition walls**

(a) The member forces acting on partition walls can be calculated in the same way as described in ① Outer walls.

(b) In cases wherein earth pressure might be generated owing to a difference in the filling height between neighboring chambers in the process of construction, partition walls shall be designed to be strong against the earth pressure. The member length and actions can be determined as shown in **Fig. 2.4.7**. Bending moments in a partition wall can be calculated by assuming the wall as a beam with both ends fixed (refer to **equation (2.4.5)**).

$$\left.\begin{aligned} M_C &= -\frac{wl^2}{12} \\ M_B &= \frac{wl^2}{24} \end{aligned}\right\} \qquad \textbf{(2.4.5)}$$

where

M_C : moment at end (kN·m);

M_B : moment at center (kN·m);

w : load intensity (kN/m);

l : span (m).

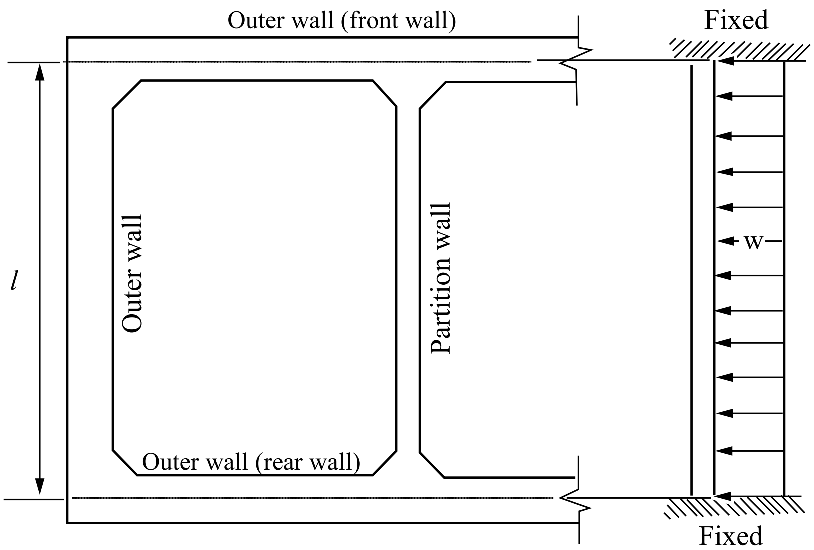


Fig. 2.4.7 Determination of the Member Length of the Partition Wall and the Actions on It

(c) The span to be considered in calculations shall be the center-to-center distance in principle.

③ **Footings**

(a) In performance verification, footings may be assumed as cantilever slabs supported by the outer walls.

(b) The member length of a footing may be considered the distance from the front of the outer wall to the tip of the footing.

(2) Other Types of Cellular Blocks

① **Front wall**

(a) The performance verification of the front wall may be conducted by assuming it as a slab supported by side walls in principle. For the front wall protruding from the right and left sides of the frame, unbalanced moments at supporting points shall be assumed to be conveyed to the side walls.

(b) The member length of the front wall shall be the distance between the centerlines of side walls in principle.

(c) Actions that work on the front wall from behind can be determined as shown in **Fig. 2.4.8**. Actions in the vertical direction can be calculated as uniformly distributed loads.

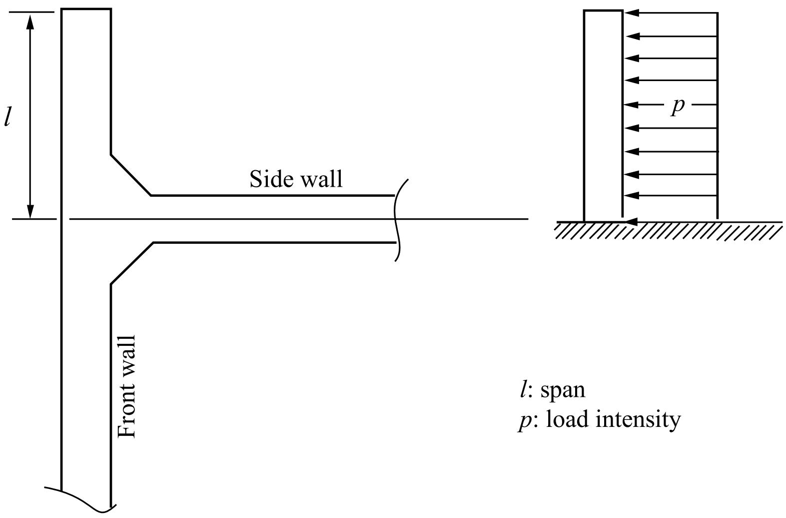


Fig. 2.4.8 Member Length of the Front Wall and the Actions on It on a Cross Section with a Projected Edge

② **Rear wall**

(a) When a cellular block is used for a mooring facility or a revetment, the rear wall differs from the front wall: the front surface of the front wall is free, whereas the surface of the rear wall is subjected to the earth pressure generated by the soil behind it. However, it is common that filling precedes backfilling in the process of construction, thus making the front wall and rear wall subjected to the same condition. In view of this, the performance verification of the rear wall may be conducted in the same way as that of the front wall.

(b) The earth pressure generated by filling shall be considered the action on the rear wall. It is generally unnecessary to consider the active earth pressure behind the rear wall.

③ **Side walls**

(a) The performance verification of side walls shall be conducted in terms of performance against reactions from the front wall and rear wall and against the moments transmitted from them. The member length of a side wall and the actions on it may be considered as shown in **Fig. 2.4.9**.

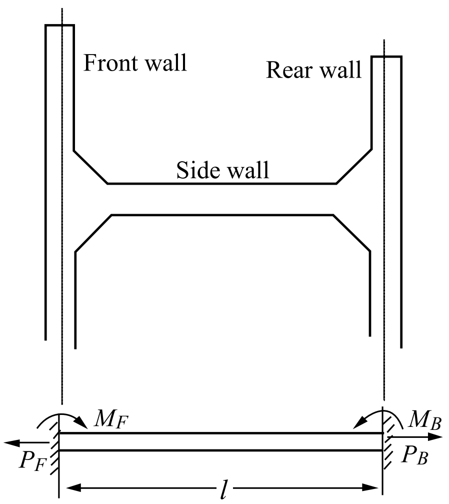


Fig. 2.4.9 Member Length of the Side Wall and the Actions on It on a Cross Section with Projected Edges

Symbols in **Fig. 2.4.9** stand for the following:

P_F : reaction from front wall (kN);

M_F : moment transmitted from front wall (kN・m);

P_B : reaction from the rear wall (kN);

M_B : moment transmitted from rear wall (kN・m);

l : span (m).

(b) The member length of a side wall may be considered the distance between the centerlines of the front and rear walls.

(c) In cases wherein earth pressure might be generated owing to a difference in the filling height between neighboring chambers in the process of construction, side walls shall be designed to be strong against the earth pressure.

④ **Bottom slab**

In cases wherein the bottom slab is provided at the bottom of a cellular block, performance verification shall be conducted in terms of the surcharge acting on the upper surface of the bottom slab, the self-weight of the bottom slab, and the bottom reaction acting on the lower surface of the bottom slab.

2.4.5 Verification of Lifting Points

(1) For the performance verification of lifting points, refer to **Part III**, **Chapter 2**, **2.2.5 Verification of Suspension Hooks during Lifting** or **Part III**, **Chapter 2**, **2.3.5 Verification of Lifting Points**.

(2) The lifting points of a cellular block should be carefully arranged out of the center of a member.

(3) When stacking cellular blocks in tiers, it is advisable to take measures to prevent suspension hooks from interfering with the stacking of blocks, e.g., by recessing the surface in which a suspension hook is embedded. **Fig. 2.4.10** shows an example of recessing the surface.

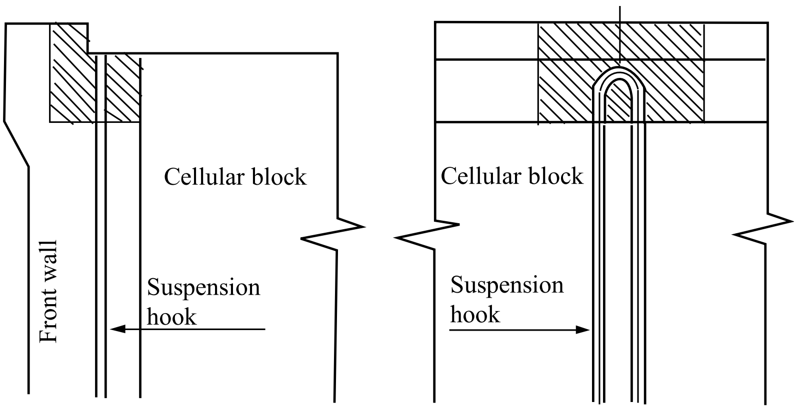


Fig. 2.4.10 Example of Recessing the Surface in which a Suspension Hook is Embedded

2.5 Upright Wave-Absorbing Caissons

[Public Notice] (Performance Criteria of Upright Wave-Absorbing Caissons)

Article 26

1 The provisions of Article 23 apply mutatis mutandis to the performance criteria of an upright wave-absorbing caisson made of reinforced concrete (hereinafter referred as an "upright wave-absorbing caisson" in this Article) with modifications as necessary.

2 In addition to the provisions of the preceding paragraph, the performance criteria of an upright wave-absorbing caisson shall be as subscribed respectively in the following items in consideration of the type of facility:

(1) The risk of impairing the integrity of the members of the wave-absorbing part of an upright wave-absorbing caisson shall be equal to or less than the threshold level, under the variable situation in which the dominating action is variable waves.

(2) The degree of damage under the accidental situation in which the dominating action is the impact of drifting objects shall be equal to or less than the threshold level.

[Interpretation]

8. Members Composed of Facilities Subject to the Technical Standards

(6) Performance Criteria of Upright Wave-Absorbing Caissons (Article 7, paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 26 of the Public Notice)

The performance criteria of caissons and their interpretation shall be applied correspondingly to upright wave-absorbing caissons. Furthermore, the following provisions shall be applied to the wave-absorbing parts of upright wave-absorbing caissons.

① **Under a variable situation in which the dominating action is variable waves**

Serviceability shall be the required performance for members of the wave-absorbing part of an upright wave-absorbing caisson under the variable situation in which the dominating action is variable waves. The performance verification items and standard indices for setting the limit values for each member of the wave-absorbing part shall be as follows:

a) Front wall slits

The performance verification items and standard indices for setting the limit values for front wall slits, which are members of the wave-absorbing part, in terms of performance against actions on them shall be in accordance with **Attached Table 8-11**.

Attached Table 8 -11 Performance Verification Items and Standard Indices for Setting the Limit Values for the Front Wall Slits of the Wave-Absorbing Part of an Upright Wave-Absorbing Caisson (under variable situation)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting the limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	−	26	2	1	Serviceability	Variable	Variable waves*1)	Water pressure, axial force transmitted from the top of the front wall	Cross-sectional failure of the front wall slit	Design ultimate capacity
								Variable waves*2)		Width of a crack in the cross section of the front wall slit	Crack widthcaused by bending
								Repeated action of waves*3)		Fatigue failure of the front wall slit	Design fatigue strength

*1) The waves indicated here shall be the waves that were defined in accordance with Item (1), Paragraph 1, Article 8 of the Public Notice and were considered in the performance verification of the structural stability of the facility of interest.
*2): In principle, the waves indicated here shall be the waves that were defined in accordance with Item (2), Paragraph 1, Article 8 of the Public Notice as the standard waves on the assumption that waves higher than the standard waves will strike the facility approximately 10,000 times during its design service life.
*3): The waves here shall be the waves that were defined in accordance with Item (2), Paragraph 1, Article 8 of the Public Notice as the waves with heights and periods that were set appropriately depending on the frequency of occurrence during the design service life.

(b) Partition wall slits and side wall slits

The performance verification items and standard indices for setting the limit values that are shown in **Attached Table 8-11** shall be applied to the partition wall slits and side wall slits, which are members of the wave-absorbing part, provided that the non-dominating actions shown in the table shall be replaced with the water pressure alone. Furthermore, the term “front wall slits” shall be replaced with “partition wall slits and side wall slits.”

(c) Upper beams

The performance verification items and standard indices for setting the limit values that are shown in **Attached Table 8-11** shall be applied to upper beams, which are members of the wave-absorbing part, provided that the non-dominating actions shown in the table shall be replaced with the water pressure, the support reaction transmitted from the slit part, the wave force acting on the ceiling slab, the self-weight of the ceiling slab, and the self-weight of the upper beams. Furthermore, the term “front wall slits” shall be replaced with “upper beams.”

(d) Lower beams

The performance verification items and standard indices for setting the limit values that are shown in **Attached Table 8-11** shall be applied to lower beams, which are members of the wave-absorbing part, provided that the non-dominating actions shown in the table shall be replaced with the water pressure and the support reaction transmitted from the slit part and lower slabs. Furthermore, the term “front wall slits” shall be replaced with “lower beams.”

② **Under an accidental situation in which the dominating action is the impact of drifting objects**

Serviceability shall be the required performance for the wave-absorbing part of an upright wave-absorbing caisson under the accidental situation in which the dominating action is the impact of drifting objects. The performance verification items for the action and standard indices for setting the limit values shall be in accordance with **Attached Table 8-12**.

Attached Table 8-12 Performance Verification Items and Standard Indices for Setting the Limit Values for Front Wall Slits in the Wave-Absorbing Part of an Upright Wave-Absorbing Caisson

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting the limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	26	2	2	Serviceability	Accidental	Impact of drifting objects, such as driftwood carried by water	Self-weight, water pressure	Cross-sectional failure of the members of the wave-absorbing part	Design ultimate capacity

2.5.1 Fundamentals of Performance Verification

(1) Upright wave-absorbing caissons are caissons with a slit wall at the front and have one or more internal water chambers, which serve to absorb waves; this type of caisson is used in quaywalls, breakwaters, and similar facilities. Upright wave-absorbing caissons can be broadly classified as permeable or impermeable. Regarding the slit shape, the vertical slit type is most commonly used, and the horizontal slit type and perforated-wall type have

also been used in actual facilities. In the performance verification of the members of upright wave-absorbing caissons, it is preferable to adequately study the characteristics of the respective structures and to perform hydraulic model experiments that are suited to the conditions.

(2) For the performance verification of upright wave-absorbing caissons, refer to **Part III**, **Chapter 2**, **2.2 Caissons**.

(3) **Fig. 2.5.1** shows the names of the members of a relatively common vertical slit-wall caisson.

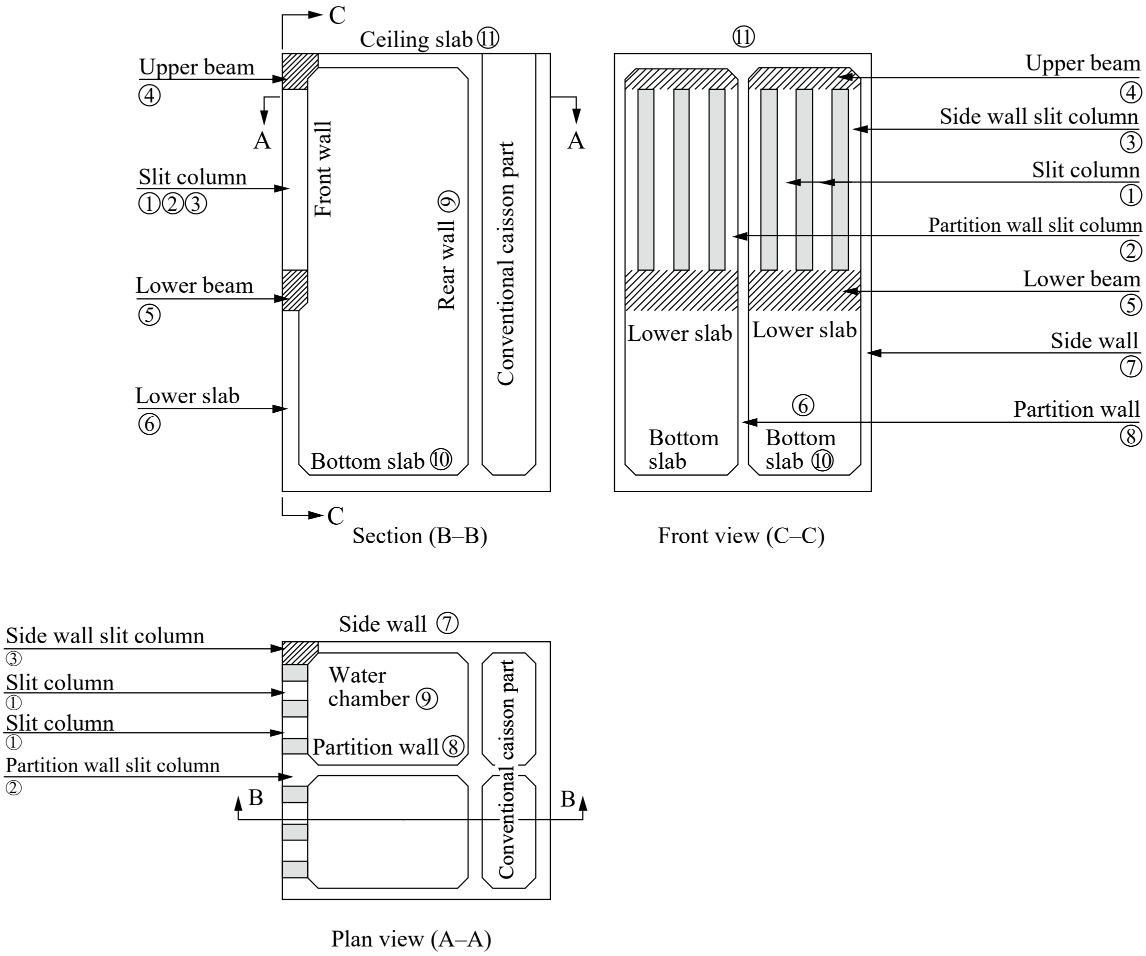


Fig. 2.5.1 Names of the Members of a Vertical Slit-Wall Caisson

(4) The vertical slit width (opening width) is generally set to approximately 0.4 to 0.5 m. The slit width is determined from the opening rate that minimizes the reflectance. However, the opening rate might decrease owing to shells, algae, and other marine organisms adhering to caissons; therefore, it is advisable to perform a thorough research of existing facilities in the vicinity in advance and to determine the appropriate slit width on the basis of the thicknesses of adhering organisms.

(5) In general, a slit caisson has an asymmetric cross section; therefore, the center of gravity is not located at the center. It is necessary to consider putting a ballast into the caisson before lifting it with a crane or floating it on water as a means to keep the caisson in balance. When providing the slit part with a joint board to allow the caisson to float, it is necessary to consider doing appropriate water sealing work that maximizes the water sealing effect.

2.5.2 Actions

(1) For the actions that are considered in the performance verification of upright wave-absorbing caissons, refer to **Part III**, **Chapter 2**, **2.2 Caissons**.

(2) Wave forces acting on the members of upright wave-absorbing caissons vary significantly depending on the structure of the water chamber and whether it has a ceiling slab. Therefore, it is advisable to examine wave forces

via performance verification by referring to the examples of upright wave-absorbing caissons installed in the past and by conducting appropriate model experiments depending on the conditions in each case.

(3) For the wave forces acting on members, refer to **Part II**, **Chapter 2**, **6.2.7 Wave Forces Acting on Upright Wave-Absorbing Caisson** and **Reference 15)**.

(4) **Fig. 2.5.2** and **equation (2.5.1)** show an example of the determination of wave forces acting on the members of an upright wave-absorbing caisson based on model experiments [16), 17)].

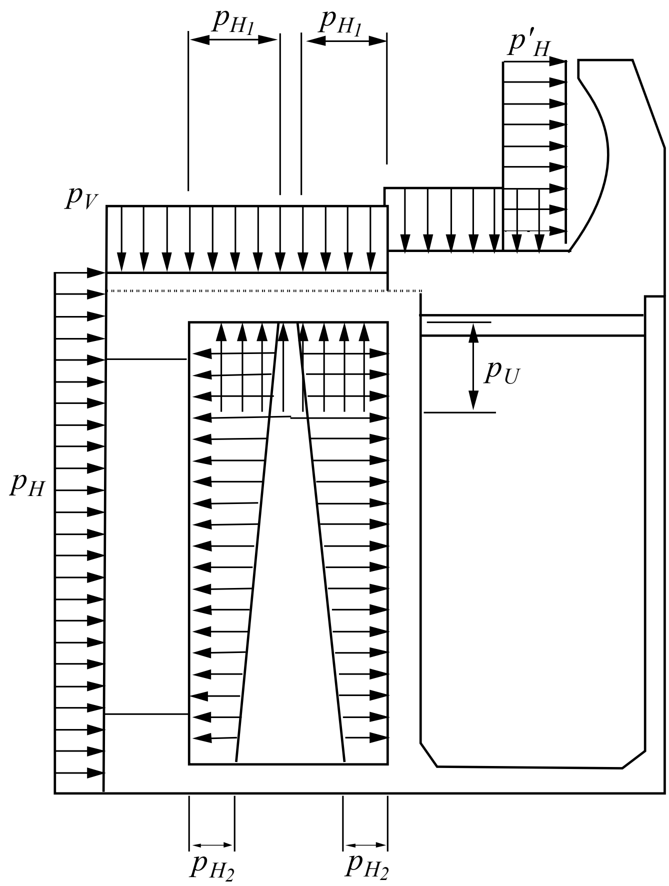


Fig. 2.5.2 Example of the Distributions of Wave Forces Acting on Members

$$\left.\begin{aligned} p_H &= 1.0 w_0 H_{max} \\ p'_H &= 1.5 w_0 H_{max} \\ p_V &= 1.5 w_0 H_{max} \\ p_U &= 2.0 w_0 H_{max} \\ p_{H_1} &= 2.0 w_0 H_{max} \\ p_{H_2} &= 1.0 w_0 H_{max} \end{aligned}\right\} \quad \textbf{(2.5.1)}$$

where

p_H : intensity of wave pressure acting on the front wall (kN/m^2);

p'_H : intensity of wave pressure acting on the parapet on top of a caisson (kN/m^2);

p_V : intensity of wave pressure acting on the ceiling slab from above (kN/m^2);

p_U : intensity of wave pressure acting on the ceiling slab from below (kN/m^2);

p_{H1} : intensity of wave pressure acting on the upper part of the water chamber (kN/m^2);

p_{H2} : intensity of wave pressure acting on the lower part of the water chamber (kN/m^2);

H_{max} : maximum wave height (m);

w_0 : unit weight of sea water (kN/m^3).

(5) If the top of a water chamber is completely sealed by the ceiling slab, an impulsive pressure may be generated by the compression of the air trapped in the upper part of the water chamber due to waves. The impulsive pressure can be reduced by providing the ceiling slab with ventilation holes with a suitable opening rate. The opening rate of these holes should be carefully determined. If it is too great, the wave surface collides directly with the ceiling slab, and this could produce a greater impulsive uplift than that acting on the nonporous ceiling slab. For details, refer to **References 16)** and **17)**.

(6) **Fig. 2.5.3** shows an example of a model experiment that indicates how the experimental value $P_{\varepsilon 1}$ of uplift intensity changed when the ceiling slab opening rate $_{\varepsilon 1}$ was changed.[17] By providing the ceiling slab with ventilation holes with the opening rate of approximately 0.5% to 1.0%, it is generally possible to reduce the air pressure acting on the ceiling slab to 50% to 70% of that acting on the ceiling slab with no ventilation holes.

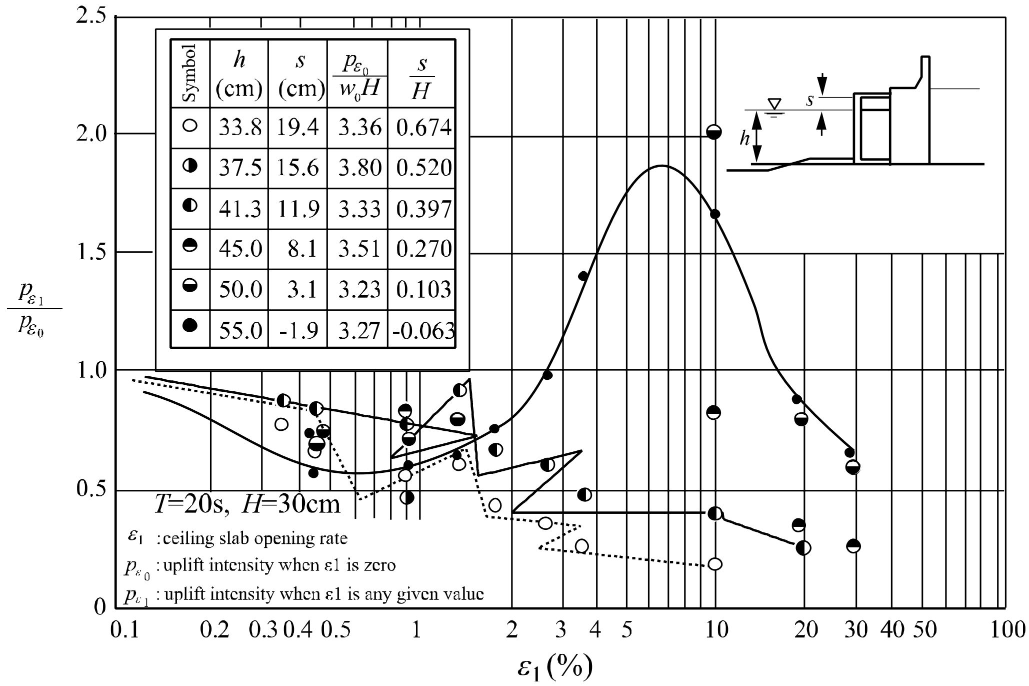


Fig. 2.5.3 Example of an Experiment Indicating the Changes in Uplift Intensity Depending on Ceiling Slab Opening Rate[17]

(7) Front wall slit columns are tall and slender vertical supports. It must be noted that when a drifting object such as driftwood collides with the central part of a slit column and a concentrated load acts on the column, large sectional forces are generated at supporting points. There are many actual cases wherein the impact load caused by a drifting object was assumed to be approximately 78.4 kN, and collisions were treated as accidental actions. The impact load of 78.4 kN was calculated for driftwood under the following conditions on the assumption that slit columns are elastic bearing slabs.

Shape of driftwood: length of 10 m, diameter of 0.5 m, and specific gravity of 0.75
Speed at the moment of collision: 0.5 m/s (approximately 1 kt)
Shape of slit column: length of 4 m and cross section of 0.5 m × 0.7 m

(8) **Table 2.5.1** shows the actions to be considered in the performance verifications of the members of wave chambers of an upright wave-absorbing caisson. In general, these actions shall be assumed to work on the members in the following ways.

Table 2.5.1 Actions to Be Considered for the Members of the Water Chambers of Wave-Dissipating Caisson

Member		Member number	Actions	Remarks
Front wall	Slit column	①	• Water pressure while afloat • Wave pressure (parallel/perpendicular to face line) • Impact load caused by driftwood and other drifting objects • Axial force transmitted from the upper part of front wall	
	Partition wall slit column	②	• Wave pressure including the wave force transmitted from the partition wall	
	Side wall slit column	③	• Water pressure while afloat including the wave force transmitted from side walls • Wave pressure (ditto)	
	Upper beam	④	• Axial loads from above and below • Water pressure while afloat (reaction transmitted from slit columns) • Wave pressure (wave force acting on the beam itself and slit column reaction)	Examine the extraction that might occur when the reaction of a side wall slit column to the wave pressure acts on the upper beam.
	Lower beam	⑤	• Water pressure while afloat (the reaction from slit columns and lower slabs and the load acting on the beam itself) • Wave pressure (ditto)	Ditto
	Lower slab	⑥	• Water pressure while afloat • Wave pressure	
Side wall		⑦	• Water pressure while afloat • Wave pressure	
Partition wall		⑧	• Wave pressure acting on both sides separately in the directions parallel to the face line • Fender reaction	Examine the extraction that might occur when the internal wave pressure acts on the slit columns.
Rear wall		⑨	• Wave pressure • Earth pressure and residual water pressure	Ditto
Bottom slab		⑩	• Bottom reaction and bottom slab weight in each design situation, water head difference, and water pressure while float	
Ceiling slab		⑪	• Wave pressure (upwards, downwards) • Surcharge • Self-weight	

Note: Member numbers correspond to those shown in **Fig. 2.5.1**.

① Slit columns

(a) The actions to be considered in the examination of sectional forces in the slit columns shall be 1) water pressure while afloat, 2) waves, and 3) impact load caused by driftwood and other drifting objects. For the distributions of actions, refer to **Figs. 2.5.4 (a)** to **(c)**.

1) Water pressure while afloat (**Fig. 2.5.4 [a]**)

$$P_a = p_a' \cdot l \qquad \textbf{(2.5.2)}$$

where

P_a : design value of the load acting on one slit column (kN/m);

p_a' : water pressure acting when caisson is afloat (kN/m^2);

l : distance between the centerlines of slit columns (m).

2) Waves (**Fig. 2.5.4 [b]**)

i. When the wave pressure acts from the direction perpendicular to the face line

$$P_{H_1} = p_{H_1}{'}B_1 \quad \textbf{(2.5.3)}$$

where

P_{H1} : design value of the load acting on one slit column (in direction perpendicular to face line) (kN/m);

$p_{H1}{'}$: intensity of wave pressure acting in direction perpendicular to the face line (kN/m^2);

B_1 : width of the slit column in the direction parallel to the face line (m).

ii. When the wave pressure acts from a direction parallel to the face line

$$P_{H_2} = p_{H_2}{'}B_2 \quad \textbf{(2.5.4)}$$

where

P_{H2} : design value of the load acting on one slit column (kN/m);

$p_{H2}{'}$: intensity of wave pressure acting in the direction parallel to the face line (kN/m^2);

B_2 : width of a slit column (m).

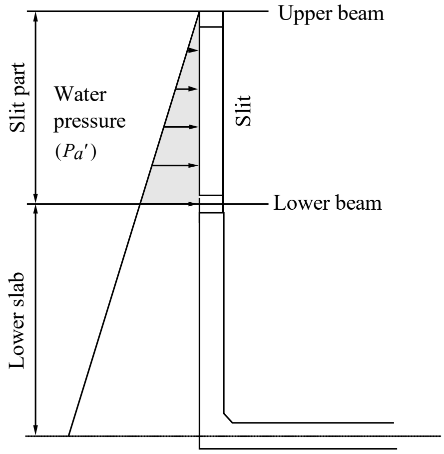
3) Impact load caused by driftwood and other drifting objects (**Fig. 2.5.4 [c]**)

Although the intensity of the impact load caused by driftwood and other drifting objects has not been fully clarified yet, there was a case in which the impact load was calculated to be the following values under the conditions shown in **Part III, Chapter 2**, **2.5.2 (7)**.

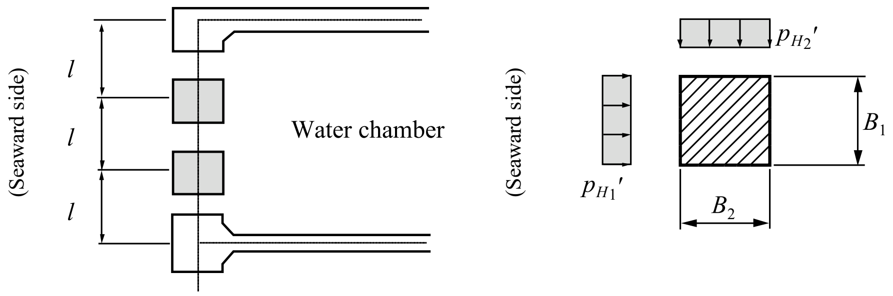
P = 78.4 (kN per slit column) (accidental action); **(2.5.5)**

P' = 52.3 (kN per slit column) (variable action)**.** **(2.5.6)**

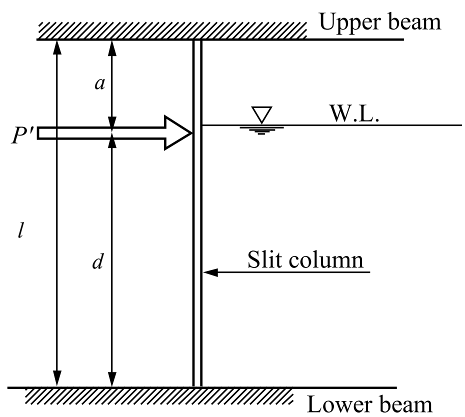
It is preferable to examine the points at which the loads act both in the case wherein the water level is LWL and in the case wherein the water level is HWL.



(a) Water pressure while afloat



(b) Actions of wave pressure



(c) Impact load caused by driftwood and other drifting objects

Fig. 2.5.4 Actions on Slit Columns

(b) The axial forces on slit columns shall be calculated by referring to **equations (2.5.7)** and **(2.5.8)**.

$$P_c = P_v + w_1 + w_2 \tag{2.5.7}$$

$$P_t = P_U - w_1 - w_2 \tag{2.5.8}$$

where

P_c : design value of the axial compressive force acting on slit columns (kN);

P_t : design value of the axial tensile force acting on slit columns (kN);

P_v : downward wave force acting on the ceiling slab and borne by the upper beams as load (kN);

P_U : uplift acting on the ceiling slab and borne by the upper beams as load (kN);

w_1 : self-weight of the ceiling slab (kN);

w_2 : self-weight of the upper beams (kN).

For the axial actions on upper beams, refer to **Fig. 2.5.5**.

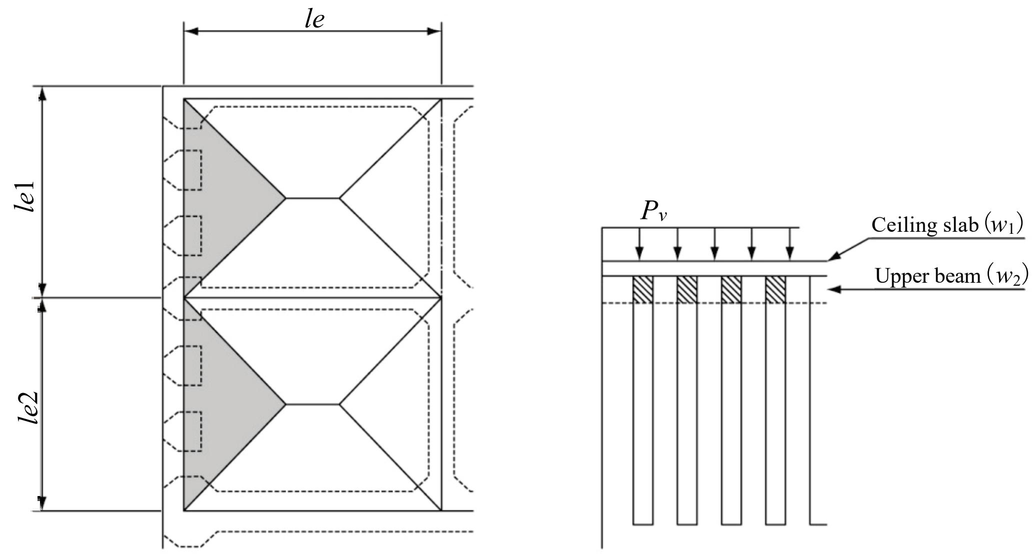


Fig. 2.5.5 Axial Actions on Slit Columns

② **Partition wall slit columns**

The wave pressure acting on the inside of water chambers shall be considered in the examination of sectional forces in a partition wall slit column. The actions on the partition wall slit column can be calculated by using **equation (2.5.9)** and by referring to the distributions of the wave pressure shown in **Fig. 2.5.6**.

$$P_p = p_a\left(b + l_0/2\right) \quad \textbf{(2.5.9)}$$

where

P_p : design value of the action on the partition wall slit column (kN/m);

p_a : intensity of the wave pressure (kN/m^2);

b : width of the slit column (m);

l_0 : width of the water chamber (m).

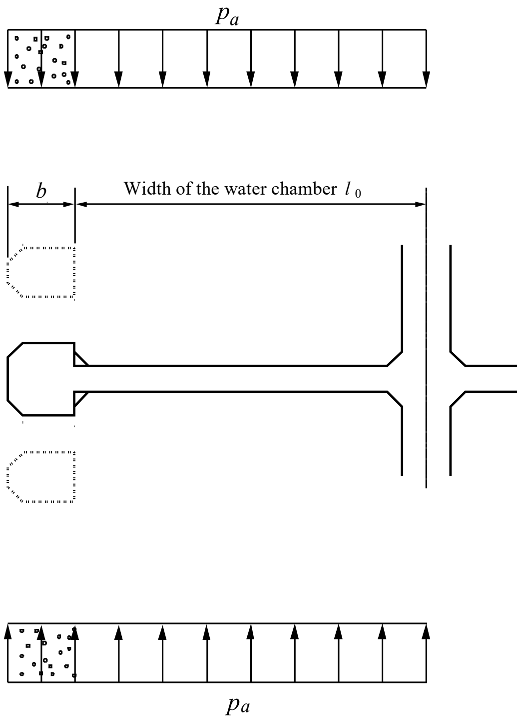


Fig. 2.5.6 Wave Pressure Acting on Partition Wall Slit Column

③ **Side wall slit column**

The water pressure while afloat and the wave pressure acting on the inside of water chambers shall be considered in the examination of sectional forces in a side wall slit column. The actions on the side wall slit column can be calculated by using **equation (2.5.10)** and by referring to the distributions of the wave pressure shown in **Fig. 2.5.7**.

$$P_p = p_a\left(b + l_0/2\right) \qquad \textbf{(2.5.10)}$$

where

P_s : design value of action on side wall slit column (kN/m);

p_a : intensity of water pressure or wave pressure (kN/m^2);

b : width of slit column (m);

l_0 : width of water chamber (m).

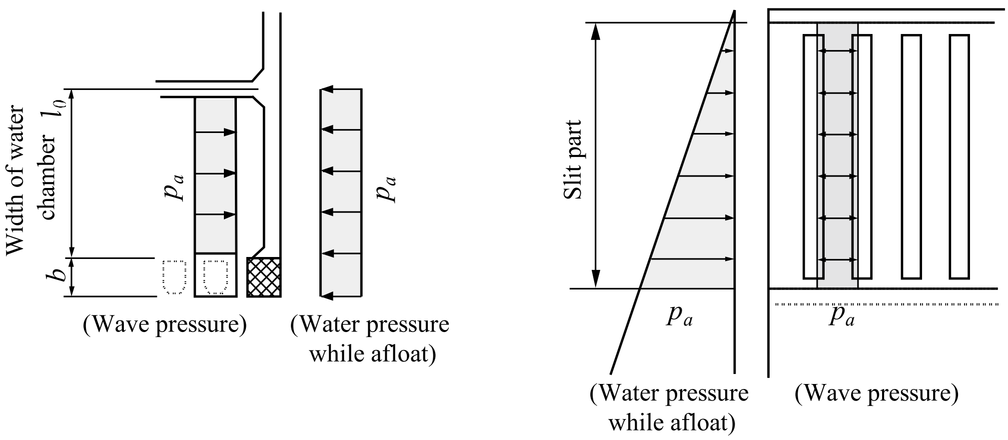


Fig. 2.5.7 Water Pressure and Wave Pressure Acting on the Side Wall Slit Column

④ **Upper beams**

(a) Horizontal actions

The horizontal actions to be considered in the examination of sectional forces in the upper beams shall be the support reaction transmitted from the slit columns and the direct actions on the beams themselves, i.e., the water pressure while afloat and the wave pressure. For the actions on upper beams, refer to the distributions of the water pressure while afloat and the wave pressure shown in **Fig. 2.5.8**. An upper beam shall be assumed as a continuous beam supported by side walls and partition walls, and the action that maximizes sectional forces shall be taken into consideration.

(b) Vertical actions

The wave pressure acting on the ceiling slab and the self-weights of the ceiling slab and upper beams shall be considered vertical actions on the upper beams. In the examination of sectional forces in an upper beam, it shall be assumed a beam fixed at both ends of which the span is the distance between the centerlines of slit columns. Furthermore, the axial forces and vertical forces on the slit columns shall be determined.

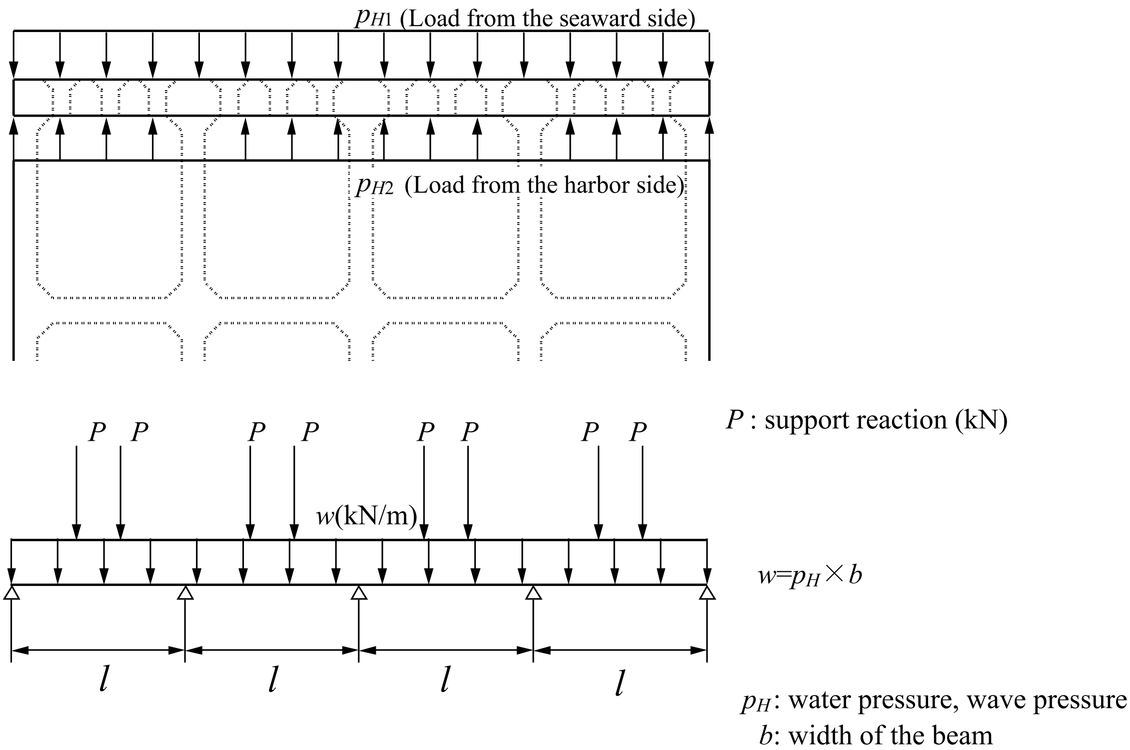


Fig. 2.5.8 Actions on the Upper Beam

⑤ **Lower beams**

The actions to be considered in the examination of sectional forces in lower beams shall be the support reaction transmitted from the slit columns and lower slabs and the direct actions on the beams themselves, i.e., the water pressure while afloat and the wave pressure. For the actions transmitted from lower slabs, refer to the distributions of actions shown in **Fig. 2.5.9**. The support reaction transmitted from slit columns and the actions on the beams themselves can be determined in the same way as that for upper beams.

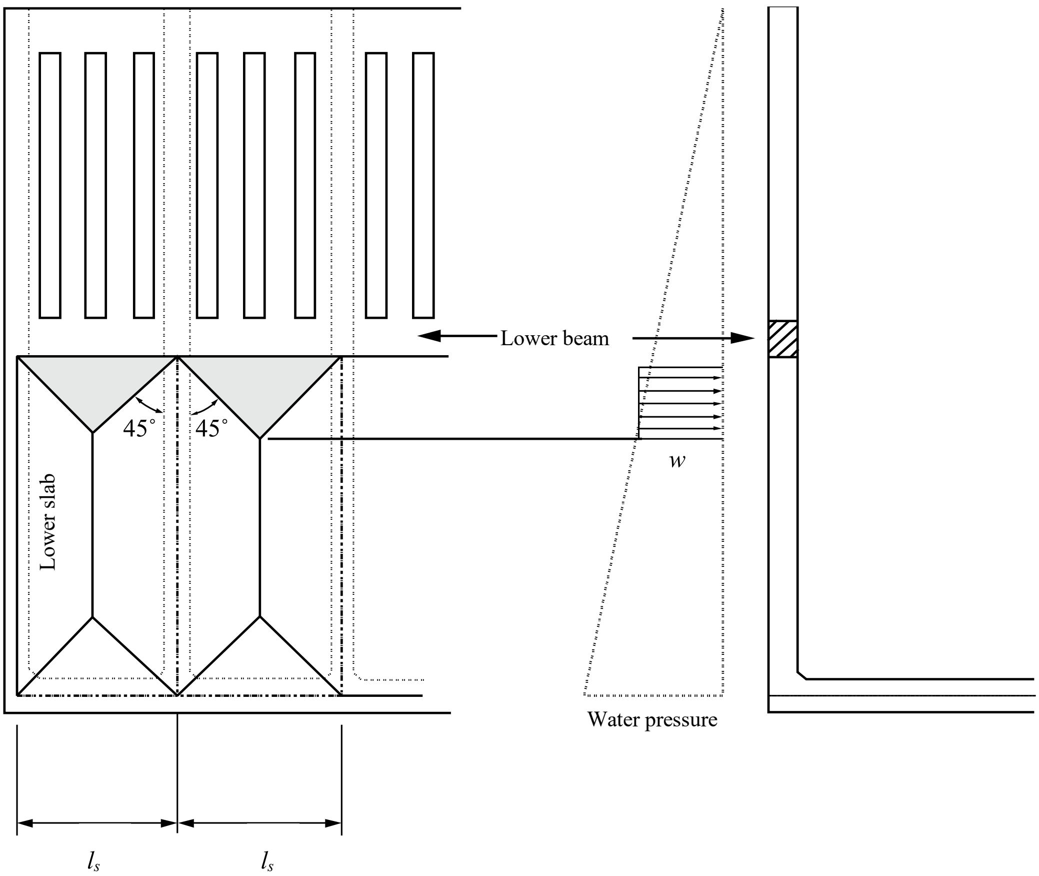


Fig. 2.5.9 Actions on the Lower Beam

⑥ For actions on the other members of upright wave-absorbing caissons, refer to **Part III**, **Chapter 2**, **2.2 Caissons** and descriptions about the relevant matters of similar facilities. Fender reaction acting on partition walls may be determined in accordance with **Part III**, **Chapter 5**, **9.2 Fender Systems**.

2.5.3 Performance Verification of Members

(1) The span to be considered in calculations shall be the distance between the centerlines of bearing members in principle.

(2) **Table 2.5.2** shows the common methods for calculating the sectional forces in the members of water chambers.

(3) For mooring facilities composed of upright wave-absorbing caissons, it is common that a rubber fender or another type of fender is installed on the front surface of a partition wall slit column located in the central part of a caisson. For a partition wall in such a state, it is advisable to examine the stress on its members caused by the ship berthing force.

(4) For the ceiling slab, upper beams, slit columns, and other slit caisson members that will be exposed to an environment where they might be damaged by seawater, appropriate measures should be taken to prevent the members from losing their required performance during the design service life owing to material deterioration. For the performance verification of the members, refer to **Part III**, **Chapter 2**, **1.2.4 Examination of Change in Performance over Time**.

Table 2.5.2 Analytical Models for the Members of the Water Chambers of Slit Caissons

Member		Member number	Analytical model	Remarks
Front wall	Slit column	①	Beam fixed at both ends	Performance verification shall include the examination of axial forces.
	Partition wall slit column	②	Beam fixed at both ends	
	Side wall slit column	③	Beam fixed at both ends	
	Upper beam	④	Continuous beam Beam fixed at both ends	Performance verification shall include the examination of changes in performance over time for intermediate beams.
	Lower beam	⑤	Continuous beam Beam fixed at both ends	Performance verification shall include the examination of changes in performance over time for lower beams, if they might be exposed to a severe marine environment.
	Lower slab	⑥	Slab fixed on four sides	
Side wall		⑦	Slab fixed on three sides and free on one side Slab supported on four sides	If integrated with the ceiling slab
Partition wall		⑧	Slab fixed on three sides and free on one side Slab supported on four sides	If integrated with the ceiling slab
Rear wall		⑨	Slab fixed on three sides and free on one side Slab fixed on four sides	If integrated with the ceiling slab
Bottom slab		⑩	Slab fixed on four sides	
Ceiling slab		⑪	Slab free at four sides Slab fixed on four sides Slab fixed on three sides and free on one side	Depending on the ceiling slab structure and bearing conditions

Note: Member numbers correspond to those shown in **Fig. 2.5.1**.

(5) For the performance verification of lifting points for an upright wave-absorbing caisson that will be lifted, refer to **Part III**, **Chapter 2**, **2.3.5 Verification of Lifting Points**.

2.6 Hybrid Caissons

[Public Notice] (Performance Criteria of Hybrid Caissons)

Article 27

The provisions of Article 23 apply mutatis mutandis to the performance criteria of a hybrid caisson (a caisson having a composite structure of steel plates and concrete)

[Interpretation]

8. Members Composed of Facilities Subject to the Technical Standards

(7) Performance Criteria of Hybrid Caissons (Article 7, paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 27 of the Public Notice)

① Performance criteria of caissons and their interpretation shall be applied correspondingly to hybrid caissons.

② In addition to the provisions of the preceding item, serviceability shall be the required performance for hybrid caissons under the permanent situation in which the dominating action is the internal earth pressure of caissons and under the variable situation in which the dominating actions are water pressure during installation, variable waves, and Level 1 earthquake ground motions. The performance verification items for the actions and standard indices for setting the limit values shall be in accordance with **Attached Table 8-13**.

Attached Table 8-13 Performance Verification Items and Standard Indices for Setting the Limit Values for Hybrid Caissons

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design situation</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index for setting the limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>Situation</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="4">7</td><td rowspan="4">1</td><td rowspan="4">-</td><td rowspan="4">27</td><td rowspan="4">-</td><td rowspan="4">-</td><td rowspan="4">Serviceability</td><td rowspan="4">Permanent and variable</td><td rowspan="2">Water pressure during installation</td><td rowspan="2">—</td><td>Cross-sectional failure of the partition wall (axial force, bending, and shear)</td><td>Design ultimate capacity
Design ultimate capacity considering local buckling</td></tr>
<tr><td>Extrusion of members</td><td>Design ultimate capacity for the extrusion of members</td></tr>
<tr><td rowspan="2">Variable wave [Level 1 earthquake ground motion]</td><td rowspan="2">Self-weight, surcharge, bottom slab reaction, internal earth pressure, internal water pressure, earth pressure, and force transmitted from footing</td><td>Cross-sectional failure of the partition wall (axial force, bending, and shear)</td><td>Design ultimate capacity
Design ultimate capacity considering local buckling</td></tr>
<tr><td>Extrusion of members</td><td>Design ultimate capacity for the extrusion of members</td></tr>
</table>

Ministerial Ordinance			Public notice			Performance requirements	Design situation			Verification item	Standard index for setting the limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
								Internal earth pressure [Variable wave] [Level 1 earthquake ground motion]	Internal earth pressure*1), internal water pressure, and force transmitted from footing	Cross-sectional failure of the outer wall of a composite structure*2) (Horizontal slip shear force)	Design horizontal shear transfer capacity
										Cross-sectional failure of the outer wall of a composite structure*2) (Bending, shear)	Design ultimate capacity Design ultimate capacity considering local buckling
										Cracking of the outer wall of a composite structure*2)	Crack width caused by bending

*1): Action to be considered under situations in which the dominating action is variable waves or Level 1 earthquake ground motions

*2): A slab member (composite slab) comprising steel plates and concrete integrated by appropriate shear connectors

2.6.1 General

(1) **Fig. 2.6.1** shows an example of a hybrid caisson structure used in port and harbor facilities. Hybrid caisson structures are commonly comprised of two types of members as shown in **Fig. 2.6.2**: a composite slab structure with steel plates arranged on one side only, and a steel-reinforced concrete (SRC) structure with embedded H-shaped steel.

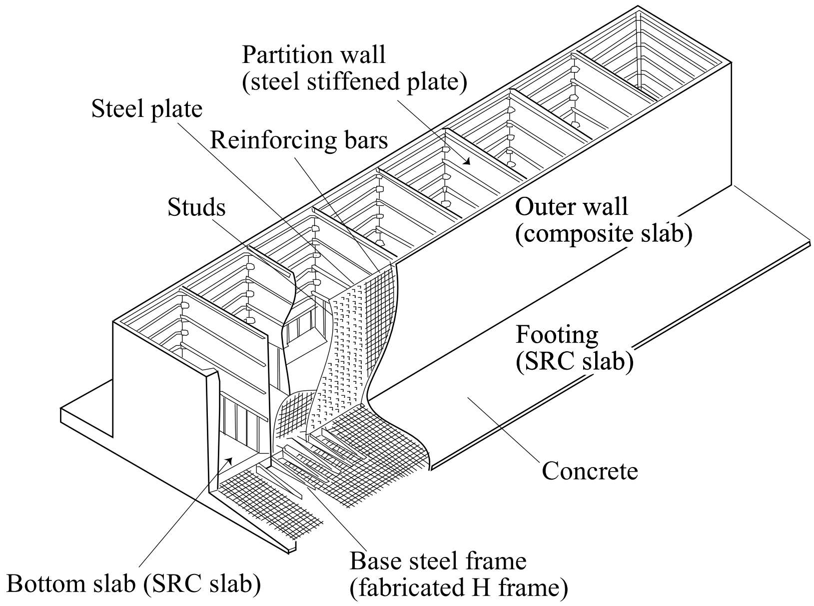


Fig. 2.6.1 Example of a Hybrid Caisson Structure

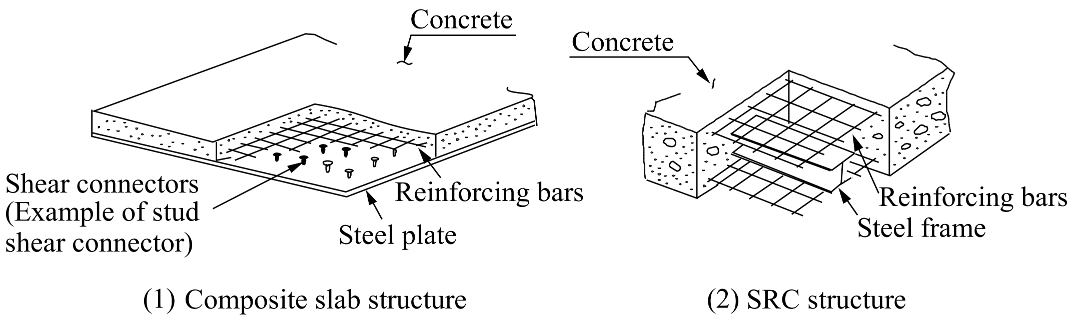


Fig. 2.6.2 Hybrid Caisson Structural Members

(2) Hybrid caissons have the following structural and functional characteristics, which require careful attention:

① **Materials used**

(a) Steel plates and steel frames are placed as substitutes for reinforcing bars. This improves the mechanical properties of caissons and allows them to have the required load bearing capacity and deformability (toughness) even with thin members, thereby increasing the flexibility of the structure.

(b) It is necessary to take measures to prevent steel plates from corroding.

② **Cross-sectional shape**

(a) Generally, the footings of hybrid caissons can be made longer than those of conventional reinforced concrete caissons, and long footings can contribute to the reduction of the subgrade reaction that occurs on the bottom of the caisson.

(b) It is possible to design large caissons with a hybrid structure. However, when doing so, it is necessary to pay close attention to the effect of torsion.

③ **Self-weight of a box-shaped structure**

It is possible to design lightweight caissons with a small draft.

④ **Others**

(a) The presence of steel plates ensures sufficient watertightness even after cracking occurs in concrete.

(b) It is possible to facilitate the reinforcing bar arrangement work, e.g., via the automatic welding of bars in a workshop. Timbering, concrete formwork, and construction joint treatment work can also be reduced by utilizing steel plates for forms that are used for concrete placement.

(c) It is possible to achieve the weight reduction of structures and improve constructability.

(d) Shear connectors and other steel materials are densely arranged in members; therefore, it is necessary to place concrete carefully.

2.6.2 Fundamentals of Performance Verification

(1) For the performance verification of hybrid caissons, refer to the **Hybrid Caisson Deign Manual** [18] and **References 19)** and **20)**.

(2) For the performance verification of hybrid caissons, refer to **Part III**, **Chapter 2**, **2.2 Caissons** in principle. For the performance verification of composite slabs, refer to **Fig. 2.6.3**.

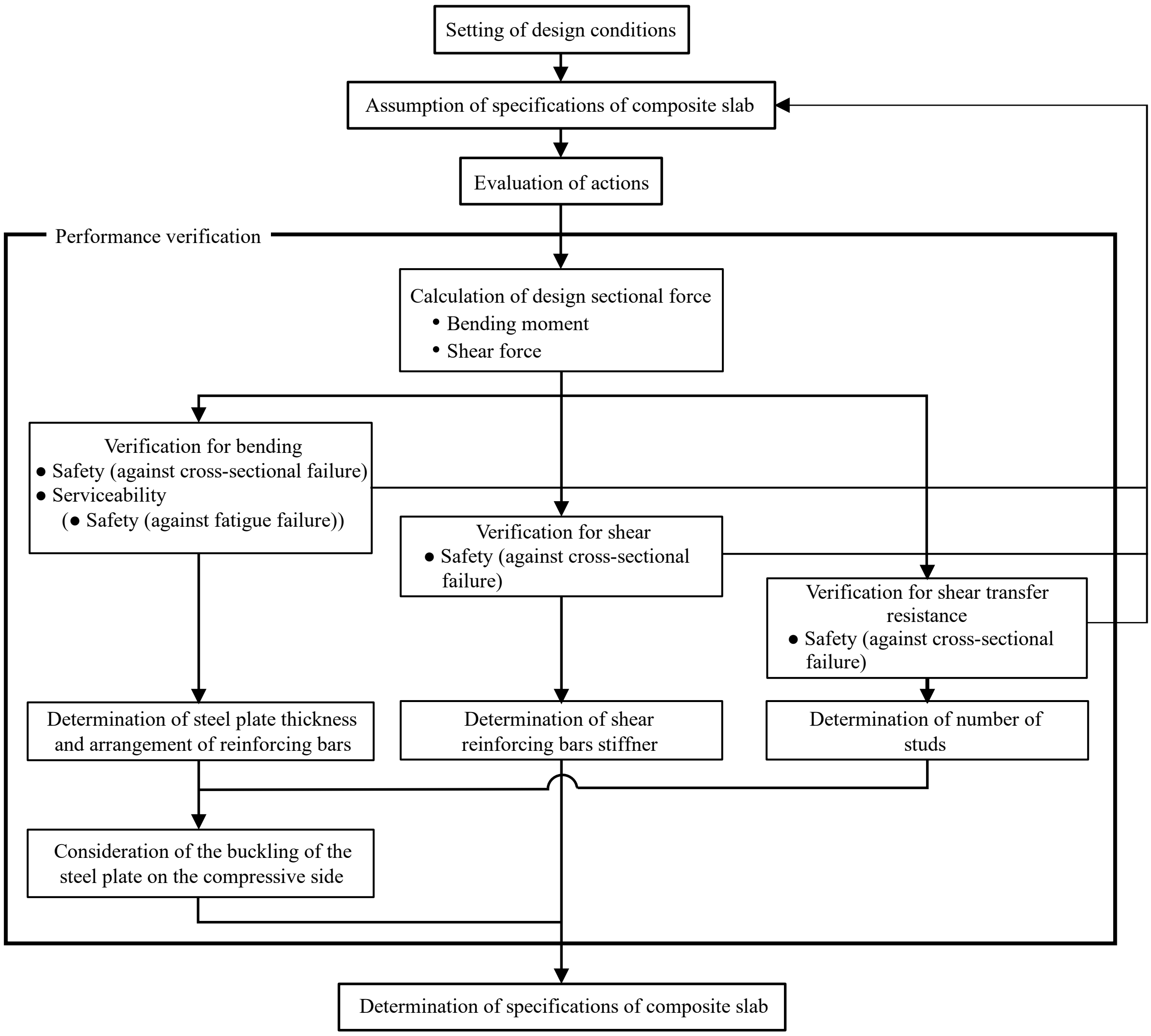


Fig. 2.6.3 Example of the Performance Verification of the Composite Slab of Hybrid Caisson

2.6.3 Actions

For the actions to be considered in the performance verification of hybrid caissons, refer to **Part III**, **Chapter 2**, **2.2.3 Actions**. In cases wherein partition walls in a hybrid caisson are made of steel, it is preferable to consider the actions due to the difference in water pressure between the inside and outside of the caisson while afloat and during installation; the actions of earth pressure, waves, and others; and the bottom reaction of the bottom slab and footings as actions on the partition walls.

2.6.4 Performance Verification

(1) Calculation of Sectional Forces

① Sectional forces in the footings, bottom slab, outer walls, partition walls, corners, and other members of a hybrid caisson shall be examined in principle. For the calculations of sectional forces, refer to **Part III**, **Chapter 2**, **2.2.4 Performance Verification**.

② In cases wherein a caisson has long protruding footings and their bases are subjected to large bending moments, it is advisable to consider the effects of the bending moments on the bottom slab and outer walls in the way described in the **Hybrid Caisson Deign Manual** [18].

③ Some in-plane deformation occurs in the partition walls of a caisson when they are subjected to actions. This in-plane deformation affects sectional forces. Specifically, bending moments occur in outer walls serving as fixed slabs and additional bending moments occur in the corners of outer walls owing to the deformation of partition walls. These effects can generally be disregarded for a hybrid caisson with ordinary dimensions and specifications. However, there are cases wherein these effects cannot be disregarded owing to the dimensions of a caisson and the magnitudes of actions on it. In such cases, it is advisable to examine the effects in the way described in the **Hybrid Caisson Deign Manual** [18].

④ It is important to examine the performance of steel plates subject to compressive stress against buckling. Therefore, it is advisable to carefully determine the buckling length and boundary conditions with consideration to the relative stiffness of the shear connectors to be used and other conditions.

⑤ When designing large caissons, it is advisable to examine torsions.

(2) Performance Verification of Composite Slabs

In the performance verification of composite slabs, the following items shall be considered in principle:

① Bending moments in a composite slab can be calculated as bending moments acting on a cross section of a steel plate or as the tensile or compressive reinforcement of a double-reinforced concrete member.

② The shear force in a composite slab can be calculated in the same manner as that in a reinforced concrete slab.

③ Integration of Steel and Concrete

Shear connectors are particularly important structural elements for the integration of materials in a hybrid structure. In composite slabs, stud shear connectors and shape steel are most commonly used as shear connectors. The required quantity and arrangement of shear connectors shall be determined appropriately to ensure that they work adequately to prevent steel plates from separating from concrete in the out-of-plane direction (particularly when compressive stress is active) and transmit the shear force occurring on the interface between steel plates and concrete.

(3) Performance Verification of SRC Members

① SRC members shall be verified against the bending moments and shearing force by taking into account the mechanical characteristics due to the differences in the structural type of steel frame.

② SRC members can normally be classified as follows, depending on the structural type of steel frame:

(a) Full-web type

(b) Truss-web type

③ The bending moments in an SRC member can be calculated as bending moments acting on a cross section of a reinforced concrete member with reinforcements converted from steel frames. When the fixing of steel frame ends with concrete is insufficient in a full-web-type SCR member, the bending moments acting on the cross section may be calculated separately for the steel frame part and the reinforced concrete part of the SRC member, and the sum of the calculated bending moments may be considered the strength of the entire SRC member.

④ The shear force in a truss-web-type SRC member can be calculated as a shear force acting on a cross section of a reinforced concrete member with reinforcements converted from steel frames. For a full-web-type SRC member, performance verification can be conducted by giving appropriate consideration to the fact that the steel frames can resist the shear force.

(4) Performance Verification of Partition Walls

① Considering that partition walls function as the supported edges of outer walls and the bottom slab, it shall be confirmed by performance verification that the cross sections of partition walls have stability against the sectional forces calculated on the basis of the actions on these supported edges.

② The performance verification procedure for the partition walls of a caisson should include the examinations of performance during flotation and the installation of the caisson and the verification of the examination results at the time of completion of the caisson in principle.

(a) It is advisable to examine performance against the buckling of members during the flotation of the caisson.

(b) It is advisable to examine performance against the buckling and out-of-plane bending of members during the installation of the caisson.

(c) It is advisable to verify the performance of partition walls against the in-plane stress of members at the time of completion of the caisson.

(5) Performance Verification of Corners and Joints

① Corners and joints shall be designed to smoothly and firmly transmit sectional forces and to be easily fabricated and constructed.

② To secure sufficient strength at corners and joints, it is desirable to firmly connect the steel materials on the tensile side to those on the compressive side. It is also desirable to provide shear reinforcing steel stiffeners (haunches) against concrete tensile stress that occurs in joints.

(6) Performance Verification for Fatigue Failure

① Hybrid caissons have a large number of welded joints that connect steel plates to each other and attach shear connectors, shear reinforcing steel stiffeners, and the like. Therefore, the hybrid caissons with members that will be subjected to a repeated action require the verification of the fatigue strength of the welds of the members.

② Revetments and quaywalls are less affected by repeated actions. However, in the performance verifications of breakwaters, it is necessary to examine the performance of hybrid caissons against fatigue failure if the stress on members due to the repeated action of waves varies significantly.

(7) Verification of Caissons during Lifting

For the performance verification of the lifting points of hybrid caissons that will be lifted, refer to **References 18)** and **21)** and **Part III**, **Chapter 2**, **2.3.5 Verification of Lifting Points** in consideration of the structure of the lifting points.

2.6.5 Corrosion Protection

(1) The corrosion protection of hybrid caissons shall be determined appropriately by considering the performance requirements, maintenance level, construction conditions, and other relevant factors.

(2) The main cause of deterioration of hybrid members is the corrosion of their steel materials. Furthermore, there are cases in which the corrosion of the steel materials may result in the cracking of concrete. Therefore, appropriate corrosion prevention measures should be taken for steel plates in order to improve the durability of the hybrid members. The deterioration characteristics of the concrete itself of hybrid caissons may be considered to be the same as those of concrete of reinforced concrete caissons.

(3) When corrosion protection is applied to hybrid caissons, an appropriate method shall be determined on the basis of research on the performance of existing steel port facilities and by utilizing data on corrosion.

(4) Steel materials used on the outside of hybrid caissons are generally covered with concrete or asphalt mats. The steel materials used on the inside of hybrid caissons are isolated from the external atmosphere by concrete lids and are in contact with filling sand in a static state and with seawater. Therefore, when designing a hybrid caisson, it is common to adopt a structure that prevents direct contact between the steel plates of members and the marine environment by placing steel plates inside so that they will not be directly exposed to wave actions but will be protected by concrete from corrosion. If steel plates will be in direct contact with seawater, appropriate measures should be taken to prevent them from losing their required performance during the design service life due to material deterioration. For performance verification, refer to **Part III**, **Chapter 2**, **1.4.4 Examination of Change in Performance over Time**.

2.7 Armor Stones and Blocks

[Public Notice] (Performance Criteria of Armor Stones and Blocks)

Article 28

Performance criteria of rubbles and concrete blocks armoring a structure exposed to actions of waves and water currents, as well as the armor stones and armor blocks of a foundation mound, shall be such that the risk of exceeding the allowable degree of damage under the variable situation, in which the dominating actions are variable waves and water currents, is equal to or less than the threshold level.

[Interpretation]

8. Members Composed of Facilities Subject to the Technical Standards

(8) Performance Criteria of Armor Stones and Blocks (Article 7, paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 28 of the Public Notice)

① Serviceability shall be the required performance for armor stones and blocks under the variable situation in which the dominating actions are variable waves and water currents. The performance verification items for the actions and standard indices for setting the limit values shall be in accordance with **Attached Table 8-14**.

Attached Table 8-14 Performance Verification Items and Standard Indices for Setting the Limit Values for Armor Stones and Blocks

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting the limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	28	-	-	Serviceability	Variable	Variable waves [Water currents]	Self-weight, water pressure	Extent of damage	Damage rate, degree of damage, or deformation level

* The action shown in brackets is an alternative dominating action.

② Standard indices for setting the limit values shall be determined appropriately by considering the design service life of the facility of interest, its construction conditions, the time and cost required for its restoration, conditions of waves and water currents, and other factors.

2.7.1 Fundamentals of Performance Verification

(1) The performance verification of armor units, such as slope armor units for sloping breakwaters, armor stones and blocks for the mounds of composite breakwaters, and rubble and other materials for mounds exposed to water currents, shall be conducted on the basis of **Part II**, **Chapter 2**, **6.6 Stability of Armor Stones and Blocks against Waves**.

(2) When plain concrete members are used as armor stones and blocks, the verification of lifting points shall be conducted in accordance with **Part III**, **Chapter 2**, **2.7.2 Verification of Plain Concrete Members during Lifting**.

2.7.2 Verification of Plain Concrete Members during Lifting

(1) Verification of Suspension Hooks

For the verification of suspension hooks to be used for lifting plain concrete members, refer to **Part III**, **Chapter 2**, **2.3.5 Verification of Lifting Points**.

(2) Verification of the Cross Section of Members during Lifting

① The verification of plain concrete members during lifting for relocation or installation may be conducted by using **equation (2.7.1)**.

$$\gamma_i \frac{M_d}{M_{ud}} \leq 1.0 \qquad \textbf{(2.7.1)}$$

where

M_d : design value of the bending moment (kN・m/m);

M_{ud} : design bending strength of a plain concrete member (kN・m/m);

γ_i : structure factor (=1.1).

② The design value of the bending moment of a plain concrete member may be calculated as the bending moment generated by its self-weight on the assumption that the concrete member is a projecting beam supported at the positions of the suspension hooks.

③ The bending strength of plain concrete shall be calculated as the strength against a crack caused by bending.

$$f_{bck} = k_{0b} k_{1b} f_{tk}$$

$$k_{0b} = 1 + \frac{1}{0.85 + 4.5(h / l_{ch})} \qquad \textbf{(2.7.2)}$$

$$k_{1b} = \frac{0.55}{\sqrt[4]{h}} \quad (\geqq 0.4)$$

where

f_{bck} : characteristic value of the strength of concrete against a crack caused by bending (N/mm^2);

k_{0b} : factor that expresses the relationship between bending strength and tensile strength due to the tension softening property of concrete;

k_{1b} : factor that expresses the reduction in crack strength due to drying, heat of hydration, etc.;

h : height of a member (m) (> 0.2);

l_{ch} : characteristic length (m) (=$G_F E_C / f_{tk}^2$);

G_F : fracture energy of concrete (N/m) (=$10(d_{max})^{1/3} \cdot f'_{ck}{}^{1/3}$);

E_C : Young's modulus of concrete (kN/mm^2);

f_{tk} : characteristic value of tensile strength (N/mm^2);

d_{max} : maximum dimension of an aggregate (mm);

f'_{ck} : characteristic value of compressive strength (N/mm^2).

$$M_{ud} = \frac{1}{2} \cdot f_{bcd} \cdot \frac{h}{2} \cdot b \cdot z \qquad \textbf{(2.7.3)}$$

where

f_{bcd} : design value of the strength of concrete against a crack caused by bending (=f_{bck}/γ_c) (This value may be calculated by assuming that the material factor γ_c is 1.3);

M_{ud} : design bending strength of a plain concrete member (kN・m/m);

b : width of a member (m)

z : center-to-center distance of a member for tensile stress and compressive stress (=2h/3).

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3 Foundations

3.1 General Comments

(1) The foundation structures of the port facilities shall be selected appropriately, giving due consideration to the importance of the facilities and soil conditions of the foundation ground.

(2) When the stability of the foundation structures seems not to be secured, countermeasures such as introduction of pile foundation or soil improvement, etc. should be applied as necessary.

(3) When the foundation ground is soft, excessive settlement or deformation arises owing to the lack of bearing capacity. When the ground consists of loose sandy soil, liquefaction of the ground induced by seismic ground motion may cause facility failure or significant damage to its functions. In such cases, the in-situ stress generated by the weight of facilities needs to be reduced or the foundation ground needs to be reinforced by improvement.

(4) For the stability of foundations, Part III, Chapter 2, **3.2 Shallow Foundations**, Part III, Chapter 2, **3.3 Deep Foundations**, or Part III, Chapter 2, **4 Stability of Slopes** can be used as reference. For settlement of foundations, Part III, Chapter 2, **3.5 Settlement of Foundations**, and for liquefaction induced by seismic ground motion, **Part II, Chapter 7 Ground Liquefaction** can be used as reference. For the performance verification of pile foundations, Part III, Chapter 2, **3.4 Pile Foundations** can be used as reference. In cases where the performance of facility under seismic ground motion needs to be verified, the verification shall be performed corresponding to the characteristics of the respective foundations.

(5) Methods of Reducing In-situ Stress

The followings are methods of reducing *in-situ* stress generated by the weight of structures.

① Reduction of the weight of the structure itself
② Expansion of the area of the bottom of the structure
③ Use of a pile foundation, etc.

Reduction of shear stress induced by weight of facilities, namely improvement of stability, can also be achieved by reduction of eccentricity of actions, which is carried out by increasing resistant by counterweight fill, or reducing load by light weight soil or others.

(6) Soil Improvement Method

For method of soil improvement, Part III, Chapter 2, **5 Soil Improvement Methods** can be used as reference.

3.2 Shallow Foundations

3.2.1 General

(1) When the embedment depth of the foundation is less than the minimum width of the foundation, the foundation may generally be examined as a shallow foundation.

(2) In general, the bearing capacity of a foundation is expressed as the sum of the bottom bearing capacity and the side resistance of the foundation. Bottom bearing capacity of a foundation is the pressure applied to the foundation bottom considered necessary to cause plastic failure in the ground. The side resistance of a foundation is the frictional resistance or the cohesion resistance acting between the sides of the foundation and the soil. Although considerable research has been done on the bottom bearing capacity, relatively little research has been done on side resistance. In the case of shallow foundations, since the magnitude of the side resistance will be small in comparison with that of the bottom bearing capacity, it is not generally necessary to consider the side resistance.

(3) When examining foundations subjected to an eccentric and inclined action, **Part III, Chapter 2, 3.2.5 Bearing Capacity for Eccentric and Inclined Actions** can be used as reference.

3.2.2 Bearing Capacity of Foundations on Sandy Ground

(1) The following equation derived from the Terzaghi's bearing capacity formula shown in (3) can be used to examine the bearing capacity of the foundations on sandy ground.

$$q_{\mathrm{d}} = \frac{1}{m_{\mathrm{B}}}\left(\beta\rho_{1\mathrm{k}}g\frac{B}{2}N_{\gamma\mathrm{k}} + \rho_{2\mathrm{k}}gD\left(N_{q\mathrm{k}}-1\right)\right) + \rho_{2\mathrm{k}}gD \qquad \textbf{(3.2.1)}$$

where

q_d : design value of foundation bearing capacity considering buoyancy of submerged part (kN/m^2)

m_B : adjustment factor for bearing capacity

β : shape factor of a foundation (refer to **Table 3.2.1**)

$\rho_{1k}g$: characteristic value of unit volume weight of soil of ground below the foundation bottom, or unit volume weight in water, if submerged (kN/m^3)

B : minimum width of foundation (m)

$N_{\gamma k}$, N_{qk} : characteristic value of bearing capacity coefficients for strip foundation

$\rho_{2k}g$: characteristic value of unit volume weight of soil of ground above the foundation bottom, or unit volume weight in water, if submerged (kN/m^3)

D : embedment length of foundation in the ground (m)

The adjustment factor m_B for the bearing capacity of shallow foundation is a factor to consider the safety margin of bearing capacity and may be set to an adequate value of 2.5 or more for sandy ground. The bearing capacity of shallow foundation on sandy ground is verified by confirming that the design value of the load strength (the value divided the design value of vertical action load by the ground contact area of foundation) does not exceed the design value of bearing capacity q_d calculated using **equation (3.2.1)**. In the case of sandy ground, attention is required because the adjustment factor regarding bearing capacity is significantly different from the partial factor (the partial factor regarding the resistance moment is identical to the one regarding the shear resistance force) to multiply to resistant term used for circular slip failure analysis and others.

(2) As the application of action to the ground is increased, the ground initially settles in proportion to the action. When the action reaches a certain value, the settlement rapidly increases and a shear failure occurs in the ground. The load strength required to occur a shear failure of the ground is called the ultimate bearing capacity of foundation. The design value of the bearing capacity of foundation can be calculated by dividing the ultimate axial bearing capacity obtained from the bearing capacity formula by the adjustment factor m_B. However, the bearing capacity corresponding to overburden pressure at embedment depth, the stability of which is guaranteed, needs not to be divided by the adjustment factor m_B.

(3) Terzaghi's Bearing Capacity Formula

The bearing capacity q_k is given in **equation (3.2.2)** following the ultimate axial bearing capacity formula for sandy ground indicated by Terzaghi.

$$q_k = \beta\rho_{1k}g\frac{B}{2}N_{\gamma k} + \rho_{2k}gDN_{qk} \tag{3.2.2}$$

where

q_k : characteristic value of the ultimate axial bearing capacity (value considering buoyancy of submerged part) (kN/m^2)

B : minimum width of foundation (diameter in the case of round foundation) (m)

D : embedment length of foundation (m)

$\rho_{1k}g$: characteristic value of unit volume weight of soil of ground below the foundation bottom, or unit volume weight in water, if submerged (kN/m^3)

$\rho_{2k}g$: characteristic value of unit volume weight of soil of ground above the foundation bottom, or unit volume weight in water, if submerged (kN/m^3)

$N_{\gamma k}$, N_{qk} : characteristic value of the bearing capacity coefficient (refer to **Fig. 3.2.1**) [1]. The characteristic value of the bearing capacity coefficient is expressed in the following equation:

$N_{q_k} = \dfrac{1+\sin\phi_k}{1-\sin\phi_k}\exp(\pi\tan\phi_k)$ (Solution of Prandtl)

$N_{\gamma_k} = (N_{q_k}-1)\tan(1.4\phi_k)$ (Solution of Meyerhof)

β : shape factor of foundation (refer to **Table 3.2.1**)

The first term of the right side of **equation (3.2.2)** is the bearing capacity exerted by self-weight $\rho_1 g$ of the soil in the ground when there is no pressing load above the foundation bottom. The bearing capacity coefficient of this term is called N_γ. The second term of the right side is the bearing capacity exerted by the pressing load when there is no soil weight below the foundation bottom. The bearing capacity coefficient of this term is called N_q.

The design value q_d of the bearing capacity is expressed as the load strength by subtracting buoyancy from the whole actions including self-weight of the foundation. Based on the concept that there will be no shear in the ground unless the load strength applied to the foundation bottom exceeds the effective overburden pressure acted at the position of the foundation bottom before excavation, it is reasonable to use the following equation which gives the net ultimate bearing capacity where effective overburden pressure is subtracted rather than using **equation (3.2.2)**. The right side is divided by the adjustment factor m_B as in **equation (3.2.1)**.

$$q_d - \rho_{2k} g D = \frac{1}{m_B}\left(\beta \rho_{1k} g \frac{B}{2} N_{\gamma k} + \rho_{2k} g D (N_{qk} - 1)\right) \qquad \textbf{(3.2.3)}$$

If the second term of the left side is moved to the right side, **equation (3.2.3)** coincides with **equation (3.2.1)**.

Table 3.2.1 Shape Factors

Shape of foundation	Continuous	Square	Round	Rectangular
β	1	0.8	0.6	1–0.2 (B/L)

B: length of short side of rectangle, L: length of long side of rectangle

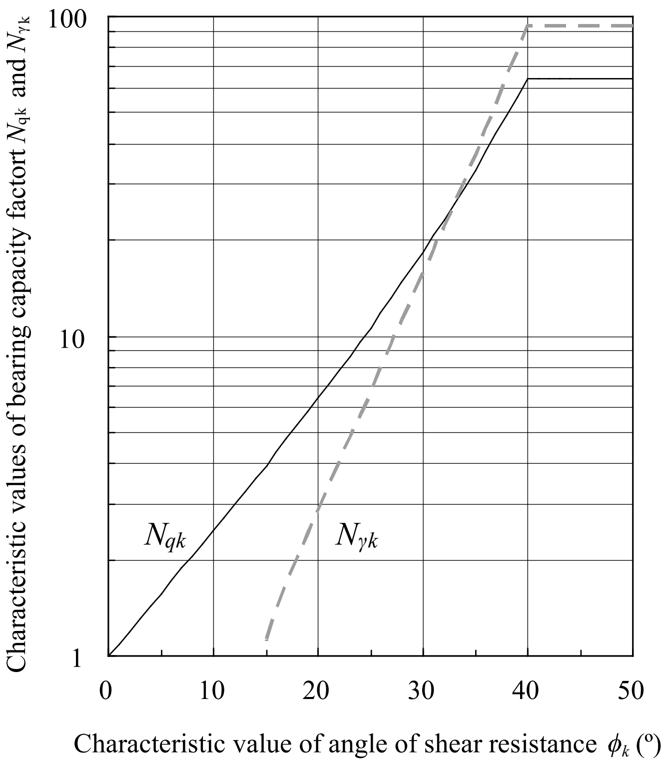


Fig. 3.2.1 Relationship between Bearing Capacity Coefficients N_γ and N_q and Angle of Shear Resistance ϕ

(4) General Shear and Local Shear

Fig. 3.2.2 shows the plastic equilibrium condition in ground supposed on Terzaghi's bearing capacity theory. Sliding surface is considered only below the depth of foundation bottom, and the soil above it is considered as pressing load. In the plastic equilibrium condition of the ground shown in **Fig. 3.2.2**, shear failure reaches to the ground level (to the depth of foundation bottom in **Fig. 3.2.2**). This is the shear failure that occurs when the ground is much dense or stiff and failure strain is small; which Terzaghi called the general shear failure. On the contrary, if the soil is loose or soft and highly compressive, local shear failure of the soil below foundation induces large settlement before plastic failure reaches the range shown in **Fig. 3.2.2** and may result in a practical failure. This type of failure is called local shear failure (or partial shear failure). **Fig. 3.2.3** shows these two failures in terms of the relation between the load strength and the settlement in loading test.

The types of these two shear failures are mainly discriminated base on individual judgment. Terzaghi advocates to empirically use two thirds of tanϕ when local shear failure is expected, which can be adopted.

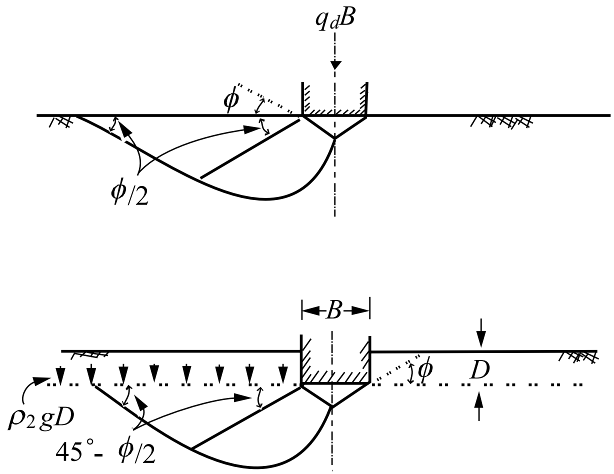


Fig. 3.2.2 Plastic Equilibrium Condition in the Ground below Continuous Foundation

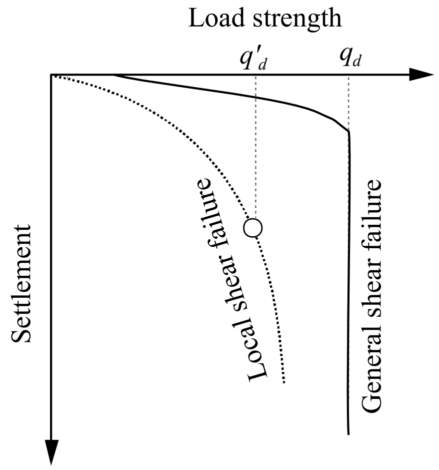


Fig. 3.2.3 Relation between Load Strength and Settlement in Loading Test

3.2.3 Bearing Capacity of Foundation on Clayey Ground

(1) The next equation can be used for examination of bearing capacity of foundations on clayey ground.

$$q_d = \frac{1}{m_B} N_{c0k} \left(1 + n \frac{B}{L} \right) c_{0k} + \rho_{2k} gD \tag{3.2.4}$$

where

q_d : design value of foundation bearing capacity considering buoyancy of submerged part (kN/m^2)

m_B : adjustment factor for bearing capacity

N_{c0k} : characteristic value of bearing capacity coefficient for strip foundation adhesive force

n : shape factor of foundation (refer to **Fig. 2.2.4**)

B : minimum width of foundation (m)

L : length of foundation (m)

c_{0k} : characteristic value of undrained shear strength of clayey soil at the foundation bottom (kN/m^2)

$\rho_{2k}g$: characteristic value of unit volume weight of soil of ground above the foundation bottom, or unit volume weight in water, if submerged (kN/m^3)

D : embedment length of foundation in the ground (m)

The adjustment factor m_B for the bearing capacity of shallow foundation is a factor to consider the safety margin of bearing capacity and may be set to an adequate value of 1.5 or more for clayey ground. When it is expected that slight settlement or deformation of the ground significantly impair the function of superstructure such as crane, it is desirable to set an adequate value corresponding to characteristic of facilities such as an adjustment factor m_B of 2.5 or more.

(2) Bearing Capacity Coefficient of Clayey Ground Considering the Increase in Strength in Depth Direction

As the shear strength of clayey ground in port areas often increases linearly with depth, the characteristic value N_{c0k} of bearing capacity coefficient in **equation (3.2.4)** can be calculated using **Fig. 3.2.4** that takes account of the change in shear strength within the ground. Here, k is the increase rate of strength in the depth direction. If the surface strength is c_0, the undrained shear strength at depth z is expressed by $c_0 + kz$. The characteristic value N_{c0k} of bearing capacity coefficient for strip foundation shown in **Fig. 3.2.4** was given by Davis et al [2] who numerically resolved the Kötter Equation. The shape factor n for homogeneous soil ground is 0.2, while for ground where strength increases in depth direction, it is determined based on the broken line in **Fig. 3.2.4**. The broken line indicates interpolated values using the result of circular slip failure analysis [3]. Round foundations can be considered to correspond to square foundations.

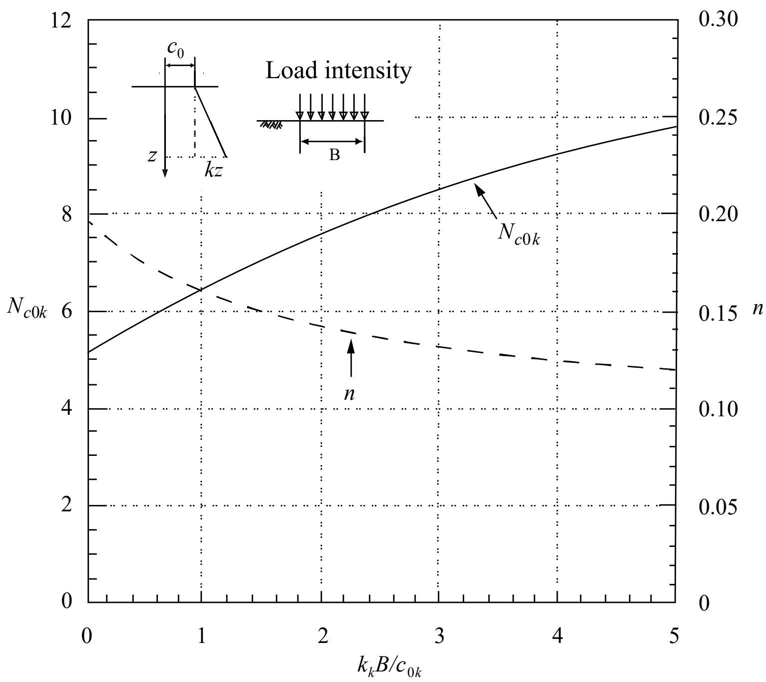


Fig. 3.2.4 Bearing Capacity Coefficient N_{c0k} of Clayey Ground in which Strength Increases in Depth Direction and Shape Factor n

(3) Examination of Bearing Capacity by Stress Distribution in Ground

In the analysis of bearing capacity, the stability, settlement, and deformation properties of the ground at the foundation of facilities are examined. Traditionally, the bearing capacity in the ground at each depth is examined considering the underground stress distribution generated by actions from facilities. If the stability as a whole is ensured by stability of slope analysis and others, there is no need to consider the stress distribution in the ground from the viewpoint of stability evaluation, but it is meaningful as a simple way from the viewpoint of verifying the possibility of relatively large settlement in the ground induced by actions from facilities.

(4) Practical Equation to Calculate the Bearing Capacity

The design value of bearing capacity in the case of continuous foundation can be calculated from the bearing capacity coefficient shown in **Fig. 3.2.4** in the range of $kB/c_{0k} \leq 4$ using **equation (3.2.5)**. Same symbols are used as in **equation (3.2.4)**.

$$q_d = \frac{1}{m_B}(1.018kB + 5.14c_{0k}) + \rho_{2k}gD \qquad (\text{where, } kB/c_{0k} \leq 4) \tag{3.2.5}$$

3.2.4 Bearing Capacity of Multi-layered Ground

(1) Examination of stability for the bearing capacity when the foundation ground has a multi-layered structure can be performed by circular slip failure analysis. Assuming the overburden pressure above the level of the foundation bottom as the surcharge, circular slip failure analysis, which is described later in detail in **Part III, Chapter 2, 4.2 Examination of Stability**, is performed by the modified Fellenius method for an arc passing through the edge of the foundation, as shown in **Fig. 3.2.5**. A value of 1.5 or more can generally be set as the adjustment factor m_B regarding the bearing capacity of multi-layered ground, but in cases where settlement will have a large effect on the functions of the facilities like crane foundation, it is preferable to set a value of not less than 2.5.

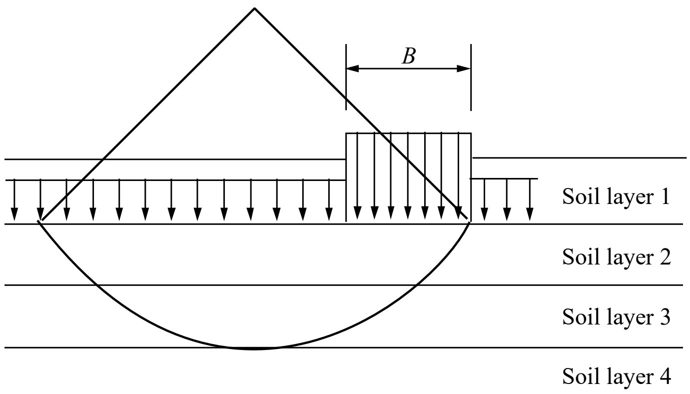


Fig. 3.2.5 Calculation of Bearing Capacity of Multi-layered Ground by Circular Slip Failure Analysis

(2) If the cohesive soil layer thickness H is significantly less than the smallest width of the foundation B (i.e., $H < 0.5B$), a punching shear failure, in which the cohesive soil layer below the surcharge plane is squeezed out, is liable to occur. The bearing capacity used for design against this kind of squeezed-out failure can be calculated by the following equation [4].

$$q_d = \frac{1}{m_B}(4.0 + 0.5B/H)c_{uk} + \rho_{2k}gD \tag{3.2.6}$$

where

q_d : design value of bearing capacity considering the buoyancy of the submerged part (kN/m²)

B : smallest width of foundation (m)

H : cohesive soil thickness (m)

c_{uk} : characteristic value of mean undrained shear strength in layer of thickness H (kN/m²)

$\rho_{2k}g$: characteristic value of unit volume weight of soil of ground above the level of the foundation bottom or unit volume weight in water, if submerged (kN/m^3)

m_B : adjustment factor for bearing capacity (refer to **Part III, Chapter 2, 3.2.3 Bearing Capacity of Foundation on Clayey Ground**)

D : embedment length of foundation (m)

3.2.5 Bearing Capacity for Eccentric and Inclined Actions

(1) Examination of the bearing capacity for eccentric and inclined actions acting on the foundation ground of gravity-type structures can be performed by circular slip failure analysis with the simplified Bishop method using the following equation. Partial factors γ_S and γ_R and adjustment factor m shall be appropriate values corresponding to the characteristics of the facilities. It is necessary to appropriately set the strength constant of the ground and others, the forms of the actions, and other factors considering the structural characteristics of the facilities, etc. m is the parameter corresponding to the safety factor considering designing with the traditional safety factor method since γ_S and γ_R are usually set to 1.00, as described later.

$$\frac{m}{\gamma_R}\cdot\frac{\gamma_S\cdot\sum\{(W_k+q_k)\sin\theta+aP_{Hk}/R\}}{\sum\left[\{c_k s+(W'_k+q_k)\tan\phi_k\}\dfrac{\sec\theta}{1+\tan\theta\tan\phi_k/(m/\gamma_R)}\right]}\leq 1 \qquad \textbf{(3.2.7)}$$

where

R : radius of slip circle in circular slip failure (m)

c_k : characteristic value of undrained shear strength in case of clayey ground, or characteristic value of apparent cohesion in drained condition in case of sandy ground (kN/m^2)

W'_k : characteristic value of effective weight of divided segment per unit length (weight of soil; effective weight in water if submerged) (kN/m)

q_k : characteristic value of vertical action from top of divided segment (kN/m)

θ : angle of bottom of divided segment to horizontal plane (°)

ϕ_k : 0 in case of clayey ground, or characteristic value of angle of shear resistance in drained condition (°) in case of sandy ground

W_k : characteristic value of total weight of divided segment per unit length (total weight of soil and water) (kN/m)

P_{Hk} : characteristic value of horizontal action on lumps of soil in slip circle in circular slip failure (kN/m)

a : arm length from the center of slip circle in circular slip failure at position of P_H action (m)

s : width of divided segment (m)

γ_S : partial factor to multiply to the action term

γ_R : partial factor to multiply to the resistance term

m : adjustment factor

The basic form of verification is expressed in the following equation:

$$m\cdot\frac{S_d}{R_d}\leq 1.0 \qquad S_d=\gamma_S S_k \qquad R_d=\gamma_R R_k \qquad \textbf{(3.2.8)}$$

where

S_k : characteristic value of the action term

S_d : value to be used for design of the action term

R_k : characteristic value of the resistance term

S_k : value to be expected in design of resistance term

Equation (3.2.7) described after this shall be the following:

$$m \cdot \frac{S_d}{R_d} \leq 1 \tag{3.2.9a}$$

$$S_d = \gamma_S S_k = \gamma_S \cdot \sum \{(W_k + q_k)\sin\theta + aP_{Hk}/R\} \tag{3.2.9b}$$

$$R_d = \gamma_R R_k = \gamma_R \sum \left[\{c_k s + (W'_k + q_k)\tan\phi_k\} \frac{\sec\theta}{1 + \tan\theta \tan\phi_k/(m/\gamma_R)} \right] \tag{3.2.9c}$$

In the strict sense, the above equation cannot be expressed in the form of **equation (3.2.8),** since R_d includes an adjustment factor m in the equation. Because it is also important to confirm the expected failure mechanism by determining an arc that gives the computationally minimum stability in the circular slip failure calculation using the simplified Bishop's method, the stability can be verified by calculating the minimum m with the iterative calculation and confirming that the value is greater than the value of the adjustment factor m given as a set value.

The concept of divided segment and its weight (W_k and W'_k) in the circular slip failure analysis is shown in **Fig. 3.2.6**, where divided segment also including water mass part without soil (including structures), i.e. water between the water surface and the ground surface, is considered. If the arc reaches the ground surface, but not the water surface, let the hydrostatic pressure act on the vertical surface of the divided segment of an edge.

Sliding moment is calculated based on the total weight (W_k) of soil mass and water mass. If the sliding moment of soil mass considering the total weight (W_k) and the sliding moment by water mass part and the hydrostatic pressure are added (actually subtracted), it coincides with the sliding moment of soil mass considering the effective weight (W'_k). This is because the moment attributable to the hydrostatic pressure that acts on a total system composed of soil mass and water mass generally coincides with the moment attributable to buoyancy. The hydrostatic pressure that acts along the arc is not necessary to calculate the moment because it points to the center of arc.

On the other hand, the shear strength to calculate the resistant moment is calculated from the effective overburden pressure based on the effective weight (W'_k) of soil mass.

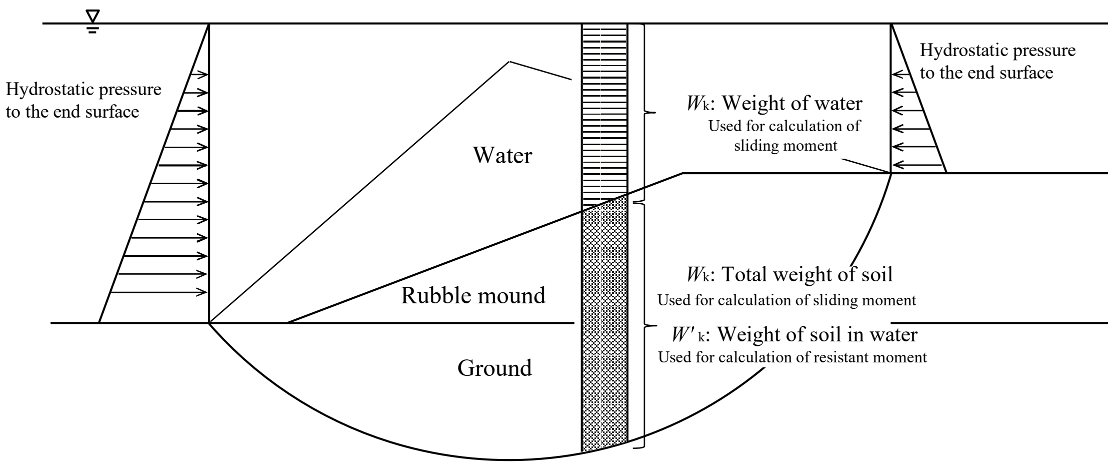


Fig. 3.2.6 Divided Segment in Circular Slip Failure Analysis and Concept of Its Weight

(2) In gravity-type quaywalls and gravity-type breakwaters, actions due to self-weight, earth pressure, wave force, ground motion and others shall be considered. The resultant forces of these actions are normally both eccentric and inclined. Therefore, examination for eccentric and inclined actions is necessary in examination of the bearing capacity of foundations. Here, eccentric and inclined action means an action with an inclination ratio equal to or greater than 0.1.

(3) Because normal gravity-type structures are two-layered structures with a rubble mound layer on foundation ground, an examination method which adequately reflects this feature is necessary. It has been confirmed that circular slip failure calculations by the simplified Bishop method can accurately express stability (safety factor) for bearing capacity in a series of research results, including laboratory model experiments, in-*situ* loading experiments, and analysis of the existing breakwaters and quaywalls; therefore, this method is used as a general method.[5)]

(4) Analysis of Bearing Capacity by Circular Slip Failure Calculation Based on the Simplified Bishop Method

Circular slip failure analysis based on the simplified Bishop method is considered more precise than the normal circular slip failure analysis based on the modified Fellenius method, except when a vertical action exerts on horizontal sandy ground (in this case, the bearing capacity formula shown in **Part III, Chapter 2, 3.2.2. Bearing Capacity of Foundation on Clayey Ground** is used rather than the circular slip failure). Therefore, the circular slip failure analysis by the simplified Bishop method is applied under the condition that eccentric and inclined forces act. As shown in **Fig. 3.2.7 (a)**, the start point of the sliding surface is set symmetrical to one of the foundation edges that is closer to the load acting point. In this case, the vertical action exerting on the wall bottom is converted into uniformly distributed load acting on the width between fore toe of the wall bottom and the start point of the sliding surface as indicated in **Figs 3.2.6 (b)** and **(c)**. The horizontal force shall act at the wall bottom. However, when calculating the bearing capacity during an earthquake action, actions on mound and the ground due to seismic force will not be considered.

When subgrade reaction has a trapezoidal distribution; $q = \frac{(p_1+p_2)}{4\,b'}B$

When subgrade reaction has a triangular distribution; $q = \frac{p_1 b}{4\,b'}$

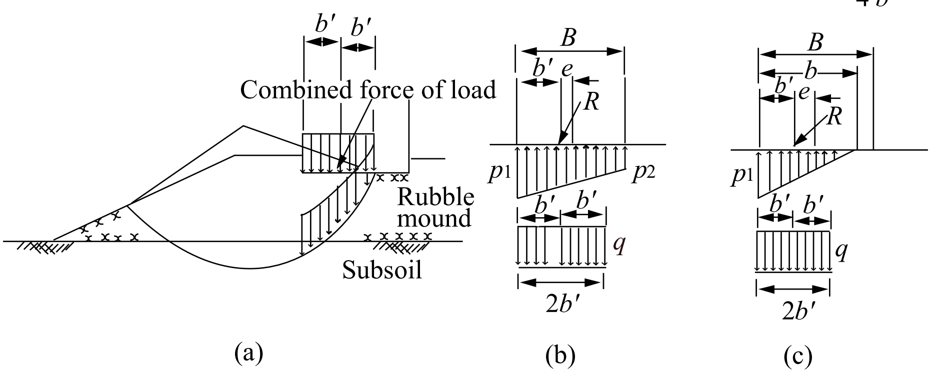


Fig. 3.2.7 Analysis of Bearing Capacity for Eccentric and Inclined Actions

(5) Verification and Partial Factors

① The verification of safety shall be performed by confirming that the ratio of the action moment by action and soil weight multiplied by the partial factor γ_S (S_d in **equation (3.2.9b)**) to the resistant moment by shear resistance multiplied by the partial factor γ_R (R_d in **equation (3.2.9c)**) multiplied by the adjustment factor m is 1.0 or less (**equation (3.2.9a)**). The values in **Table 3.2.2** may be used as general values for the partial factor.

② In normal soil structures, 1.00 is set to partial factors γ_S and γ_R and a value more than 1.00 is set to an adjustment factor m. For the performance verification of the permanent state of mooring facility subject to long-term actions, an adjustment factor value m equal to or more than 1.20 can generally be used.

③ Regarding actions on breakwaters due to ground motion, few examples of damage are available, and the degree of damage is also small. As the reasons for this, in many cases, large displacement does not occur because actions due to seismic ground motion are basically equal in the harbor direction and the outer sea direction and have the short duration. Accordingly, examination of the bearing capacity at the time of actions of seismic ground motion may be omitted in the case of ordinary breakwaters. However, detailed examination by dynamic analysis is desirable for breakwaters where stability at the time of actions of seismic ground motion may be a serious problem.

Table 3.2.2 Standard Lower Limit Values of Adjustment Factor *m* in Analysis Method for Bearing Capacity for Eccentric and Inclined Actions (Simplified Bishop Method)

	Quaywalls, etc.	Breakwaters
Permanent state	1.20 or more	□
Variable situation for Level 1 earthquake ground motion	1.00 or more	□
Variable situation for waves	□	1.00 or more

Note) In case partial factors are indicated by structural type, the partial factor for the part concerned shall be used.
The partial factor γ_S to be multiplied to the action term shall be 1.00 and the partial factor γ_R to be multiplied to the resistance term shall be 1.00.
The standard value of the adjustment factor *m* shall be set as the lower limit of the minimum value *m* obtained by the simplified Bishop's method.

(6) Strength Constants for Mound Materials and Foundation Ground

① Mound materials

Model and field experiments on bearing capacity subject to eccentric and inclined actions have verified that accurate results can be obtained by conducting circular slip failure analyses based on the simplified Bishop method, applying the strength constants obtained by triaxial compression tests [5)]. Large-scale triaxial compression test results of crushed stone have confirmed that the strength constants of large diameter particles are approximately equal to those obtained from similar grained materials with the same uniformity coefficient [6)]. Therefore, triaxial compression tests using samples with similar grained materials are preferably conducted in order to estimate the strength constants of rubbles accurately. If the strength tests are not conducted, the values of cohesion c_D = 20 kN/m^2 and shearing resistance angle ϕ_D = 35º are applied as the characteristic values as the standard strength constants for normal rubbles generally used. The strength of rubbles is expected to differ corresponding to the packing density of actual rubbles in the field, but as it is quite difficult to understand the situation of rubbles in the field, values of standard strength constants may be used.

The standard values have been determined as slightly safe side values based on the results of large-scale triaxial compression tests of crushed stones. The values are also appropriate from the analysis results of the existing breakwaters and mooring facilities. Cohesion c_D = 20 kN/ m^2 as a strength constant is the apparent cohesion to take account of variations of the shear resistance angle ϕ_D of crushed stones under variable confining pressures (tendency of decrease in shear resistance angle due to increase in confining pressure). **Fig. 3.2.8** is the summary of ϕ_D obtained assuming c_D = 0 as the results of triaxial tests on various types of crushed stones [5)]. It shows that as the confining pressure increases, ϕ_D decreases due to particle crushing. The values indicated by the solid line in the figure represents the calculated values under the assumption that the apparent cohesion is c_D = 20 kN/m^2 and ϕ_D= 35º. Here, the dependency of ϕ_D on the confining pressure is reflected by taking the apparent cohesion into account. According to the result of investigation of the relation between unconfined compressive strength in the mother rock and the strength constant, these standard values can be applied only to the stone material with an unconfined compressive strength in the mother rock of 30 MN/m^2 or more. If weak stone material with the strength of the mother rock of 30 MN/m^2 or less are used as a part of the mound, the strength constants will be around c_D = 20 kN/m^2 and ϕ_D = 30º [7)].

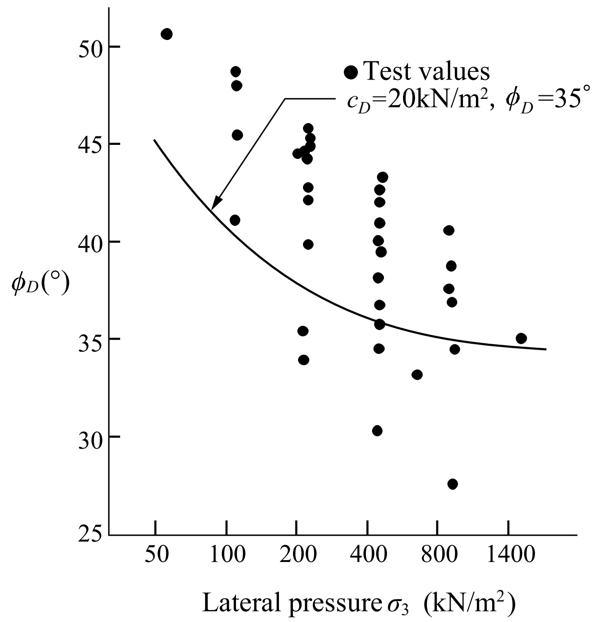


Fig. 3.2.8 Relationship between ϕ_D and Lateral Confining Pressure σ_3 and Apparent Cohesion

② **Foundation Ground**

As foundations subject to eccentric and inclined actions often have shallow sliding surfaces, the strength in the vicinity of surface of the foundation ground becomes a problem. In case of sandy ground, the strength constant ϕ_D is usually estimated from SPT-N value, but the estimation formulas until now did not consider the influence of the effective surcharge pressure in-situ and thus ϕ_D obtained from the SPT-N value in shallow sandy grounds tended to be underestimated.

Fig. 3.2.9 shows the compiled result of triaxial compression tests on undisturbed sand in Japan compared to the formulas proposed in the past. Even with the SPT-N values equal to or less than 10, around 40º has been obtained as a ϕ_D value. The following values are generally used as characteristic values for ϕ_D in foundation ground considering results of inverse analysis of past damage examples and that the bearing capacity for eccentric and inclined actions becomes a problem in performance verification against not static actions in permanent state but dynamic actions such as wave and ground motio.

Sandy ground with SPT-N value of less than 10: ϕ_D = 40º

Sandy ground with SPT-N value of 10 or more: ϕ_D = 45º

In case of clayey ground, the strength may be set in the method indicated in **Part II, Chapter 3, 2.3.3 Shear Characteristics**.

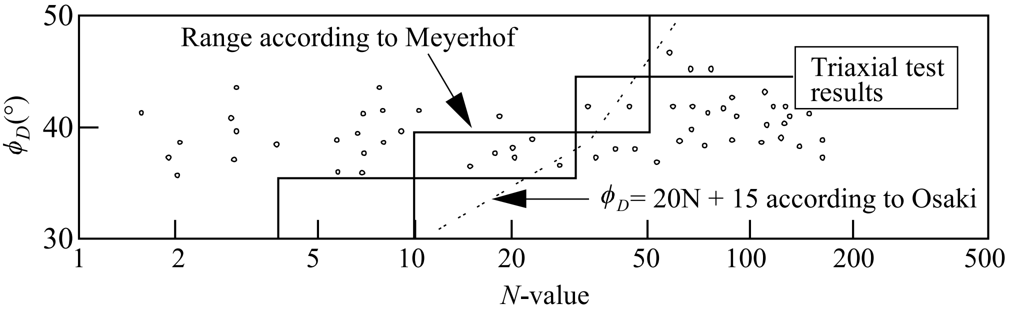


Fig.3.2.9 Relationship between ϕ_D and the SPT-N value Obtained by Triaxial Tests on Undisturbed Sand Samples

3.3 Deep Foundations

3.3.1 General

(1) Deep foundations transmit and support the load acting from the superstructure and so on to the strong soil strata in deep locations in the ground. The foundation is generally verified as deep one when its embedment depth is larger than its minimum width.

(2) The type of deep foundations includes caisson foundation, steel pipe sheet pile foundation, consecutive underground wall foundation and pile foundation. Here, foundation types except pile foundation are considered as deep foundations and the verification method for them is described. The verification method for pile foundations is described in **Part III, Chapter 2, 3.4 Pile Foundations**.

(3) Methods to distinguish the deep foundations from pile foundations include the one by judging that βL (β: characteristic value of pile, L: embedment length of pile) calculated by Chang's method (see **Part III, Chapter 2, 3.4.7 Calculation of Deflection of Piles by Chang's Method)** is 1 or less. However, βL may exceed 1 when applying a deep foundation to the foundation of large structures or on other cases.

3.3.2 Fundamentals of Performance Verification

(1) The performance of a deep foundation shall be properly verified taking into account the soil conditions, the structural property, the method of construction, etc.

(2) For the performance verification of caisson foundation, steel pipe sheet pile foundation and consecutive underground wall foundation, **Standard Specifications for Road Bridges and Their Manual, IV Substructures** [8)] may be referred to.

(3) Deep foundations used for relatively small structures and so on may be verified in the method described in **Part III, Chapter 2, 3.3.3 Performance Verification**.

3.3.3 Performance Verification

(1) The bearing capacity of deep foundations shall be verified by examination of the subgrade reaction generated by the action of vertical force and horizontal force to deep foundations. The subgrade reaction is determined by assuming that the surrounding ground is an elastic body and has modulus of subgrade reaction proportional to the depth from the ground surface as shown in **equation (3.3.1)** [9)].

$$p = Kxy \tag{3.3.1}$$

where

p : subgrade reaction (kN/m^2)

K : rate of increase in modulus of subgrade reaction with depth (kN/m^4)

x : depth (m)

y : displacement at depth x (m)

Subgrade reaction is composed of the vertical subgrade reaction acting on the foundation bottom and the horizontal subgrade reaction acting on the foundation side. K is generally considered to have different values for the horizontal subgrade reaction and the vertical subgrade reaction because the soil of deep foundation differs in side and bottom.

(2) Deep foundations are assumed to be rigid bodies and rotate by the action of horizontal force. Then, the horizontal subgrade reaction shows a parabolic distribution taking a value 0 at the ground surface. On the other hand, the distribution profile of vertical subgrade reaction differs if the action position of the resultant force of loads at the bottom slab of deep foundation is inside the core or not. The distribution of the subgrade reaction is assumed to be trapezoidal when the action position of the resultant force of loads is inside the core, while it is assumed to be rectangular when outside of the core. The distribution profile of the subgrade reaction is shown in **Fig. 3.3.1** and **Fig. 3.3.2**.

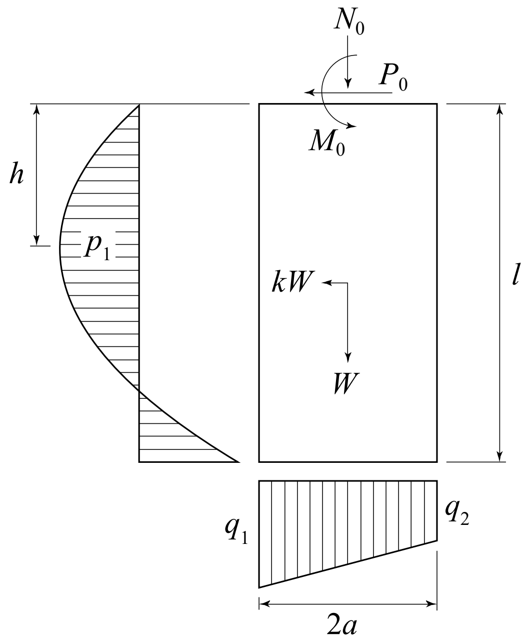


Fig. 3.3.1 Distribution of the Subgrade Reaction When the Action Position of the Resultant Force of Loads Is Inside the Core

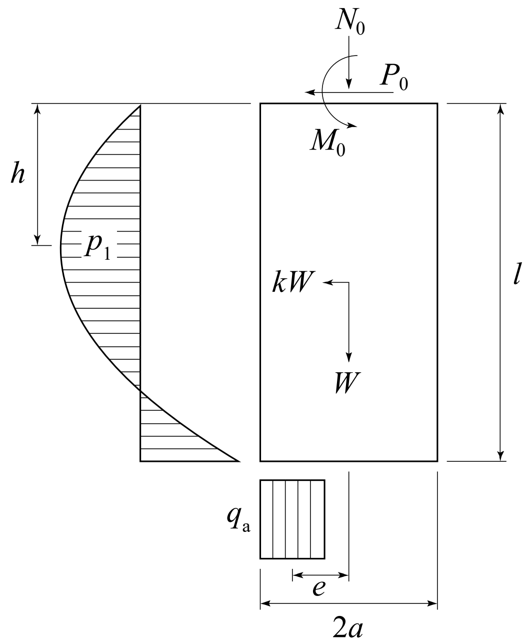


Fig. 3.3.2 Distribution of the Subgrade Reaction When the Action Position of the Resultant Force of Loads Is Outside of the Core

(3) When the action position is within the range of 1/6 of the basic width (width in the direction parallel with the horizontal force) from the center line of the foundation, the action position of the resultant force of loads at the bottom slab of deep foundation is called to be inside the core. At this time, the whole foundation bottom behaves as if it is pressed to the ground and the vertical subgrade reaction acts to the whole bottom surface. This is the reason why the trapezoidal distribution of the subgrade reaction as shown in **Fig. 3.3.1** is assumed.

On the other hand, if the action position of the resultant force of loads is outside of the core, one side of the foundation bottom behaves as if it floats and the vertical subgrade reaction acts only to the limited range of the foundation bottom. At this time, the vertical subgrade reaction acting on the foundation bottom shows triangular distribution, but assuming such distribution profile makes the calculation of subgrade reaction complex. Therefore, a method assuming a rectangular vertical subgrade reaction distribution [10] as shown in **Fig. 3.3.2** is used here as a simple method.

If **equation (3.3.2)** is true, the action position of the resultant force of loads is judged to be inside the core.

$$\frac{N_0 + w_1 l}{A} \geq \frac{3aK'\left(kw_1 l^2 + 4P_0 l + 6M_0\right)}{b\left(l^3 + 24\alpha K' a^3\right)} \tag{3.3.2}$$

where

N_0 : vertical force acting at ground level position (kN)

P_0 : horizontal force acting on structure above ground surface (kN)

M_0 : moment due to P_0 at ground surface (kN・m)

w_1 : self-weight of foundation per unit depth (kN/m)

l : embedment depth (m)

A : bottom area (m^2)

$2a$: width of a foundation parallel to horizontal force (m)

$2b$: width of a foundation perpendicular to horizontal force (m)

K' : $K'=K_2/K_1$

K_1 : rate of increase in coefficient of vertical subgrade reaction in the depth direction (kN/m^4)

K_2 : rate of increase in coefficient of horizontal subgrade reaction in the depth direction (kN/m^4)

k : horizontal seismic coefficient

α : constant determined by bottom shape
(1.00 for rectangular shape and 0.588 for round shape)

(4) When the action position of the resultant force of loads at the bottom slab is inside the core, the maximum vertical subgrade reaction acting on the foundation bottom, the maximum horizontal subgrade reaction acting on the foundation side and the depth where horizontal subgrade reaction is maximum can be obtained from **equations (3.3.3)** – **(3.3.5)**, respectively.

$$q_1 = \frac{N_0 + w_1 l}{A} + \frac{3aK'\left(kw_1 l^2 + 4P_0 l + 6M_0\right)}{b\left(l^3 + 24\alpha K' a^3\right)} \tag{3.3.3}$$

$$p_1 = \frac{3\left\{kw_1 l^4 + 3P_0 l^3 + 4M_0 l^2 + 8\alpha K' a^3\left(kw_1 l + P_0\right)\right\}^2}{4bl^3\left(l^3 + 24\alpha K' a^3\right)\left(kw_1 l^2 + 4P_0 l + 6M_0\right)} \tag{3.3.4}$$

$$h = \frac{kw_1 l^4 + 3P_0 l^3 + 4M_0 l^2 + 8\alpha K' a^3\left(kw_1 l + P_0\right)}{2l\left(kw_1 l^2 + 4P_0 l + 6M_0\right)} \tag{3.3.5}$$

where

q_1 : maximum vertical subgrade reaction acting on the foundation bottom (kN/m^2)

p_1 : maximum horizontal subgrade reaction acting on the foundation side (kN/m^2)

h : depth where horizontal subgrade reaction is maximum (m)

(5) When the action position of the resultant force of loads at the bottom slab is outside of the core, the vertical subgrade reaction acting on the foundation bottom is calculated assuming that it equals to the design value of the

vertical bearing capacity of deep foundation (see **Part III, Chapter 2, 3.3.4 Vertical Bearing Capacity of Deep Foundations**). The maximum horizontal subgrade reaction acting on the foundation side and the depth where horizontal subgrade reaction is maximum for rectangular foundation bottom can be obtained from **equations (3.3.6)** and **(3.3.7)**, respectively.

$$p_1 = \frac{3\left(kWl + 4M_0 - 4N_0 e - 4We + 3P_0 l\right)^2}{4bl^2\left(kWl + 6M_0 - 6N_0 e - 6We + 4P_0 l\right)} \quad \textbf{(3.3.6)}$$

$$h = \frac{l\left(kWl + 4M_0 - 4N_0 e - 4We + 3P_0 l\right)}{2\left(kWl + 6M_0 - 6N_0 e - 6We + 4P_0 l\right)} \quad \textbf{(3.3.7)}$$

where

W : self-weight of foundation (kN)

e : eccentricity (m)

$$e = a - \frac{W + N_0}{4bq_{ad}}$$

q_{ad} : design value of the vertical bearing capacity of deep foundation (kN/m^2)

When the shape of a foundation bottom is circular, a method to obtain the eccentricity by converting it to a rectangular bottom using **equations (3.3.8)** and **(3.3.9)** has been proposed [11)].

$$2a = \frac{3}{4}D \quad \textbf{(3.3.8)}$$

$$2b = \frac{\pi}{3}D \quad \textbf{(3.3.9)}$$

where

D : diameter of the foundation bottom (m)

Adaptability of **equations (3.3.8)** and **(3.3.9)** needs to be carefully examined since they are approximate conversion equations induced to equalize the area of the foundation bottom and the section modulus.

(6) Stability of the foundation is verified with **equations (3.3.10)** and **(3.3.11)** when the action position of the resultant force of loads at the bottom slab of a deep foundation is inside the core, and with **equation (3.3.10)** when outside of the core.

$$m\frac{\gamma_S p_1}{\gamma_R p_{pk}} \leq 1.0 \quad \textbf{(3.3.10)}$$

$$q_{ad} \geq q_1 \quad \textbf{(3.3.11)}$$

where

p_{pk} : characteristic value of passive earth pressure at depth h (kN/m^2)

q_{ad} : design value of vertical bearing capacity of deep foundations (kN/m^2)
(see **3.3.4 Vertical Bearing Capacity of Deep Foundations in this Chapter**)

γ_S : partial factor to multiply to action

γ_R : partial factor to multiply to resistance

m : adjustment factor

Partial factors γ_S and γ_R to multiply to action and resistance shall be 1.0. Adjustment factor m for important facilities shall be 1.5 or more, otherwise 1.1 or more. However, adoption of more sophisticated verification method should be examined than using **equations (3.3.10)** and **(3.3.11)** when verifying important facilities (see **Part III, Chapter 2, 3.3.2 Fundamentals of Performance Verification**).

3.3.4 Vertical Bearing Capacity of Deep Foundations

(1) The design value of vertical bearing capacity of a deep foundation can be calculated as the sum of bearing capacity at foundation bottom and friction resistance force at foundation sides as shown in **equation (3.3.12)**.

$$q_{ad} = q_{u1d} + q_{u2d} \tag{3.3.12}$$

where

q_{ad} : design value of vertical bearing capacity of deep foundation (kN/m^2)

q_{u1d} : design value of bearing capacity at foundation bottom (kN/m^2)

q_{u2d} : design value of friction resistance force at foundation sides (kN/m^2)

However, if the surrounding ground may become loose due to construction of deep foundations, the bearing capacity at foundation bottom is considered to be the vertical bearing capacity of deep foundations, ignoring the friction resistance force at foundation sides.

(2) The design value of bearing capacity at foundation bottom can be considered to be equal to that of bearing capacity of shallow foundation. For the design value of bearing capacity of shallow foundations, **Part III, Chapter 2, 3.2.2 Bearing Capacity of Foundations on Sandy Ground** or **Part III, Chapter 2, 3.2.3 Bearing Capacity of Foundation on Clayey Ground** can be referred to according to the nature of the soil of foundation bottom.

(3) The design value of friction resistance at foundation sides in sandy ground can be calculated by **equation (3.3.13)**.

$$q_{u2d} = \frac{1}{m_B}\left(1+\frac{B}{L}\right)\frac{D^2}{B}K_{ak}\gamma_{2k}\mu_k \tag{3.3.13}$$

where

B : minimum width of foundation (width in the direction of narrow side) (m)

L : maximum width of foundation (width in the direction of wide side) (m)

D : embedment depth of foundation (m)

K_{ak} : characteristic value of coefficient of active earth pressure (δ=0°)
(see **Part II, Chapter 4, 2 Earth Pressure**)

γ_{2k} : characteristic value of unit volume weight of soil of ground above the foundation bottom, or submerged unit weight if submerged (kN/m^3)

μ_k : characteristic value of friction coefficient between foundation sides and sandy soil

$$\mu_k = \tan\frac{2}{3}\phi_k$$

ϕ_k : characteristic value of shear resistance angle of sandy soil (°)

m_B : adjustment factor for bearing capacity of foundation

The friction angle between the foundation sides and sandy soil does not exceed the shear resistance angle of soil. **Equation (3.3.13)** determines the friction coefficient between the foundation sides and sandy soil assuming that the foundation sides are made of concrete and the friction angle between concrete and sandy soil is (2/3) ϕ_k.

The value of adjustment factor m_B for bearing capacity of foundation is determined according to **Part III, Chapter 2, 3.2.2 Bearing Capacity of Foundations on Sandy Ground**.

(4) The design value of friction resistance force at foundation sides in clayey ground can be calculated by **equation (3.3.14)**.

$$q_{u2d} = \frac{2}{m_B}\left(1+\frac{B}{L}\right)\frac{D_c}{B}\overline{c}_{ak} \qquad \textbf{(3.3.14)}$$

where

D_c : embedment depth of foundation below groundwater level (m)

$\overline{c}_{ak}$: characteristic value of mean adhesion in embedment depth of foundation below groundwater level (kN/m^2)

m_B : adjustment factor for bearing capacity of foundation

The soil above the groundwater level of clayey ground has a possibility of drying shrinkage during summer; therefore, the friction resistance force of foundation sides in this portion is not to be expected. **Equation (3.3.14)** determines the characteristic value of friction resistance force of foundation sides using the area and mean adhesion of foundation sides below groundwater level.

For practical adhesion in clayey ground, see **Table 3.3.1**. Friction resistance force on foundation sides cannot be expected if the ground surrounding the foundation is soft sandy soil.

The value of adjustment factor m_B for bearing capacity of foundation is determined according to **Part III, Chapter 2, 3.2.3 Bearing Capacity of Foundations on Clayey Ground**.

(5) The negative skin friction acting on the foundation sides shall be examined if deep foundations penetrate the ground generating consolidation and reach the bearing layer. For the method of examination, see **Part III, Chapter 2, 3.4.11 Negative Skin Friction Force**.

Table 3.3.1 Relationship between Unconfined Compression Strength and Mean Adhesion of Clayey Soil

	Unconfined Compression (kN/m^2)	Mean Adhesion (kN/m^2)
Soft clayey soil	20 - 50	-
Medium clayey soil	50 - 100	6 - 12
Hard clayey soil	100 - 200	12 - 25
Extremely hard clayey soil	200 - 400	25 - 30
Consolidated clayey soil	400 or more	30 or more

3.4 Pile Foundations

3.4.1 General

(1) Pile foundation means a type of foundation which transfers actions on the facilities to the ground by means of a single pile or multiple piles.

(2) Pile means a columnar structure which is provided underground in order to transfer actions on the superstructures or the foundation to the ground. Piles are classified into steel pile, concrete pile, wooden pile, and so on by their material, and into driven pile, bored pile, cast-in-place pile, and so on by their construction method.

Piles are used as a single pile or as coupled piles in port facilities. Single piles are individually used straight pile (built in vertical direction) or batter pile (built with a certain angle of inclination to the vertical line). Coupled piles connect two batter piles having a different angle of inclination to the vertical line at the head and used as an integral structure.

(3) A pile whose bottom is embedded in so-called bearing stratum such as dense sandy ground, gravel ground, and rock ground are called bearing piles. On the other hand, piles whose bottom is not embedded in stratum considered to be supportive but remains in a relatively soft stratum are called friction piles.

In the past, piles were classified into bearing piles where the base resistance is dominant and friction piles where the skin friction force is dominant by focusing on the ratio of base resistance to skin friction force in the pushing resistance force of a pile in its axial direction. On the other hand, some people noted that the classification of bearing and friction piles is not absolute because the ratio of base resistance to skin friction force in the pushing resistance force in its axial direction varies by the amount of load, loading rate, loading time, etc. Here, a definition capable of distinguishing the bearing pile and the friction pile regardless of load conditions is adopted in order to avoid confusion due to such circumstances.

3.4.2 Fundamentals of Performance Verification of Pile Foundations

(1) The performance of pile foundations is verified from the viewpoint of bearing capacity of pile foundations, displacement, stress caused in pile body, etc. The verification procedure is often complex and needs trial and error because the bearing capacity of pile foundations, displacement, and stress caused in pile body are interrelated. High degree of freedom in selection of the shape of pile foundation and setting of the pile arrangement necessitates adequate consideration of economy in their examination. **Fig. 3.4.1** shows an example of performance verification sequence of pile foundations.

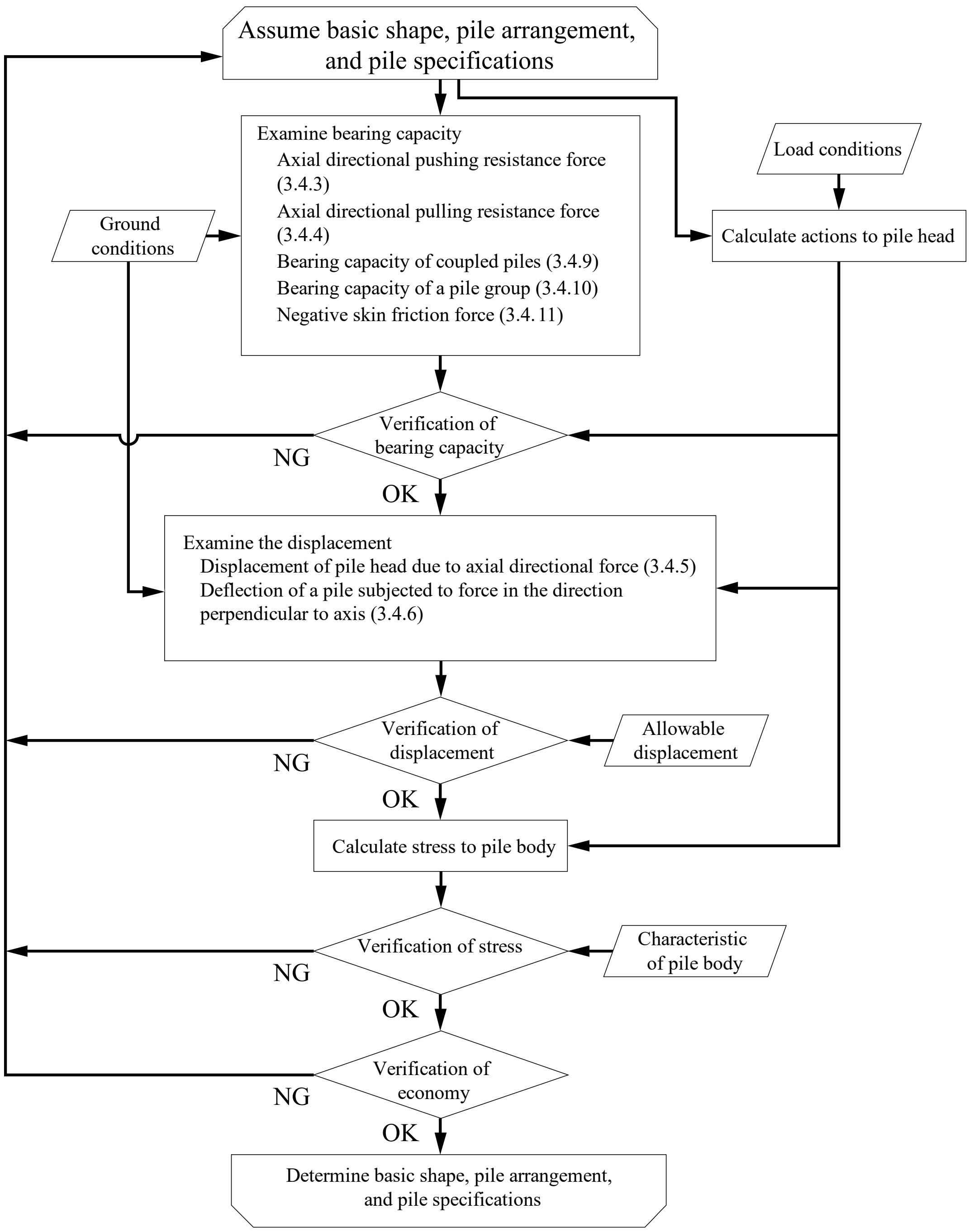


Fig. 3.4.1 Example of Performance Verification Sequence of a Pile Foundation

(2) The vertical load acting on the pile foundation shall be supported only with piles. The bearing capacity of floor slab in superstructure or ground contacting the bottom surface and others of footing of pile foundations shall not be expected.

As time goes by, the floor slab of superstructure or the bottom of footing contacted with the ground at the end of construction may be separated due to difference in the amount of settlement between the pile foundation and the ground. Friction piles are relatively less prone to cause such difference, but when the ground turbulence occurred at the time of pile construction is recovered after construction, the consolidation phenomenon accompanies, and the ground relatively and slightly settles. Thus, for the sake of safety, the bearing capacity of floor slab in superstructure or ground contacting the bottom surface of footing of pile foundations shall not be expected.

(3) The horizontal load acting on the pile foundation shall be supported only with piles. The passive earth pressure resistance of the ground in the front of the superstructure or pile foundation embedment shall not be expected.

If the passive earth pressure resistance of the ground in the front of structures or pile foundation embedment can be properly evaluated, the resistance may be considered. Then, it is necessary to confirm that both of the passive earth pressure resistance and the resistance force perpendicular to the axis of pile take reasonable values against the displacement of foundation. That is, care should be taken so that increase in displacement of the pile head part before the passive earth pressure resistance reaches an expected value does not cause flexure fracture of piles. The evaluation of the passive earth pressure resistance considering such influence is generally difficult, and it often requires sophisticated examination.

When single piles and coupled piles are used together in a pile foundation, horizontal force is considered to be totally borne by coupled piles in general. This takes into consideration that coupled piles have a structure less prone to cause horizontal displacement than single piles and that they exert significantly large bearing capacity to the same horizontal displacement than single piles.

(4) When verifying the bearing capacity of a pile foundation, it is necessary to examine whether each pile has enough bearing capacity for load acting on the head of the pile composing a pile foundation. Coupled analysis of superstructure and pile foundations may be needed depending on the kind of facilities supported by the pile foundations or the kind of load that acts.

(5) The bearing capacity of a pile used as a single pile is examined in each direction by breaking down the load acting on the pile head into the element in the pile axis direction and the element in the direction perpendicular to the pile axis.

If pulling force in the axial direction acts on a pile, particularly careful examination is needed since the pulling failure may fatally damage the facilities the foundation bears. The pulling force in the axial direction acting on a pile behaves as if it raises the ground. In other words, the effective stress in the ground surrounding the pile decrease, and the ground tends to lose. The contact area of pile and the ground decreases as upward displacement of piles by pulling increases. Therefore, increase in upward displacement of piles by pulling and continuation of loading time are disadvantageous elements to pulling resistance force of a pile in its axial direction. In clayey ground, in particular, it is anticipated that creep phenomenon occurs more prominently than in the case of pushing. As seen above, as the behavior to displace piles upwardly by pulling promotes instability of the structure as a whole, it is desirable to minimize the pulling force in the axial direction acting on piles to the extent possible. In particular, when there exists a pile that is subjected to large pulling force in the axial direction for a long time, examine rearrangement of piles. Moreover, when the pile head and superstructure are insufficiently connected, verify that the pile and superstructure are surely connected if the pulling force in the axial direction acts on the pile since the pulling resistance force of a pile in its axial direction is not exerted.

Care should be taken as the pile head is greatly displaced when force in the direction perpendicular to the axis acts on piles in soft clayey ground. There are many issues, such as the necessity of considering the influence of the ground consolidation or creep phenomenon. Thus, it should be avoided to design expecting resistance force in the direction perpendicular to the axis of piles in soft clayey ground.

(6) The bearing capacity of a single pile in its axial direction shall be verified by comparing the force acting on the pile head in the axial direction and the resistance force of the pile in its axial direction. Also, confirm that the pile body does not fail by the axial stress caused in the pile body. It shall also be confirmed that the settlement and upward displacement of the pile head does not exceed the value determined from the allowable displacement of the facility the foundation bears.

For calculation of resistance force of a pile in its axial direction, refer to **Part III, Chapter 2, 3.4.3 Pushing Resistance Force of a Pile in Its Axial Direction** and **Part III, Chapter 2, 3.4.4 Pulling Resistance Force of a Pile in Its Axial Direction**.

For resistance of a pile body used in verification of its failures, refer to **Part II, Chapter 11, 2.2 Characteristic Values of Steel Members** and **Part II, Chapter 11, 3.6 Concrete Pile Materials**. When using a spliced pile, the minimum value of resistance of each part and joint shall be the resistance of a pile body. The effective cross-sectional area of a pile needs to be properly evaluated when verifying failures of a pile body. In general, the minimum cross-sectional area shall be used for a concrete pile or a wooden pile, whereas the cross-sectional area considering the influence of corrosion shall be used for a steel pile. Safety margin depending on the structural type shall be properly considered in verification. When force in the axial direction and the direction perpendicular to the axis acts to a single pile at the same time, the failure of a pile body shall be verified in the condition where axial stress and bending stress calculated from the examination for each direction are overlaid.

For the examination of settlement and upward displacement of piles, refer to **Part III, Chapter 2, 3.4.5 Displacement of Pile Head due to Axial Directional Forcer**.

(7) The bearing capacity in the direction perpendicular to the axis of a single pile shall be verified by calculating the deflection of a pile when the force in the direction perpendicular to the axis acts on the pile head and confirming that the bending stress caused in the pile body does not fail the pile body and the displacement and the angle of inclination of a pile head do not exceed the value determined from the allowable displacement of facilities that the foundation bears.

For calculation of deflection of the pile on which the force in the direction perpendicular to the axis acts, refer to **Part III, Chapter 2, 3.4.6 Deflection of a Pile Subjected to Force in the Direction Perpendicular to the Axis**. The verification of failure of a pile body shall be performed similarly to the bearing capacity of a pile in its axial direction (see **(6)**).

(8) For the verification of bearing capacity of coupled piles, see **Part III, Chapter 2, 3.4.9 Bearing Capacity of Coupled Piles**.

(9) The influence of behavior as a pile group or negative skin friction force shall be considered when verifying bearing capacity of a pile. For behavior as a pile group and negative skin friction force, see **Part III, Chapter 2, 3.4.10 Bearing Capacity of a Pile Group** and **Part III, Chapter 2, 3.4.11 Negative Skin Friction Force**, respectively.

(10) For structural types such as piled-raft foundation [12] and soft-landing-moundless structure with piles (see **Part III, Chapter 4, 3.9 Breakwater Sitting on Soft Ground**) which control the settlement of facilities by utilizing piles as friction piles, it shall be reasonable to add the bearing capacity of the grounds at the floor slab of structures. Performance verification considering the behavioral characteristic of structures in full is needed for such structural types.

(11) When constructing piles at the final waste disposal site built on the sea level, an appropriate construction method shall be selected according to the property of waste ground at the construction position. Care should be taken not to drive the waste together with the pile bottom at that time. In case piles penetrate the bottom water sealing stratum of the disposal site, it is necessary to ensure sufficient water sealing performance after penetration of piles in construction design. A report on an example of pile field trial at an actual disposal site [13] may be referred to.

3.4.3 Pushing Resistance Force of a Pile in Its Axial Direction

(1) The characteristic value of pushing resistance force of a pile in its axial direction is generally determined in reference to the pushing resistance force of a pile in its axial direction exerted when the ground failure condition is reached by the action of pushing force in the axial direction on the pile head. Normally, the second-limit-resistance force of piles (see **Reference (Part II), Chapter 1, 3.10.5 Pile Load Test**) shall be the characteristic value. Depending on the kind or purpose of structures the pile foundation bears, the first-limit-resistance force (see **Reference (Part II), Chapter 1, 3.10.5 Pile Load Test**) where the pushing resistance force of a pile in its axial direction becomes a yield situation may be the characteristic value.

(2) As shown in **equation (3.4.1)**, the characteristic value of pushing resistance force of a pile in its axial direction is expressed as the sum of pile's characteristic values of base resistance and skin friction force.

$$R_{tk} = R_{pk} + R_{fk} \tag{3.4.1}$$

where

R_{tk} : characteristic value of the pushing resistance force of a pile in its axial direction (kN)

R_{pk} : characteristic value of the base resistance force of a pile (kN)

R_{fk} : characteristic value of the skin friction force of a pile (kN)

In the case of an open-ended pile, which is the pile with its bottom opened such as a steel pipe pile, the pushing resistance force of a pile in its axial direction may be considered as the sum of three, namely the base resistance force of a pile tip, the skin friction force acting on the inner surface of a pile, and the skin friction force acting on the outer surface of a pile. However, since little is known as to the base resistance force of a pile tip and the skin friction force acting on the inner surface of a pile, **equation (3.4.1)** is used, and the method to separately consider the plugging ratio (see **(8)**) is actually adopted even for an open-ended pile.

As shown in **equation (3.4.2)**, the characteristic value of the skin friction force of a pile shall be considered to be determined by multiplying the skin friction force per unit contact area of the pile shaft and the ground by the perimeter surface area of the pile.

$$R_{fk} = \sum_i \overline{r_f}_{ki} A_{si} \quad \textbf{(3.4.2)}$$

where

$\overline{r}_{fki}$: mean skin friction force per unit contact area of pile and the ground in the *i*-th layer (kN/m^2)

A_{si} : contact area of pile and the ground in the *i*-th layer (m^2)
$A_{si} = U_{si} \cdot l_i$

U_{si} : perimeter length of pile cross section in the *i*-th layer (m)

l_i : length of pile in the *i*-th layer (m)

(3) The characteristic value of the pushing resistance force of a pile in its axial direction shall be determined by the loading test of the pile. The test pile used for the loading test shall have the same specifications as far as possible as the pile to be used for actual construction. If the specifications and ground conditions for test piles and actual piles coincide, the characteristic value of the pushing resistance force in the axial direction can be directly obtained from the result of the loading test. If the specifications or ground conditions of test piles and actual piles are different, individually obtain the characteristic values of the pile base resistance force and the skin friction force per unit contact area in each layer from the result of the loading test and calculate the characteristic value of the pushing resistance force in the axial direction with **equations (3.4.1)** and **(3.4.2)**. At this time, it is necessary to carefully determine each characteristic value by fully considering the influence of the difference in conditions between the test pile and the actual pile to the pushing resistance force of a pile in its axial direction. Note also that the influence of consolidation or creep of clayey ground cannot be confirmed because of the short duration time of load in the loading test of piles.

It is important to collect and verify the information concerning the pile construction method during the construction of a test pile together with the loading test. If the premised pile construction method is judged to be problematic, it is necessary to examine the way to resolve issues, change the construction method of piles as needed, verify the bearing capacity of the piles, etc. If different construction methods are adopted for the test pile and the actual pile, determine the characteristic value taking it into consideration as the bearing capacity of a pile is affected by the construction method. The same is true if different supplementary construction method is used for construction of piles.

For the loading test methods, see **Reference (Part II), Chapter 1, 3.10 Pile Load Test**.

(4) When verifying the bearing capacity of a pile, the design value determined by considering the safety margin based on the characteristic value of the pushing resistance force of a pile in its axial direction shall be used. The safety margin to allow for varies according to the kind or purpose of facilities the foundation bears.

The purpose of this safety margin is to take into consideration uncertainties contained in ground conditions, pile conditions, construction conditions, loading conditions, etc. Therefore, the size of safety margin should properly be set in accordance with the quality and quantity of given information, analysis method, etc. When a loading test of a pile is performed, the characteristic value of the pushing resistance force in the axial direction can generally be estimated more precisely than using an estimation formula. Thus, it has been proposed to change the foctor of safety by taking it into account [14)].

The safety margin was allowed for in the past by setting a safety factor of 2.5 or more for stationary load, 1.5 or more for load on bearing pile, and 2.0 or more for load on friction pile during earthquake. These values can be referred to as they are considered to give a safe side result in performance verification of port facilities in general. These safety factors have been determined corresponding to the pushing resistance force of a pile in its axial direction when the ground reaches the failure condition. Using a safety factor of 2.5 for stationary load is considered to ensure a safety factor on the order of 2.0 against a yield situation.

In the past, verification was performed with a safety factor of 3.0 or more by estimating the pushing resistance force of a pile in its axial direction using the calculation result with the dynamic bearing capacity management equation (see **Reference (Part II), Chapter 1, 3.10.10 Dynamic Bearing Capacity Management Equation**) such as Hiley's equation. However, this method must not be used except in special cases where information to confirm the validity of the calculation result is available and so on, because the dynamic bearing capacity management equation has no precision enough to estimate the characteristic value of the pushing resistance force of a pile in its axial direction.

(5) If preliminary loading test is difficult to perform, it is necessary to estimate the characteristic value of the pushing resistance force in the axial direction based on various estimation formulas. In this case, it is required to perform the loading test to confirm the bearing capacity of piles at the beginning of construction and check the validity of values used for performance verification. However, if the applicable range of used estimation formula and the error and dispersion of estimated values in that range is clearly indicated, the verification fully considering that error and dispersion can replace the loading test.

There are many estimation formulas for characteristic value of the pushing resistance force of a pile in its axial direction with different theoretical background and targeted pile construction methods. When using an estimation formula in verification, it is necessary to confirm that conditions of piles to be verified are included in the applicable range of each estimation formula.

When examining the applicability of an estimation formula, pay attention to the construction method, specifications of piles (diameter, length, shape of the bottom, etc.), soil of the bearing stratum, ground conditions (failure condition, yield situation, etc.) corresponding to estimated characteristic values, etc. Estimated formula shall be used after investigating references and confirming their prerequisites and basis, such as conditions for data acquisition, because even if the applicable range of an estimation formula is not specified, there may be implicitly assumed applicable ranges.

Even a slight difference in construction method or procedure changes the ground condition around the piles and influences the bearing capacity of piles. The same is true in the case where a supplementary construction method is used to improve constructability when constructing piles. High degree of judgment for the applicability of an estimation formula is required in this case. Estimated formulas must not be used beyond their applicable range.

Many estimation formulas of the pushing resistance force of a pile in its axial direction do not consider the self-weight of piles as pushing force in the axial direction, unlike in the case of loading tests. The influence of self-weight of piles is generally ignored when determining the characteristic value of the pushing resistance force of a pile in its axial direction, but if the pile is extremely heavy, it is necessary to subtract the self-weight of the pile (underwater weight if in water) from the characteristic value estimated with the estimation formula.

(6) The characteristic value of the base resistance force of piles constructed with the hummer driving method and having sandy ground as the bearing stratum can be estimated with **equation (3.4.3)**.

$$R_{pk} = 300 N A_p \qquad \textbf{(3.4.3)}$$

where

A_p : cross-sectional area at the pile bottom (m^2)
If pile's diameter is B,
$A_p = \pi B^2/4$

N : SPT-N value of the ground around the pile bottom

$$N = \frac{N_1 + \overline{N_2}}{2}$$

N_1 : SPT-N value of the ground at the pile bottom ($N_1 \leq 50$)

$\bar{N}_2$: mean SPT-N value in the range above the pile bottom to distance of $4B$ ($\bar{N}_2 \leq 50$)

B : diameter or width of a pile (m)

Equation (3.4.3) is a corrected equation by adding results of the test performed in Japan based on the equation [15] proposed with the results of pile loading tests in sandy ground performed in many foreign countries and others.

It should be noted that the characteristic value of the base resistance force estimated from **equation (3.4.3)** may be excessive if the bearing stratum is not good. It is necessary that these effects shall be considered if the depth of bearing stratum is insufficient and there is soft layer below the bearing stratum or the ground weakens in the direction of depth within the bearing stratum.

(7) The characteristic value of the base resistance force of piles constructed with the hummer driving method and the bottom of which is embedded in clayey ground can be estimated with **equation (3.4.4)**.

$$R_{pk} = 6 c_p A_p \tag{3.4.4}$$

where

c_p : undrained shear strength in the ground at the pile bottom position (kN/m^2)

Equation (3.4.4) is induced according to the bearing capacity of shallow foundation on clayey ground (see **Part III, Chapter 2, 3.2.3 Bearing Capacity of Foundation on Clayey Ground**). The **equation (3.4.4)** is induced since B/L = 1.0 and KB/c_p < 0.1 are generally true for piles, and thus the bearing capacity coefficient at the pile bottom is 6. The undrained shear strength obtained from the unconfined compression test is often used in **equation (3.4.4)**.

(8) The characteristic value of the base resistance force of a pile estimated from **equation (3.4.3)** or **(3.4.4)** assumes that the pile bottom is completely closed. The characteristic values estimated by these equations are overmuch for an open-ended pile, which is the pile with its bottom opened, such as a steel pipe pile.

The behavior of the bottom of open-ended piles is quite different from that of closed-ended piles. However, the behavior of the ground in the vicinity of the bottom of open-end piles and interaction between piles and ground are still in the process of research, and no versatile estimation formula on the base resistance force of open-ended piles have been devised yet. As such, in practical use, the base resistance force of closed-ended piles estimated from **equation (3.4.3)** or **(3.4.4)** is reduced by multiplying a coefficient called the plugging ratio, as shown in **equation (3.4.5)** and used as the estimated value of the base resistance of open-ended piles.

$$R_{pok} = \eta R_{pk} \tag{3.4.5}$$

where

R_{pok} : characteristic value of the base resistance force of open-end piles (kN)

η : plugging ratio

Plugging ratio is affected by different factors such as diameter of piles, embedment length of piles, characteristic of the ground, and construction methods. Although various methods to estimate the plugging ratio taking these factors into consideration are examined, currently, no standard method has been established. Therefore, when using open-ended piles, it is necessary to estimate the pushing resistance force of a pile in its axial direction by way of a loading test. The specifications of the pile bottom and construction conditions in the loading test must be as close as possible to actual piles.

Past performance shows that the plugging ratio may be considered to be 100% provided that the pile diameter is 60 cm or less for steel pipe piles and the short side is 40 cm or less for H-shaped steel piles. Large-diameter steel pipe piles frequently used in port facilities are reported to have significant effect of the pile diameter on the plugging ratio [16] and may be referred to, as shown in **Fig. 3.4.2**.

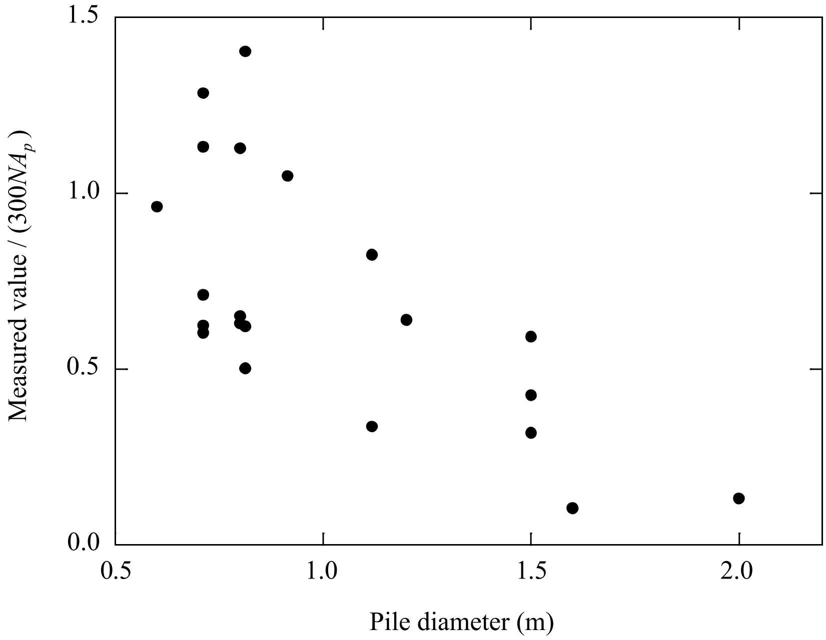


Fig. 3.4.2 Relation between Diameter of Open-ended Piles and Plugging Ratio (Kikuchi et al. [16], added and altered)

(9) Partition boards are sometimes provided to the pile bottom to increase the plugging ratio of large-diameter steel pipe piles. Some reports indicated that this method contributed to improve the plugging ratio [17] [18] [19], whereas others indicated that there was no clear effect [20]. The effect of partition boards is difficult to uniformly evaluate since it is affected by the shape of cross-section and length of installation of the partition board, diameter of piles, property of the ground around the pile bottom, etc. When improvement of plugging ratio with the partition board is anticipated in performance verification, it is required to confirm the effect with the loading test.

Note that providing partition boards narrows the range of options for countermeasures when the pushing resistance force of a pile in its axial direction is insufficient. Partition boards at the pile bottom may make it difficult to take countermeasures such as improvement of the ground around the pile bottom through the space inside of the piles.

(10) The characteristic value of the skin friction force per unit contact area of piles constructed by the hummer driving method in sandy ground can be estimated by **equation (3.4.6)**.

$$\overline{r_{f\,k_i}} = 2\,\overline{N} \qquad \textbf{(3.4.6)}$$

where

$\overline{N}$: mean SPT-N value in the i-th layer

(11) The characteristic value of the skin friction force per unit contact area of piles constructed by the hummer driving method in clayey ground can be estimated by **equation (3.4.7)**.

$$\overline{r_{f\,k_i}} = \overline{c_a} \qquad \textbf{(3.4.7)}$$

where

$\overline{c}_a$: mean adhesion of pile and the ground in the i-th layer (kN/m^2)

The mean adhesion of pile and the ground in **equation (3.4.7)** is often calculated by **equation (3.4.8)** from undrained shear strength of the ground.

$$\overline{c_a} = \begin{cases} c & c \leq 100 \\ 100 & c > 100 \end{cases} \tag{3.4.8}$$

where

c : undrained shear strength of the ground in the i-th layer (kN/m^2)

However, it has been suggested that obtaining the mean adhesion of piles and the ground from undrained shear strength of the ground is theoretically problematic [21]. The mean adhesion may be weaker than undrained shear strength in over consolidated ground or when the pile length is extremely long [22]. The value of mean adhesion should be determined by taking due care of property of the ground, pile conditions, etc.

(12) For the estimation formula for characteristic value of the pushing resistance force of a pile in its axial direction constructed in cast-in-place pile method, inside digging pile method, pre-boring pile method, steel pipe soil cement pile method, and other methods, see **Specifications for Highway Bridges, IV Substructures** [23]. Equations for extended bearing capacity theory of shallow foundations [24] and equations based on the cavity expansion theory [25] are also proposed and can be referred to for verification. If an estimation formula for the pushing resistance force in the axial direction corresponding to the new pile construction method developed in recent years is indicated, it can be used by fully confirming its applicable range.

Vibratory hammer has been used for the construction of actual piles in recent years. As the principle of pile driving by vibratory hammer is different from that of hummer driving using hydraulic or other hammers, various estimation formulas concerning piles constructed by hummer driving cannot be used. When using the vibratory hammer method, constructional devices, such as compacting the ground by using the hummer driving method in the vicinity of depth to stop driving and confirmation of pile's resistance force by loading test, are needed.

(13) The embedment length of the bearing pile in the bearing stratum shall be determined considering the soil of the bearing stratum, constructability of piles, etc. When driving piles by the hummer driving method, the embedment length into bearing stratum is often determined as about 1–3 times of the diameter of piles. Previously, examples of long embedment into the bearing stratum were seen in order to improve the plugging ratio of open-ended piles. However, the embedment length into bearing stratum should not be unreasonably long since the diameter of piles is considered to dominantly influence the plugging ratio of large-diameter piles than the embedment length into bearing stratum (see **(8)**).

If the unflat surface of bearing stratum is anticipated, or in other cases, device such as making allowance for embedment length into bearing stratum in advance is desirable to cope with insufficiently embedded piles during construction. If the bearing stratum is solid ground, such as rocks, it is necessary to carefully examine the construction method as well as the embedment length into bearing stratum, because there is concern for difficulties in pile driving into such a stratum.

If a bearing stratum is thin, and a weak stratum exists below it, or in other cases, it is necessary to set the embedment length into the bearing stratum taking into account punching failure and others of the bearing stratum. As little knowledge is available concerning the problem of thin stratum bearing of piles constructed by hummer driving method, the pushing resistance force of a pile in its axial direction in this case shall be confirmed by the loading test. For the concept of bearing capacity in the case of thin stratum bearing of piles constructed in other construction method, **Pile Design Handbook for Highway Bridge Foundation** [26] may be referred to.

(14) If the bearing stratum is composed of soft rock, hard clay, bedrock, and others, the pushing resistance force of a pile in its axial direction shall be confirmed by the loading test as little knowledge is available concerning the base resistance force of piles constructed by the hummer driving method. For the cases constructed by other methods, **Pile Design Handbook for Highway Bridge Foundation** [27] may be referred to. Since little knowledge is available concerning the skin friction force of piles in such ground, the design in such ground needs not to expect the skin friction force or the skin friction force needs to be confirmed by the loading test.

When the top surface of rigid bearing stratum, such as bedrock, is inclined, the pile bottom needs to be embedded to some extent into bearing stratum to avoid sliding of piles when load acts on them. When driving a pile into bedrock, care needs to be taken not to break the pile bottom during construction.

(15) Adequate care needs to be taken as the action of seismic wave significantly lowers the shear strength of the ground, and thus the pushing resistance force of a pile in its axial direction is also known to reduce significantly. For example, since the seismic wave liquefies sandy ground or sensitive clayey ground loses strength, it is important to

determine the characteristic value of the pushing resistance force of a pile in its axial direction by considering the influence.

(16) When piles are jointed, construct under proper control and check the reliability of the joint by inspection since the joint may become a weak point of piles. If the joint is structurally uncertain, reduce the design value of the pushing resistance force of a pile in its axial direction. The rate of reduction shall be properly determined considering the structure of the joint and other factors. The rate of reduction used to be 20% per joint in the past, which may be referred to.

Field circumferential welding using a semi-automatic welding method is commonly used for steel pipe piles. In this case, there is no need to reduce the design value of the pushing resistance force in the axial direction if constructed under proper control and the reliability of the joint is checked by inspection.

For the structure of joints and others, see also **Part III, Chapter 2, 3.4.12 (3)**.

(17) Although piles are elongated and compressed members, the surrounding ground eliminates the need to consider buckling provided that there is no problem in pile body and construction of piles. However, inclination of piles occurring during construction may lower the bearing capacity of piles, and thus the design value of the pushing resistance force of a pile in its axial direction shall be reduced considering the accuracy of construction for piles whose rate of length to diameter is quite high. However, when the design value has been determined based on the result of pile loading test, there is no need to reduce the design value since the influence of construction accuracy is considered to have been added.

For the reduction rate, see **equation (3.4.9)** for steel piles and **equation (3.4.10)** for other than steel piles.

$$\alpha = \begin{cases} 0 & \dfrac{l}{d} \le 120 \\ \dfrac{l}{2d} - 60 & \dfrac{l}{d} > 120 \end{cases} \tag{3.4.9}$$

$$\alpha = \begin{cases} 0 & \dfrac{l}{d} \le 60 \\ \dfrac{l}{d} - 60 & \dfrac{l}{d} > 60 \end{cases} \tag{3.4.10}$$

where

α : reduction rate (%)

l : length of pile (m)

d : diameter of pile (m)

Equations (3.4.9) and **(3.4.10)** have been determined considering that piles may incline on the order of 1 degree during normal construction of piles. The reduction rate for steel piles is less than that of other piles considering certainty in construction of joints, resistance to bending, high accuracy in construction of pile bodies, etc.

3.4.4 Pulling Resistance Force of a Pile in Its Axial Direction

(1) The characteristic value of the pulling resistance force of a pile in its axial direction is generally determined based on the pulling resistance force of a pile in its axial direction exerted when ground reaches its failure condition by the pulling force in the axial direction acting on the pile head.

(2) The characteristic value of the pulling resistance force of a pile in its axial direction is expressed by the sum of the characteristic value of skin friction force of the pile and the characteristic value of self-weight (underwater weight if in water) of the pile. Although the weight of the soil stuck in a pile, filling material, or others may be included in the self-weight of a steel pipe or other piles, it is safer not to consider their weight since soil and others in a pile may drop from the pile of large diameter. If there is a device to surely integrate the soil in a pile, filling material, or others with the pile, their weight may be considered even for a pile of large diameter.

The characteristic value of the skin friction force of a pile is considered to be determined by multiplying the skin friction force per unit contact area of the pile shaft and the ground by the perimeter surface area of the pile as in the case of the pushing resistance force of a pile in its axial direction (see **equation (3.4.2)**). However, the value of skin friction force per unit contact area differs since behavior of the ground around the pile differs when the pile is pushed in and pulled out.

(3) The characteristic value of the pulling resistance force of a pile in its axial direction shall be determined by the loading test of the pile. The test pile used for the loading test shall have the same specifications as far as possible as the pile to be used for actual construction. If the specifications and ground conditions for test piles and actual piles coincide, the characteristic value of the pulling resistance force in the axial direction can be directly obtained from the result of the loading test. If the specifications or ground conditions of test piles and actual piles are different, the characteristic value of the pulling resistance force in the axial direction shall be calculated from the characteristic value of the skin friction force of a pile per unit contact area in each layer and the characteristic value of self-weight of a pile. When the characteristic value of the pulling resistance force of a pile in its axial direction is determined based on the characteristic value of the skin friction force per unit contact area obtained from the pile loading test other than pulling test, the difference in behavior of piles between pushing in and pulling out needs to be properly considered.

The detail in **Part III, Chapter 2, 3.4.3 (3)** should also be referred to.

(4) When verifying the bearing capacity of a pile, the design value determined by considering the safety margin based on the characteristic value of the pulling resistance force of a pile in its axial direction shall be used.

The safety margin was considered in the past by setting a safety factor of 3.0 or more for stationary load and 2.5 or more for load during earthquake. These values can be referred to except in the cases where the pulling force in the axial direction acting on the pile is extremely strong or of extremely long duration. However, as pulling failure of a pile tremendously affects the whole structure, measures such as re-examination of pile arrangement should be taken if the pulling force in the axial direction acts on the pile for a long time in permanent state or others (see **Part III, Chapter 2, 3.4.2 (5)**).

There is an opinion to exclude the self-weight of piles from the target to consider safety margin assuming that it is surely expected as pulling resistance. However, such method is generally avoided if the ratio of self-weight of piles to the pulling resistance force in the axial direction is not so large.

The detail in **Part III, Chapter 2, 3.4.3 (4)** should also be referred to.

(5) If preliminary loading test is difficult to perform, it is necessary to estimate the characteristic value of the pulling resistance force in the axial direction based on various estimation formulas. In this case, it is required to perform the loading test to confirm the bearing capacity of piles at the beginning of construction and check the validity of values used for performance verification. However, if the applicable range of used estimation formula and the error and dispersion of estimated values in that range is clearly indicated, the verification fully considering that error and dispersion can replace the loading test.

When using an estimation formula, care should be taken for the handling of self-weight of piles. In many cases, the characteristic value of the pulling resistance force in the axial direction is obtained by calculating the characteristic value of the skin friction force of a pile using an estimation formula and adding the characteristic value of self-weight of a pile. Care should be taken since the handling of self-weight of a pile may be different according to the used estimation formula.

The detail in **Part III, Chapter 2, 3.4.3 (5)** should also be referred to.

(6) For the characteristic value of the skin friction force per unit contract area of piles constructed by the hummer driving method in sandy ground and in clayey ground, see **Part III, Chapter 2, 3.4.3 (10)** and **3.4.3 (11)**, respectively. If other construction methods are used, see **Part III, Chapter 2, 3.4.3 (12)**.

Since little knowledge is available concerning the skin friction force of piles in the ground composed of soft rock, hard clay, bedrock, and others, the design in such ground should not expect the skin friction force or the skin friction force needs to be confirmed by the loading test.

(7) For the influence of seismic wave, see **Part III, Chapter 2, 3.4.3 (15)**.

(8) As how much pulling force in the axial direction can be transferred by a joint is mostly unknown, the skin friction force below the joint shall be ignored. The skin friction force below the joint may be taken into account if a good joint can be installed in steel pipe or other piles and its reliability can be confirmed. The verification of joint failure due to pulling force is needed in this case.

3.4.5 Displacement of Pile Head due to Axial Directional Force

(1) In the verification of settlement and upward displacement by pulling of a pile head, the pile head displacement of each pile due to action of axial directional force in pile head and, as needed, differential settlement and deformation of foundation and superstructure due to difference in settlement between piles shall be examined. In facilities where live load such as piled pier and crane foundations is dominant, elastic settlement of pile heads also needs to be examined.

(2) The settlement of pile heads where pushing force in the axial direction acts on a pile is expressed by **equation (3.4.11)**.

$$S_0 = S_p + S_S = S_{PE} + S_{PP} + S_{SE} + S_{SP} \tag{3.4.11}$$

where

S_0 : total settlement of the pile head (m)

S_p : deformation of the pile body (m)

S_S : deformation of the ground at the pile bottom (m)

S_{PE} : elastic deformation of the pile body (m)

S_{PP} : plastic deformation of the pile body (m)

S_{SE} : elastic deformation of the ground at the pile bottom (m)

S_{SP} : plastic deformation of the ground at the pile bottom (m)

The elastic return of a pile head and the residual settlement when pushing force in the axial direction is removed are expressed by **equations (3.4.12)** and **(3.4.13)**, respectively.

$$S_{0E} = S_{PE} + S_{SE} - S_f \tag{3.4.12}$$

where

S_{0E} : elastic return of a pile head (m)

S_f : restraint of elastic return due to skin friction (m)

$$S_{0P} = S_{PP} + S_{SP} + S_f \tag{3.4.13}$$

where

S_{0P} : residual settlement of a pile head (m)

(3) It is generally difficult to measure each value in **equations (3.4.11)**, **(3.4.12)**, and **(3.4.13)** separately in the pile loading test. Practically, examination will be performed based on the load-settlement relations of pile heads obtained from the loading test and the vertical spring constant of pile heads keeping **equations (3.4.11)**, **(3.4.12)**, and **(3.4.13)** in mind.

In the pile loading test, behavior as a pile group cannot be confirmed, and short loading time provides no information about the influence of creep and consolidation of clayey ground. Therefore, it is necessary to calculate the settlement of a pile head from the result of loading test fully considering the influence .

When setting the characteristic value of the resistance force of a pile in its axial direction by fully considering the safety margin, the creep of clayey ground on the circumferential surface of the pile may not influence much.

(4) The elastic settlement of a pile head can be obtained as a sum of elastic deformation of the pile itself and that of the ground at the pile bottom. Although elastic return (see **equation (3.4.12)**) instead of elastic settlement of a pile head is measured in the loading test of a pile, measured elastic return can practically be used as elastic settlement of a pile head. Moreover, there is a method to calculate the elastic deformation of a pile body and ground at the pile bottom, or the spring constant in the axial direction at the pile head assuming the distribution of skin friction force in depth direction, etc.[28) 29)].

(5) In friction piles in clayey layer or bearing piles having soft clayey layer below the bearing stratum, clayey layer is consolidated by the load transmitted from piles. The settlement of pile foundation at this time can be assumed to be the consolidation settlement occurring in clayey layer.

When calculating the consolidation settlement of clayey layer for friction piles in clayey layer, where to set the plane where load acts (loading plane) in clayey layer becomes the issue. The depth of loading plane can be assumed somewhere between the pile head and the pile bottom. Practically, methods such as setting the loading plane at the depth 1/3 from the lower edge of the pile embedment portion, replacing the pile foundation with a deep foundation embedded to the plane, and examining the consolidation settlement at the foundation bottom are used [30)]. However, if the settlement of pile foundation has a profound impact on the facility the foundation bears, it is desirable to calculate the case where the loading plane is also set to the head or bottom of a pile and compare. Moreover, if the superstructure is not stiff, an unexpected event may happen by differential settlement of piles due to settlement of the whole foundation. In the ground where especially big settlement is anticipated, care needs to be taken to make structures safe for settlement.

For friction piles in sandy soil layer where weak layer exists below the pile bottom, the settlement of a foundation can also be calculated with a similar method. At this time, care needs to be taken for penetration failure into the weak layer when the pile bottom is not separated by on the order of 3 times of the pile diameter from the upper edge of the weak layer.

(6) When piles behave as a pile group, note that the settlement may be greater than behaving as a single pile of the same load acting per one pile [31)]. For behavior as a pile group, see **Part III, Chapter 2, 3.4.10 Bearing Capacity as a Pile Group**.

(7) Pile foundations using bearing piles on bearing stratum of good quality bedrock settle less. Plastic deformation and residual deformation do not normally exceed several millimeters since the elastic deformation of pile body accounts for the most part of settlement of a pile head. Pile group may settle more to a certain degree, but no special consideration for it is necessary.

If the quality of bedrock is not so good, the settlement needs to be examined after exploring the ground, understanding the compression characteristics, etc.

(8) **Design Recommendations for Foundations of Buildings**[32)] indicates proposed values for allowable settlement, which may be referred to.

(9) It is desirable to judge the upward displacement of piles when the pulling force in the axial direction acts on the pile by individually performing loading test (see **Reference (Part II), Chapter 1, 3.10 Pile Load Test**) as few existing documents are available. Since creep and others in clayey soil induced by long-term load affect the upward displacement, it is needed to consider long-term deformation and other properties of clayey soil together with the result of loading test. However, if the pulling resistance force in the axial direction is verified fully considering safety margin, influence of creep and others are generally considered to be moderate.

3.4.6 Deflection of a Pile Subjected to Force in the Direction Perpendicular to the Axis

(1) If force in the direction perpendicular to the axis acts on piles, the resistance force corresponding to failure conditions of the ground cannot be defined unambiguously since the range of resisting ground gradually extends from the surface to deeper range (small ground failure phenomenon happens gradually) as deflection of piles increases. Thus, the bearing capacity in the direction perpendicular to the axis of a single pile is generally verified by calculating the deflection of a pile when the force in the direction perpendicular to the axis acts on the pile and confirming that the failure of pile body is not induced by the bending stress caused in the pile body and the displacement and angle of inclination of the pile head do not exceed the values determined from the allowable displacement of facilities the foundation bears.(2) The behavior of piles to which the force in the direction perpendicular to the axis acts is greatly influenced by various conditions such as specifications of piles, ground conditions, loading conditions, and securing conditions of pile heads. Therefore, even when the horizontal loading test of piles has been performed, the behavior of test piles obtained in the test rarely coincides with the behavior of actual piles, and thus it is difficult to examine the bearing capacity of a pile in the direction perpendicular to the axis from the loading test only. In many cases, the ground constant (mainly modulus of subgrade reaction) is determined from the result of loading tests and the behavior of actual piles is estimated with an analytical method using the constant.

When performing a loading test using a test pile whose specifications, ground conditions, loading conditions, and others completely coincide with those of an actual pile and the load can be loaded until bending failure of a pile body occurs or the displacement of the pile head reaches a predetermined value, the load in the direction perpendicular to the axis at that time can be the characteristic value of the resistance force of a pile in the direction perpendicular to the axis. In this case, the bearing capacity for the force of a pile in the direction perpendicular to the axis can be verified by determining a design value considering the safety margin based on the characteristic value of the resistance force in the direction perpendicular to the axis and confirming that the force in the direction perpendicular to the axis acting on the pile head does not exceed it. A safety factor of 3.0 or more was used as the safety margin in the past, which may be referred to.

For the method of loading test, see [Reference (Action)] Chapter 1, 3.10 Pile Load Test.

(3) When estimating the behavior of a pile based on the ground constant obtained from the loading test or others, the method of analysis which assumes a pile as a beam on elastic foundation is commonly used. The basic equation that represents the behavior of a beam on elastic foundation is expressed by **equation (3.4.14)**.

$$EI\frac{d^4y}{dx^4} = -P = -pB \tag{3.4.14}$$

where

EI : bending stiffness of a pile (kN·m^2)

x : depth below the ground level (m)

y : displacement of a pile at depth x (m)

P : subgrade reaction per unit length of a pile at depth x (kN/m)

p : subgrade reaction per unit area of a pile at depth x (kN/m^2)

$p = P/B$

B : width of a pile (m)

How to express the subgrade reaction in **equation (3.4.14)** has been largely discussed. Typical ways include Chang's method and the Port and Harbour Research Institute (PHRI) method. For the calculation method of deflection of a pile by Chang's method and by the PHRI method, see **Part III, Chapter 2, 3.4.7 Calculation of Deflection of a Pile by Chang's Method** and **Part III, Chapter 2, 3.4.8 Calculation of Deflection of a Pile by the PHRI Method**, respectively.

Chang's method is simple to use as an analysis method, whereas the PHRI method is said to be able to express the behavior of a pile more accurately. Therefore, analysis by the PHRI method shall be generally used. If the PHRI method is difficult to apply in cases such as an analysis coupling the pile foundation and superstructures, Chang's method may be used.

(4) The behavior of a pile on the head of which the force in the direction perpendicular to the axis acts is extremely affected by the embedment length of a pile.

Even if the force in the direction perpendicular to the axis acts on a long embedment pile, there will be almost no displacement in a portion close to the end of the pile. The subgrade reaction does not change in a portion where there is no change in displacement and does not act effectively on the force in the direction perpendicular to the axis. The length of the portion which resists effectively to the force in the direction perpendicular to the axis within the embedment length is called effective length.

A pile the embedment length of which is longer or shorter than its effective length is called a long pile or short pile, respectively. The behavior of a long pile on which the force in the direction perpendicular to the axis acts is determined irrespective of the embedment length of a pile and is something as if its bottom were fixed to the ground. On the other hand, the behavior of a short pile changes according to the embedment length. Short embedment length makes close to circulating rather than bending behavior of a pile when the force in the direction perpendicular to the axis acts on it. In an extreme case, the ground around the pile is totally destroyed, and the pile may fall down when the force in the direction perpendicular to the axis exceeds a certain value. Therefore, examination on the failure of pile body, pile head displacement, as well as failure of the ground will be necessary for short piles. Moreover, short piles tend to be more subjected to the effect of repeated load and creep of clayey ground than long piles. These facts make it difficult to accurately forecast the behavior of short piles. As such, short piles should not be used when the force in the direction perpendicular to the axis acts on them. For details of the behavior of long and short piles, see **3.4.8 (8)** in this Chapter.

(5) The calculation of deflection of piles by Chang's and the PHRI methods assumes that the ground in the range where the subgrade reaction is caused by the action of the force on the pile head in the direction perpendicular to the axis is homogeneous in the direction of depth. Therefore, special caution is required when anticipating the behavior of a pile in multilayer ground which has a layer structure in the direction of depth.

Both Chang's method and the PHRI method are applicable to multilayer ground by numerical calculation or other means, but their validity has not been fully examined yet. The validity of a ground model used for analysis and the stability of calculation results need to be examined when the ground property changes abruptly at layer boundaries of multilayer ground or thin weak layer is sandwiched. Moreover, estimation of the behavior of piles in multilayer ground often supposes that the ground continues indefinitely in the horizontal direction. Thus, care needs to be taken in handling when there is a layer which extends definitely in horizontal direction such as replacement sand layer.

(6) In very inhomogeneous ground in horizontal direction like clayey ground improved by sand compaction piles, the behavior of a pile when the force in the direction perpendicular to the axis acts is subjected to the property of weak parts in the ground. The resistance force of a pile in the direction perpendicular to the axis in such improved grounds is not expected to increase from the condition before the ground improvement in many cases [33) 34)].

(7) The contact condition between piles and riprap in riprap layer may not be good because of wide gaps between ripraps or quasi self-standing riprap. Thus, the subgrade reaction is not sufficiently exerted, and the resistance force of a pile in the direction perpendicular to the axis becomes small when the force in the direction perpendicular to the axis acts on the pile head, especially when deflection of piles is small or in other occasions [35) 36)]. However, bigger deflection of piles may improve the contact condition between riprap and piles, and the bigger subgrade reaction may be exerted. Given this situation, when verifying the resistance force of a pile in the direction perpendicular to the axis in riprap layer, it is better to examine the case also where the modulus of subgrade reaction is different from the design condition and confirm the change in behavior of piles against the change in modulus of subgrade reaction.

(8) The validity of the method to analyze the piles embedded into bedrock as a beam on elastic foundation has not been well examined. It is necessary to examine whether modeling the bedrock as a spring is problematic or not, how to set the modulus of subgrade reaction in that case, etc. Moreover, an examination from many angles is required (i.e., to examine if piles break at the top end of bedrock or not).

(9) Construction of piles may change the ground condition around the piles from the time of preliminary ground exploration and affect the resistance force of a pile in the direction perpendicular to the axis. Especially, when piles are constructed in a way to loosen the surrounding ground, it is necessary to pay full attention as the resistance force of a pile in the direction perpendicular to the axis may decrease.

(10) The displacement of pile heads tends to increase gradually when the force in the direction perpendicular to the axis repeatedly acts in one direction. The displacement of piles with enough embedment length increases in proportion to the logarithm of the repeat count [37)]. Practically, when the force in the direction perpendicular to the axis repeatedly acts in one direction on a pile in sandy ground, it is enough to anticipate 1.4 times of pile head displacement when the force in the direction perpendicular to the axis acts once [38)]. Increase in the pile head displacement may be bigger when the force in the direction perpendicular to the axis repeatedly acts on a pile in clayey ground [39)]. The same is true in cases where the force in the direction perpendicular to the axis continuously acts for a long time.

The pile head displacement when the force in the direction perpendicular to the axis repeatedly acts in two directions is almost unchanged from the case where the force in the direction perpendicular to the axis acts once in sandy ground [40)]. In this case, it is enough to anticipate 1.1 times of pile head displacement when the force in the direction perpendicular to the axis acts once. In clayey ground, attention is needed because the displacement tends to increase as the repeat count increases.

The difference in the behavior between piles in sandy ground and those in clayey ground comes from the difference in the intensity change in surrounding ground of the piles subjected to repeated shear. The density of sandy ground tends to increase due to repeated shear, whereas increase in excess pore water pressure in clayey ground due to repeated shear leads to reduction of shear resistance. When action continues for a long time, the creep phenomenon of the surrounding ground affects the behavior of piles.

(11) When the force in the direction perpendicular to the axis repeatedly acts on piles in sandy ground, the maximum bending moment of the piles may be considered the same as when the force in the direction perpendicular to the axis acts once on piles, and the embedment length may be determined by anticipating an effective length about 1.1 times when the force in the direction perpendicular to the axis acts once.

When the force in the direction perpendicular to the axis repeatedly acts on piles in clayey ground, extra maximum bending moment caused in piles and extension of embedment length of piles need to be examined [41].

(12) When the force in the direction perpendicular to the axis dynamically acts, the bearing capacity of piles can generally be verified according to the case where statistical load is repeatedly applied. However, the reduction of subgrade reaction due to dynamic action or the influence of ground liquefaction needs to be considered. For structures where piles of long free length are used such as piled piers, verification should be performed using dynamic analysis and others considering dynamic interactions between piles and the ground (see **Part III, Chapter 5, 5.2 Vertical Pile Type Piled Pier**).

(13) When calculating the circular slip failure of the ground, assume that there are no piles if the slip failure crosses the pile unless the pile is intended to suppress slip and the effect has been adequately evaluated. Sheet pile walls can be handled as piles.

(14) When liquefaction in earthquake flows a part of the ground to a side, large force in the direction perpendicular to the axis will act on piles from the ground. Behavior of piles in this case has scarcely been clarified, but some indicated concepts [42] may be referred to.

3.4.7 Calculation of Deflection of a Pile by Chang's Method

(1) Chang's method [43] assumes that a pile subjected to force in the direction perpendicular to the axis is a beam on elastic foundation and analyzes its behavior. Chang's method assumes that the subgrade reaction per unit area is expressed by **equation (3.4.15)** in **equation (3.4.14)**, which expresses the behavior of a beam on elastic foundation.

$$p = \frac{E_s}{B} y = k_{CH}\, y \tag{3.4.15}$$

where

E_s : modulus of elasticity of the ground (kN/m^2)

k_{CH} : coefficient of lateral subgrade reaction (kN/m^3)

The basic formula for Chang's method is expressed individually for above the ground level and below the ground level as in **equation (3.4.16)**, considering that the subgrade reaction does not act above the ground level when the pile protrudes above the ground level.

$$\left.\begin{aligned} &\text{Above the ground level} \quad EI\,\frac{d^4 y_z}{dz^4} = 0 \\ &\text{Below the ground level} \quad EI\,\frac{d^4 y_x}{dx^4} + Bk_{CH} y_x = 0 \end{aligned}\right\} \tag{3.4.16}$$

where

EI : bending stiffness of a pile (kN·m^2)

z : height above the ground (m)

0 at the pile head, h on the ground level

h : protrusion length of a pile (m)

x : depth below the ground level (m)

y_z : displacement of a pile at height z (m)

y_x : displacement of a pile at depth x (m)

k_{CH} : coefficient of lateral subgrade reaction (kN/m^3)

B : width of a pile (m)

Equation (3.4.16) can be resolved analytically assuming that the modulus of elasticity of the ground $E_S = Bk_{CH}$ is constant.

Although Chang's method assumes that pile length is semi-infinite (embedment length is infinite), it is deemed that there is no big difference between the behavior of finite length piles and semi-finite length piles provided that **equation (3.4.17)** is true [44].

$$L \geq \frac{\pi}{\beta} \tag{3.4.17}$$

where

L : embedment length of a pile (m)

β : characteristic value of a pile (m^{-1})

$$\beta = \sqrt[4]{\frac{B\,k_{CH}}{4EI}}$$

That is, the behavior of a pile satisfying **equation (3.4.17)** is not subjected to the embedment length and thus considered to be a long pile. On the other hand, a pile not satisfying **equation (3.4.17)** needs to be handled as a finite length pile, and another simple solution [45] is shown. As stated in **3.4.6 (4)** in this Chapter, as it is difficult to accurately estimate the behavior of short piles, they should not be used when piles are subjected to force in the direction perpendicular to the axis.

(2) When rotation of a pile head is allowed (free head pile), the deflection curve of the pile obtained from **equation (3.4.16)** is expressed by **Equation (3.4.18)** (see **Fig. 3.4.3**).

$$\left.\begin{aligned} y_z &= y_t - \theta_t z + \frac{M_t}{2EI} z^2 + \frac{H_t}{6EI} z^3 \\ y_x &= \frac{H_t}{2EI\beta^3} e^{-\beta x} \left\{ \left(1+\beta h_0\right) \cos\beta x - \beta h_0 \sin\beta x \right\} \end{aligned}\right\} \tag{3.4.18}$$

where

H_t : force in the direction perpendicular to the axis acting on a pile head (kN)

M_t : moment acting on a pile head (kN·m)

h_0 : $h_0 = h + M_t/H_t$

y_t : displacement of a pile head (m)

$$y_t = \frac{2\left(1+\beta h\right)^3 + 1}{6EI\beta^3} H_t + \frac{\left(1+\beta h\right)^2}{2EI\beta^2} M_t$$

θ_t : inclination of a pile (rad)

$$\theta_t = \frac{\left(1+\beta h\right)^2}{2EI\beta^2} H_t + \frac{1+\beta h}{EI\beta} M_t$$

Then, the displacement of a pile on the ground level is expressed by **equation (3.4.19)**.

$$y_0 = \frac{1+\beta h_0}{2EI\beta^3} H_t \tag{3.4.19}$$

where

y_0 : displacement of a pile on the ground level (m)

The bending moment and shear caused in a pile body are expressed by **equations (3.4.20)** and **(3.4.21)**, respectively.

$$\left.\begin{aligned} M_z &= -M_t - H_t z \\ M_x &= -\frac{H_t}{\beta} e^{-\beta x} \left\{\beta h_0 \cos\beta x + (1+\beta h_0)\sin\beta x\right\} \end{aligned}\right\} \tag{3.4.20}$$

$$\left.\begin{aligned} S_z &= -H_t \\ S_x &= -H_t e^{-\beta x} \left\{\cos\beta x - (1+2\beta h_0)\sin\beta x\right\} \end{aligned}\right\} \tag{3.4.21}$$

where

M_z : bending moment at height z (kN·m)

M_x : bending moment at depth x (kN·m)

S_z : shear at height z (kN)

S_x : shear at depth x (kN)

The maximum bending moment caused in a pile body below the ground level is expressed by **equation (3.4.22)**.

$$M_{\max} = -\frac{H_t}{2\beta}\sqrt{(1+2\beta h_0)^2+1}\, e^{-\beta l_{m,\max}} \tag{3.4.22}$$

where

$M_{\max}$: maximum bending moment below the ground level (kN·m)

$l_{m,\max}$: depth at which the maximum bending moment below the ground level is caused (m)

$$l_{m,\max} = \frac{1}{\beta}\tan^{-1}\frac{1}{1+2\beta h_0} \qquad \left(l_{m,\max} = \frac{\pi}{4\beta} \text{ when } h=0 \text{ and } M_t = 0 \right)$$

The depths at which the displacement, angle of deflection, and bending moment of a pile become zero are expressed by **equations (3.4.23)**, **(3.4.24)**, and **(3.4.25)**, respectively.

$$l_{y1} = \frac{1}{\beta}\tan^{-1}\frac{1+\beta h_0}{\beta h_0} \qquad \left(l_{y1} = \frac{\pi}{2\beta} \text{ when } h=0 \text{ and } M_t = 0 \right) \tag{3.4.23}$$

$$l_{i1} = \frac{1}{\beta}\tan^{-1}\left\{-(1+2\beta h_0)\right\} \qquad \left(l_{i1} = \frac{3\pi}{4\beta} \text{ when } h=0 \text{ and } M_t = 0 \right) \tag{3.4.24}$$

$$l_{m1} = \frac{1}{\beta}\tan^{-1}\frac{-\beta h_0}{1+\beta h_0} \qquad \left(l_{m1} = \frac{\pi}{\beta} \text{ when } h=0 \text{ and } M_t = 0 \right) \tag{3.4.25}$$

where

l_{y1} : depth of the first zero point (the first fixed point) of displacement (m)

l_{i1} : depth of angle of deflection first zero point of a free head pile or depth of angle of deflection second zero point of a fixed head pile (m)

l_{m1} : depth of bending moment first zero point of a free head pile or depth of bending moment second zero point of a fixed head pile (m)

When the pile head coincides with the ground level, calculate the above equations with h = 0 (use equations for underground part for **equations (3.4.18)**, **(3.4.20)**, and **(3.4.21)**).

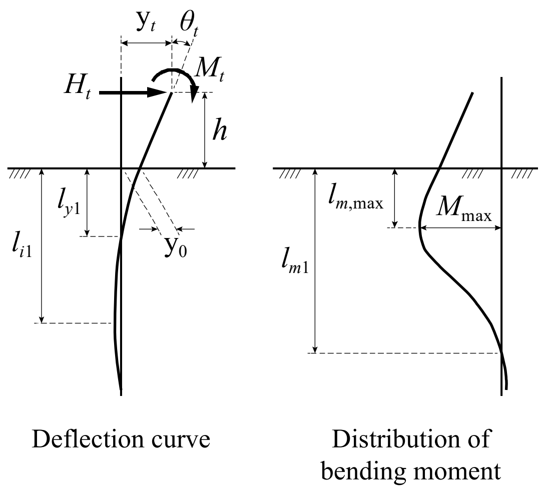


Fig. 3.4.3 Deflection Curve and Distribution of Bending Moment for Free Head Piles

(3) When the pile head does not rotate (fixed head pile), the deflection curve of the pile obtained from **equation (3.4.16)** is expressed by **equation (3.4.26)** (see **Fig. 3.4.4**).

$$\left.\begin{aligned} y_z &= y_t - \frac{(1+\beta h)\,H_t}{4EI\beta}\,z^2 + \frac{H_t}{6EI}\,z^3 \\ y_x &= \frac{H_t}{4EI\beta^3}\,e^{-\beta x}\left\{(1+\beta h)\cos\beta x + (1-\beta h)\sin\beta x\right\} \end{aligned}\right\} \tag{3.4.26}$$

where

y_t : displacement of a pile head (m)

$$y_t = \frac{(1+\beta h)^3 + 2}{12EI\beta^3}\,H_t$$

The displacement of a pile on the ground level at this time is expressed by **equation (3.4.27)**.

$$y_0 = \frac{1+\beta h}{4EI\beta^3}\,H_t \tag{3.4.27}$$

The bending moment and shear caused in a pile body are expressed by **equations (3.4.28)** and **(3.4.29)**, respectively.

$$\left.\begin{aligned} M_z &= \frac{H_t}{2\beta}\,(1+\beta h - 2\beta z) \\ M_x &= \frac{H_t}{2\beta}\,e^{-\beta x}\left\{(1-\beta h)\cos\beta x - (1+\beta h)\sin\beta x\right\} \end{aligned}\right\} \tag{3.4.28}$$

$$\left.\begin{aligned} S_z &= -H_t \\ S_x &= -H_t\,e^{-\beta x}\left(\cos\beta x - \beta h\sin\beta x\right) \end{aligned}\right\} \tag{3.4.29}$$

The maximum bending moment caused in a pile body below the ground level is expressed by **equation (3.4.30)**.

$$M_{\max} = -\frac{H_t}{2\beta}\sqrt{1+(\beta h)^2}\;e^{-\beta l_{m,\max}} \tag{3.4.30}$$

where

$l_{m,max}$: depth at which the maximum bending moment below the ground level is caused (m)

$$l_{m,\max} = \frac{1}{\beta}\tan^{-1}\frac{1}{\beta h} \qquad \left(l_{m,\max} = \frac{\pi}{2\beta} \quad \text{when} \quad h = 0 \right)$$

The depths at which the displacement, angle of deflection, and bending moment of a pile become zero are expressed by **equations (3.4.31)**, **(3.4.32)**, and **(3.4.33)**, respectively.

$$l_{y1} = \frac{1}{\beta}\tan^{-1}\frac{\beta h + 1}{\beta h - 1} \qquad \left(l_{y1} = \frac{3\pi}{4\beta} \quad \text{when} \quad h = 0 \right) \tag{3.4.31}$$

$$l_{i1} = \frac{1}{\beta}\tan^{-1}(-\beta h) \qquad \left(l_{i1} = \frac{\pi}{\beta} \quad \text{when} \quad h = 0 \right) \tag{3.4.32}$$

$$l_{m1} = \frac{1}{\beta}\left(\tan^{-1}\frac{1 - \beta h}{1 + \beta h} + \pi \right) \qquad \left(l_{m1} = \frac{5\pi}{4\beta} \quad \text{when} \quad h = 0 \right) \tag{3.4.33}$$

When the pile head coincides with the ground level, calculate the above equations with h = 0 (use equations for underground part for **equations (3.4.26)**, **(3.4.28)**, and **(3.4.29)**).

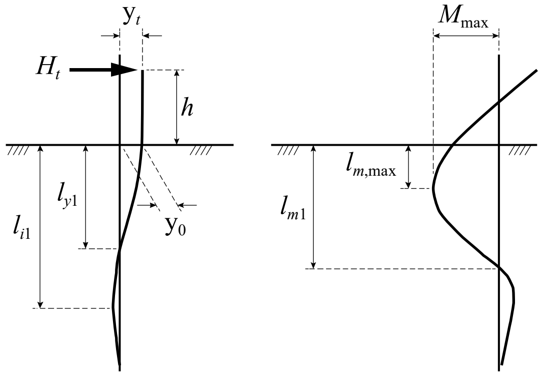


Fig. 3.4.4 Deflection Curve and Distribution of Bending Moment for Fixed Head Piles

(4) A method to obtain the coefficient of lateral subgrade reaction for clayey soil from **equation (3.4.34)** and for sandy soil from **equation (3.4.35)** has been proposed [46].

$$k_{CH} = \frac{0.2}{B}k_{CH1} \tag{3.4.34}$$

where

k_{CH} : coefficient of lateral subgrade reaction (kN/m^3)

k_{CH1} : value shown in **Table 3.4.1** (kN/m^2)

B : width of a pile (m)

$$k_{CH} = n_h\frac{x}{B} \tag{3.4.35}$$

where

n_h : value shown in **Table 3.4.2** (kN/m^3)

x : depth (m)

In the case of sandy soil, **equation (3.4.35)** shows that the coefficient of lateral subgrade reaction is a function of depth and thus cannot be introduced to Chang's method as it stands. Then, it is believed that the coefficient of lateral subgrade reaction at depth of 1/3 of the first fixed point may be used when using the coefficient of lateral subgrade reaction for sandy soil calculated from **equation (3.4.35)** in Chang's method. However, the depth of the first fixed point is a function of the coefficient of lateral subgrade reaction (see **equations (3.4.23)** and **(3.4.31)**), and thus repeated calculation is required. A method to calculate using a chart instead of repeated calculation has been proposed [47].

The coefficient of lateral subgrade reaction is inversely proportional to the width of a pile in the proposed equation shown in **equations (3.4.34)** and **(3.4.35)**. On the other hand, it has been suggested that the modulus of subgrade reaction does not depend on the width of a pile (see **(5)** and **Part III, Chapter 2, 3.4.8 (9)**).

Table 3.4.1 Proposed Values for k_{CH1}

Consistency of clayey soil	Hard	Very hard	Solid
Unconfined compressive strength of clayey soil (kN/m^2)	100–200	200–400	400 or greater
Range of k_{CH1} (kN/m^2)	16,000–32,000	32,000–64,000	64,000 or greater
Proposed value of k_{CH1} (kN/m^2)	24,000	48,000	96,000

Table 3.4.2 Proposed Values for n_h

Relative density of sand	Loose	Medium	Dense
n_h for dry or wet sand (kN/m^3)	2,200	6,600	17,600
n_h for submerged sand (kN/m^3)	1,300	4,400	10,800

(5) Results of the horizontal loading test of steel piles performed in Japan show the relation between the coefficient of lateral subgrade reaction and the SPT-N value of the ground as in **Fig. 3.4.5** [48].

It is assumed that the coefficient of lateral subgrade reaction is not affected by the width of a pile based on the results of the horizontal loading test. Moreover, as the coefficient of lateral subgrade reaction obtained from the horizontal loading test tends to reduce as the load increases, the relation shown in **Fig. 3.4.5** uses the coefficient of lateral subgrade reaction when displacement of a pile on the ground level is 1 cm. On the other hand, the mean SPT-N value corresponding to the depth from the ground level to the inverse of characteristic value of a pile β^{-1} (also called the characteristic length of a pile) is used as the SPT-N value of the ground.

Fig. 3.4.5 may be referred to when a horizontal loading test is difficult to perform and the coefficient of lateral subgrade reaction at site cannot be confirmed. When calculating the displacement of a pile using the coefficient of lateral subgrade reaction estimated from **Fig. 3.4.5**, confirm that the displacement of a pile on the ground level is on the order of 1 cm. If the displacement of a pile is large, care needs to be taken as the coefficient of lateral subgrade reaction may be overestimated.

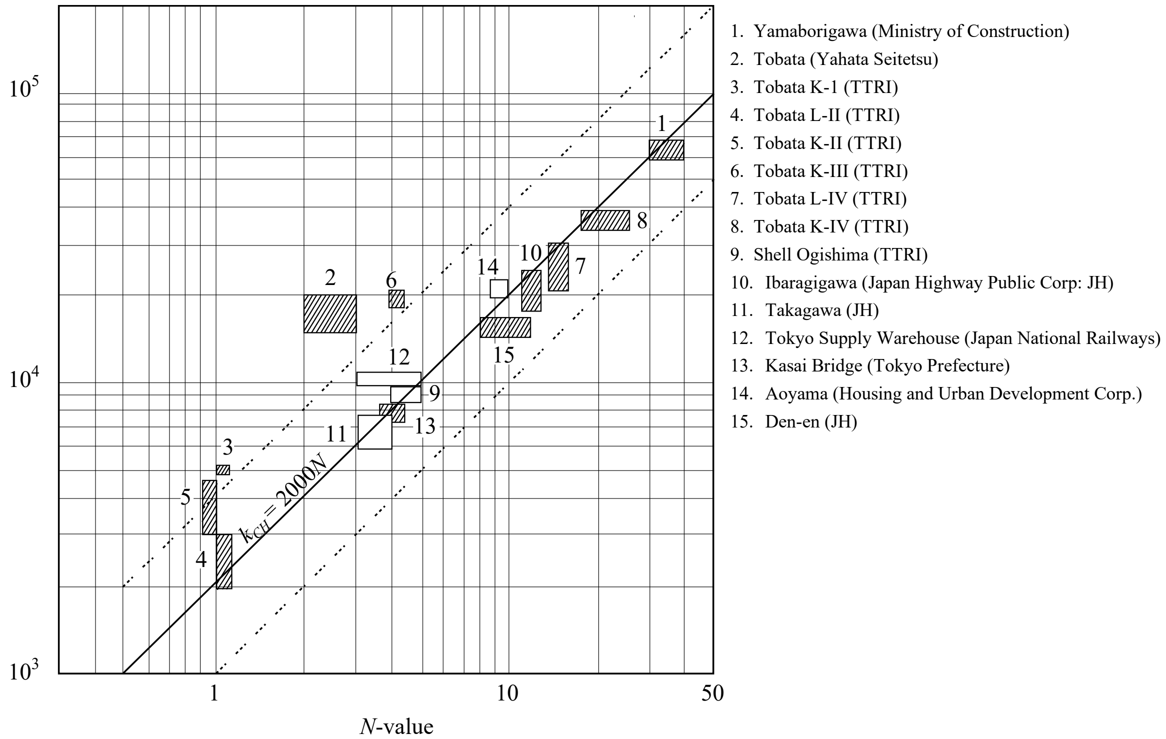


Fig. 3.4.5 Relation between the Coefficient of Lateral Subgrade Reaction Obtained from Horizontal Loading Test of a Pile and the SPT-N value of the Ground (Yokoyama [48]), added and altered)

(6) **Equation (3.4.36)** has been proposed as an estimation formula since it has been suggested that the coefficient of lateral subgrade reaction is affected by the section stiffness per unit width of a pile in the ground where the SPT-N value increases in the depth direction [49]).

$$\left.\begin{array}{ll}
\text{Free head pile, S-type ground} & k_{CH} = 103\left(\dfrac{EI}{B}\right)^{0.207} y_0^{-0.398}\, h^{-0.035}\, \bar{N}^{0.519} \\[2ex]
\text{Fixed head pile, S-type ground} & k_{CH} = 114\left(\dfrac{EI}{B}\right)^{0.216} y_0^{-0.392}\, h^{-0.088}\, \bar{N}^{0.513} \\[2ex]
\text{Free head pile, C-type ground} & k_{CH} = 721\left(\dfrac{EI}{B}\right)^{-0.001} y_0^{-0.499}\, h^{0.009}\, N^{0.649} \\[2ex]
\text{Fixed head pile, C-type ground} & k_{CH} = 705\left(\dfrac{EI}{B}\right)^{-0.005} y_0^{-0.501}\, h^{0.028}\, N^{0.651}
\end{array}\right\} \quad \textbf{(3.4.36)}$$

where

k_{CH} : coefficient of lateral subgrade reaction (kN/m^3)

EI : bending stiffness of a pile (kN·m^2)

B : width of a pile (m)

h : height of loading (protrusion length of a pile) (m)

y_0 : displacement of a pile on the ground level (m)

$\bar{N}$: increase rate of SPT-N value in depth direction (m^{-1})

N : SPT-N value

For classification of the S-type ground and the C-type ground, increase rate of the SPT-N value in depth direction used in **Equation (3.4.36)** and setting method of the SPT-N value, see **Part III, Chapter 2, 3.4.8 Calculation of Deflection of a Pile by the PHRI Method**.

Equation (3.4.36) shows that the coefficient of lateral subgrade reaction depends on the displacement of a pile on the ground level. However, as the displacement of a pile on the ground level is a function of the coefficient of lateral subgrade reaction (see **equations (3.4.19)** and **(3.4.27)**), it is necessary to repeat calculations and explore a solution in which the coefficient of lateral subgrade reaction becomes consistent with the displacement of a pile on the ground level.

(7) **Specifications for Highway Bridges, IV Substructures** [50] shows a method to estimate the coefficient of lateral subgrade reaction based on the result of various ground explorations, which can be referred to.

3.4.8 Calculation of Deflection of a Pile by the PHRI Method

(1) The PHRI method [51] analyzes the behavior of piles subjected to force in the direction perpendicular to the axis by assuming them as beams on elastic foundation. The PHRI method assumes that the subgrade reaction per unit area is expressed by **equation (3.4.37)** in **Equation (3.4.14)** which expresses the behavior of a beam on elastic foundation.

$$p = k\,x^m\,y^{0.5} \qquad \textbf{(3.4.37)}$$

where

k : lateral resistance coefficient of the ground (kN/m$^{2.5}$ or kN/m$^{3.5}$)

m : variable 1 or 0 according to the ground property

As seen in **equation (3.4.37)**, the feature of this method is that nonlinearity is introduced between the subgrade reaction per unit area p and the displacement of a pile y (p is proportional to $y^{0.5}$ rather than y). This makes it possible to express the actual behavior of piles more faithfully.

The PHRI method needs to rely on numerical analysis to obtain a solution since it cannot reach a general solution analytically. Moreover, care needs to be taken that the introduction of nonlinearity makes it impossible to apply the principle of superposition of solution. Solution method using mathematical formula or numerical table shown for specific conditions makes it possible to find a solution without numerical analysis.

The PHRI method is an analytical method for fully embedded piles (long piles). Its general application condition is that the embedment length into the ground is 1.5 l_{m1} or more. l_{m1} is the depth of the bending moment first zero point of a free head pile or depth of bending moment second zero point of a fixed head pile. For effective length of a pile, see the details in **(8)**.

(2) The PHRI method classifies ground into the S-type ground and the C-type ground. S-type ground is the ground the SPT-N value of which increases linearly with the depth such as sandy ground of uniform density and clayey ground in normal consolidation condition. C-type ground is the ground the SPT-N value of which is constant regardless of depth such as sandy ground with compacted surface and clayey ground subjected to large preconsolidation.

In **equation (3.4.37)**, m = 1 in S-type ground and m = 0 in C-type ground. In other words, **equation (3.4.37)** can be replaced with **equation (3.4.38)**.

$$\left.\begin{array}{ll} \text{S-type ground} & p = k_s\,x\,y^{0.5} \\ \text{C-type ground} & p = k_c\,y^{0.5} \end{array}\right\} \qquad \textbf{(3.4.38)}$$

where

p : subgrade reaction per unit area of a pile at depth x (kN/m^2)

k_s : lateral resistance coefficient in the S-type ground (kN/m$^{3.5}$)

k_c : lateral resistance coefficient in the C-type ground (kN/m$^{2.5}$)

x : depth (m)

y : displacement of a pile at depth x (m)

When classifying ground into the S-type ground and the C-type ground, focus on the range of the ground which affects the lateral resistance of piles. Generally, consider a range from the ground level to (0.5–1.0)l_{m1}. Although many grounds have intermediate properties, they can be handled as the ground closer to them.

(3) A method to obtain displacement of a pile head, the maximum bending moment below the ground level, deflection, and others from a numerical table based on the PHRI method has been shown [52)] for piles in relatively simple condition on the head of which only the force in the direction perpendicular to the axis acts. This method is to obtain displacement of a pile head and others of a pile to analyze (model pile) by multiplying the pre-calculated curve (reference curve) indicating relations between load and the displacement of a pile and others for a pile having the specifications shown in **Table 3.4.3** (reference pile) by a ratio determined based on a scaling law. A specific calculation method follows:

First, define the ratio of several values concerning characteristic or behavior of a model pile and the reference pile, as shown in **equation (3.4.39)**.

$$R_a = \frac{a_p}{a_s} \tag{3.4.39}$$

where

a : several values concerning characteristic or behavior of piles such as bending stiffness, displacement, bending moment.

R_a : ratio of a for a model pile to the reference pile

a_p : a of a model pile

a_s : a of the reference pile

Here, **equation (3.4.40)** is obtained by taking the logarithm of the both sided of **Equation (3.4.39)**.

$$\log R_a = \log a_p - \log a_s \tag{3.4.40}$$

In actual calculation, the relation in **equation (3.4.40)** is more convenient to use than **equation (3.4.39)**. The numerical table of the reference curve indicated below is also shown in logarithm corresponding to this.

When the ratio of several values for a model pile to the reference pile is defined as in **equation (3.4.39)** or **(3.4.40)**, scaling law induces the following **equation (3.4.41)**.

$$\left.\begin{aligned}
\log R_S &= (2m+5)\log R_x - \log R_{EI} + 2\log R_{Bk} \\
\log R_M &= (2m+6)\log R_x - \log R_{EI} + 2\log R_{Bk} \\
\log R_i &= (2m+7)\log R_x - 2\log R_{EI} + 2\log R_{Bk} \\
\log R_y &= (2m+8)\log R_x - 2\log R_{EI} + 2\log R_{Bk}
\end{aligned}\right\} \tag{3.4.41}$$

where

R_S : ratio of shear of a model pile to the reference pile

R_M : ratio of bending moment of a model pile to the reference pile

R_i : ratio of angle of deflection of a model pile to the reference pile

R_y : ratio of displacement of a model pile to the reference pile

R_x : ratio of depth of a model pile to the reference pile

R_{EI} : ratio of bending stiffness of a model pile to the reference pile

R_{Bk} : ratio of Bk of a model pile to the reference pile

B : width of a pile (m)

k : lateral resistance coefficient of the ground
k_s (kN/m$^{3.5}$) for S-type ground, k_c (kN/m$^{2.5}$) for C-type ground

m : variable 1 or 0 according to the ground property
1 for S-type ground, 0 for C-type ground

Here, R_x (ratio of depth of a model pile to the reference pile) can be understood as a scale ratio of a model pile to the reference pile in the direction of depth or height. That is, all ratios of these values concerning depth and height including protrusion length and embedment length of a pile, zero point depth of displacement, and the depth at which the maximum bending moment is caused follow this ratio. As piles are generally designed to have enough embedment length, the height of load from the ground level (protrusion length of a pile) often becomes the only given value among several values concerning depth. Thus, **equation (3.4.42)** is deemed to be true.

$$\log R_x = \log R_h \tag{3.4.42}$$

where

R_h : ratio of protrusion length of a model pile to the reference pile from the ground level

On the other hand, log R_{EI} and log R_{BK} can be calculated from **equation (3.4.40)** based on specifications of a model pile to examine and specifications of the reference pile shown in **Table 3.4.3**. Consequently, every term in the right side of **equation (3.4.41)** becomes a known amount. However, if the loading condition for a model pile is loading on the ground level (protrusion length = 0), log R_x = log R_h will not be defined, and thus **equation (3.4.41)** cannot be used. A method to calculate with another mathematical formula has been proposed in this case (see **(4)**).

Shear caused in a pile body above the ground level can be obtained from the balance to the force in the direction perpendicular to the axis acting on the head of a pile on which only the force in the direction perpendicular to the axis acts. Therefore, it is assumed that the ratio of the force in the direction perpendicular to the axis acting on a model pile to the reference pile is equal to the ratio of shear, and thus **equation (3.4.43)** is true.

$$\log R_T = \log R_S \tag{3.4.43}$$

where

R_T : ratio of the force in the direction perpendicular to the axis acting on the head of a model pile to the reference pile

Every value concerning the behavior of a model pile can be obtained by converting values read from the reference curve using the ratio of a model pile to the reference model for each value calculated above. Four reference curve groups have been calculated according to the pile head and ground conditions and given as numerical tables shown in **Tables 3.4.4–3.4.7**. Meaning of symbols in the tables are as follows:

T_S : force in the direction perpendicular to pile axis acting on the head of the reference pile (kN)

$y_{t,S}$: displacement of the head of the reference pile (m)

$\mathrm{M}_{\max,S}$: maximum bending moment of the reference pile below the ground level (kN·m)

$M_{top,S}$: bending moment at the head of the reference pile (kN·m)

$l_{m1,S}$: depth of the bending moment first zero point of a free head reference pile or of the bending moment second zero point of a fixed head reference pile (m)

$y_{0,S}$: displacement of the reference pile on the ground level (m)

$i_{t,S}$: angle of deflection of head of the reference pile (rad)

$i_{0,S}$: angle of deflection of the reference pile on the ground level (rad)

As an example, consider a case where the displacement of pile head is to be obtained when the force in the direction perpendicular to the axis T_p acts on the head of a model pile. First, a log T_S of the force in the direction perpendicular to the axis acting on the head of the reference pile can be calculated from the log R_T of the ratio of the force in the direction perpendicular to the axis acting on the heads of a model pile to the reference pile. Next, select a numerical table coinciding with the condition from **Tables 3.4.4** to **3.4.7** and search for a row corresponding to the previously calculated log T_S. Then, read a log $y_{t,s}$ of the displacement of the reference pile head on the same row. The displacement of a model pile head $y_{t,p}$ can be calculated from this log $y_{t,s}$ and the log R_y of the ratio of displacement of a model pile to the reference pile. To obtain other values, the value for a model pile may also be calculated from the value for the reference pile read from the table and the ratio of a model pile to the reference pile for the value.

There are many cases where no row is shown that coincides with the targeted value (in the above example, when a row coinciding with log T_s is not found) when reading a value from **Tables 3.4.4** to **3.4.7**. In such a case, it may be allowed to select a row corresponding to the next lower and next higher than the targeted value, create a row corresponding to the targeted value by linear interpolation of the values and use the row.

Table 3.4.3 Specifications of the reference pile

Protrusion length h_s (m)	1.0
Bending stiffness $(EI)s$ (kN·m^2)	10,000
Pile width Bs (m)	0.5
S-type ground ks,s (kN/m$^{3.5}$)	2,000
C-type ground kc,s (kN/m$^{2.5}$)	2,000

Table 3.4.4 Reference Curve of a Free Head Pile on the S-type Ground

$\log T_s$	$\log y_{t,s}$	$\log M_{\max,s}$	$\log l_{m1,s}$	$\log y_{0,s}$	$\log i_{t,s}$	$\log i_{0,s}$
13.0	14.1219	14.5236	2.1062	14.1139	12.3820	12.3819
12.5	13.4108	13.9540	2.0348	13.4014	11.7416	11.7415
12.0	12.7003	13.3847	1.9634	12.6892	11.1016	11.1014
11.5	11.9905	12.8158	1.8919	11.9774	10.4621	10.4619
11.0	11.2814	12.2474	1.8205	11.2660	9.8232	9.8229
10.5	10.5733	11.6795	1.7491	10.5551	9.1849	9.1845
10.0	9.8662	11.1122	1.6777	9.8448	8.5475	8.5469
9.5	9.1604	10.5455	1.6063	9.1352	7.9110	7.9101
9.0	8.4560	9.9797	1.5349	8.4263	7.2755	7.2743
8.5	7.7533	9.4148	1.4635	7.7184	6.6413	6.6397
8.0	7.0525	8.8510	1.3922	7.0115	6.0085	6.0064
7.5	6.3540	8.2884	1.3208	6.3057	5.3774	5.3745
7.0	5.6581	7.7272	1.2495	5.6013	4.7481	4.7442
6.5	4.9653	7.1676	1.1782	4.8984	4.1210	4.1158
6.0	4.2758	6.6098	1.1069	4.1973	3.4963	3.4894
5.5	3.5902	6.0540	1.0357	3.4981	2.8744	2.8652
5.0	2.9090	5.5005	0.9645	2.8010	2.2556	2.2434
4.5	2.2327	4.9494	0.8935	2.1063	1.6403	1.6242
4.0	1.5619	4.4009	0.8225	1.4142	1.0286	1.0078
3.5	0.8972	3.8553	0.7516	0.7248	0.4212	0.3944
3.0	0.2391	3.3128	0.6809	0.0385	-0.1817	-0.2161
2.5	-0.4119	2.7735	0.6104	-0.6447	-0.7799	-0.8234
2.0	-1.0552	2.2374	0.5401	-1.3245	-1.3730	-1.4275
1.5	-1.6904	1.7047	0.4700	-2.0010	-1.9607	-2.0285
1.0	-2.3173	1.1752	0.4002	-2.6740	-2.5430	-2.6263
0.5	-2.9355	0.6490	0.3307	-3.3434	-3.1197	-3.2211
0.0	-3.5450	0.1259	0.2616	-4.0094	-3.6907	-3.8129
-0.5	-4.1458	-0.3942	0.1928	-4.6719	-4.2560	-4.4018
-1.0	-4.7381	-0.9116	0.1245	-5.3311	-4.8160	-4.9881
-1.5	-5.3221	-1.4265	0.0565	-5.9871	-5.3705	-5.5720
-2.0	-5.8980	-1.9392	-0.0110	-6.6401	-5.9200	-6.1535
-2.5	-6.4664	-2.4499	-0.0782	-7.2902	-6.4646	-6.7329
-3.0	-7.0277	-2.9589	-0.1449	-7.9376	-7.0046	-7.3103
-3.5	-7.5824	-3.4663	-0.2113	-8.5827	-7.5404	-7.8860
-4.0	-8.1310	-3.9725	-0.2772	-9.2254	-8.0723	-8.4601
-4.5	-8.6742	-4.4776	-0.3428	-9.8662	-8.6006	-9.0329
-5.0	-9.2123	-4.9818	-0.4081	-10.5051	-9.1257	-9.6043
-5.5	-9.7459	-5.4853	-0.4731	-11.1423	-9.6478	-10.1747
-6.0	-10.2755	-5.9881	-0.5378	-11.7781	-10.1673	-10.7441
-6.5	-10.8014	-6.4903	-0.6023	-12.4125	-10.6844	-11.3125
-7.0	-11.3241	-6.9922	-0.6665	-13.0458	-11.1995	-11.8803
-7.5	-11.8440	-7.4937	-0.7305	-13.6780	-11.7126	-12.4473
-8.0	-12.3614	-7.9949	-0.7944	-14.3094	-12.2241	-13.0138
-8.5	-12.8766	-8.4959	-0.8580	-14.9399	-12.7342	-13.5797
-9.0	-13.3898	-8.9967	-0.9216	-15.5697	-13.2429	-14.1452
-9.5	-13.9013	-9.4973	-0.9850	-16.1989	-13.7506	-14.7103
-10.0	-14.4113	-9.9978	-1.0483	-16.8275	-14.2572	-15.2751
-10.5	-14.9200	-10.4983	-1.1114	-17.4557	-14.7630	-15.8396
-11.0	-15.4276	-10.9986	-1.1745	-18.0834	-15.2680	-16.4038
-11.5	-15.9344	-11.4989	-1.2376	-18.7109	-15.7726	-16.9678

Table 3.4.5 Reference Curve of a Fixed Head Pile on the S-type Ground

$\log T_s$	$\log y_{t,s}$	$\log M_{\text{top},s}$	$\log l_{m1,s}$	$\log y_{0,s}$	$\log M_{\max,s}$	$\log i_{0,s}$
13.0	13.5685	14.5204	2.1178	13.5683	14.0436	10.5138
12.5	12.8569	13.9502	2.0463	12.8566	13.4738	9.9425
12.0	12.1457	13.3802	1.9748	12.1453	12.9043	9.3711
11.5	11.4350	12.8105	1.9032	11.4345	12.3351	8.7997
11.0	10.7251	12.2410	1.8316	10.7244	11.7663	8.2284
10.5	10.0158	11.6719	1.7600	10.0148	11.1980	7.6571
10.0	9.3075	11.1032	1.6884	9.3061	10.6303	7.0857
9.5	8.6002	10.5350	1.6168	8.5984	10.0632	6.5145
9.0	7.8941	9.9673	1.5451	7.8916	9.4969	5.9432
8.5	7.1895	9.4002	1.4733	7.1861	8.9314	5.3720
8.0	6.4865	8.8338	1.4016	6.4819	8.3670	4.8008
7.5	5.7855	8.2683	1.3297	5.7793	7.8038	4.2296
7.0	5.0869	7.7037	1.2579	5.0784	7.2420	3.6585
6.5	4.3908	7.1403	1.1859	4.3794	6.6818	3.0875
6.0	3.6979	6.5780	1.1139	3.6826	6.1235	2.5165
5.5	3.0085	6.0172	1.0418	2.9881	5.5673	1.9456
5.0	2.3232	5.4579	0.9697	2.2962	5.0136	1.3748
4.5	1.6426	4.9005	0.8975	1.6071	4.4627	0.8041
4.0	0.9673	4.3449	0.8252	0.9209	3.9148	0.2336
3.5	0.2979	3.7914	0.7529	0.2377	3.3704	-0.3368
3.0	-0.3649	3.2403	0.6806	-0.4421	2.8297	-0.9069
2.5	-1.0204	2.6916	0.6084	-1.1186	2.2931	-1.4769
2.0	-1.6679	2.1456	0.5361	-1.7917	1.7607	-2.0466
1.5	-2.3071	1.6023	0.4640	-2.4614	1.2326	-2.6161
1.0	-2.9374	1.0617	0.3921	-3.1277	0.7088	-3.1852
0.5	-3.5584	0.5241	0.3204	-3.7906	0.1894	-3.7541
0.0	-4.1701	-0.0106	0.2491	-4.4502	-0.3290	-4.3225
-0.5	-4.7724	-0.5425	0.1781	-5.1066	-0.8376	-4.8906
-1.0	-5.3654	-1.0716	0.1076	-5.7601	-1.3459	-5.4584
-1.5	-5.9495	-1.5980	0.0375	-6.4107	-1.8514	-6.0257
-2.0	-6.5251	-2.1218	-0.0320	-7.0587	-2.3544	-6.5926
-2.5	-7.0927	-2.6431	-0.1010	-7.7042	-2.8554	-7.1592
-3.0	-7.6529	-3.1623	-0.1694	-8.3474	-3.3550	-7.7253
-3.5	-8.2062	-3.6793	-0.2373	-8.9886	-3.8534	-8.2912
-4.0	-8.7534	-4.1944	-0.3047	-9.6279	-4.3510	-8.8566
-4.5	-9.2949	-4.7078	-0.3716	-10.2655	-4.8481	-9.4218
-5.0	-9.8314	-5.2196	-0.4380	-10.9016	-5.3449	-9.9867
-5.5	-10.3634	-5.7300	-0.5040	-11.5364	-5.8415	-10.5513
-6.0	-10.8914	-6.2391	-0.5696	-12.1699	-6.3381	-11.1156
-6.5	-11.4159	-6.7471	-0.6348	-12.8024	-6.8347	-11.6798
-7.0	-11.9373	-7.2541	-0.6997	-13.4340	-7.3315	-12.2437
-7.5	-12.4559	-7.7602	-0.7643	-14.0647	-7.8285	-12.8075
-8.0	-12.9721	-8.2656	-0.8286	-14.6947	-8.3257	-13.3711
-8.5	-13.4862	-8.7702	-0.8928	-15.3240	-8.8230	-13.9345
-9.0	-13.9985	-9.2743	-0.9567	-15.9528	-9.3206	-14.4979
-9.5	-14.5092	-9.7778	-1.0204	-16.5811	-9.8185	-15.0611
-10.0	-15.0185	-10.2809	-1.0840	-17.2089	-10.3165	-15.6243
-10.5	-15.5266	-10.7835	-1.1474	-17.8364	-10.8147	-16.1873
-11.0	-16.0336	-11.2859	-1.2107	-18.4636	-11.3131	-16.7503
-11.5	-16.5396	-11.7879	-1.2739	-19.0905	-11.8117	-17.3132

Table 3.4.6 Reference Curve of a Free Head Pile on the C-type Ground

$\log T_s$	$\log y_{t,s}$	$\log M_{\max,s}$	$\log l_{m1,s}$	$\log y_{0,s}$	$\log i_{t,s}$	$\log i_{0,s}$
13.0	15.7181	14.9153	2.7519	15.7161	13.3980	13.3980
12.5	14.9194	14.3162	2.6519	14.9168	12.6991	12.6991
12.0	14.1211	13.7174	2.5518	14.1178	12.0005	12.0005
11.5	13.3231	13.1190	2.4518	13.3189	11.3023	11.3023
11.0	12.5257	12.5209	2.3517	12.5204	10.6046	10.6045
10.5	11.7289	11.9233	2.2516	11.7223	9.9074	9.9073
10.0	10.9330	11.3263	2.1515	10.9246	9.2109	9.2107
9.5	10.1380	10.7301	2.0514	10.1276	8.5153	8.5151
9.0	9.3445	10.1348	1.9512	9.3313	7.8209	7.8205
8.5	8.5525	9.5407	1.8510	8.5358	7.1277	7.1272
8.0	7.7625	8.9480	1.7508	7.7416	6.4362	6.4354
7.5	6.9751	8.3572	1.6505	6.9487	5.7469	5.7457
7.0	6.1909	7.7685	1.5501	6.1575	5.0602	5.0583
6.5	5.4105	7.1825	1.4497	5.3685	4.3766	4.3737
6.0	4.6349	6.5997	1.3493	4.5819	3.6968	3.6924
5.5	3.8652	6.0207	1.2488	3.7984	3.0215	3.0149
5.0	3.1026	5.4462	1.1483	3.0185	2.3516	2.3418
4.5	2.3485	4.8768	1.0479	2.2427	1.6880	1.6737
4.0	1.6046	4.3132	0.9476	1.4719	1.0317	1.0110
3.5	0.8724	3.7560	0.8475	0.7065	0.3836	0.3542
3.0	0.1536	3.2055	0.7477	-0.0529	-0.2554	-0.2964
2.5	-0.5503	2.6621	0.6484	-0.8056	-0.8845	-0.9406
2.0	-1.2376	2.1256	0.5498	-1.5512	-1.5030	-1.5783
1.5	-1.9083	1.5955	0.4520	-2.2898	-2.1108	-2.2100
1.0	-2.5612	1.0715	0.3552	-3.0210	-2.7076	-2.8355
0.5	-3.1968	0.5527	0.2595	-3.7451	-3.2937	-3.4555
0.0	-3.8155	0.0384	0.1650	-4.4622	-3.8694	-4.0703
-0.5	-4.4188	-0.4724	0.0717	-5.1730	-4.4356	-4.6806
-1.0	-5.0076	-0.9803	-0.0204	-5.8778	-4.9927	-5.2867
-1.5	-5.5834	-1.4861	-0.1114	-6.5772	-5.5419	-5.8893
-2.0	-6.1479	-1.9903	-0.2013	-7.2718	-6.0838	-6.4888
-2.5	-6.7021	-2.4932	-0.2903	-7.9621	-6.6192	-7.0856
-3.0	-7.2482	-2.9953	-0.3784	-8.6489	-7.1494	-7.6803
-3.5	-7.7867	-3.4967	-0.4658	-9.3324	-7.6748	-8.2730
-4.0	-8.3189	-3.9977	-0.5526	-10.0133	-8.1960	-8.8641
-4.5	-8.8459	-4.4984	-0.6388	-10.6918	-8.7138	-9.4540
-5.0	-9.3683	-4.9989	-0.7245	-11.3684	-9.2287	-10.0427
-5.5	-9.8869	-5.4993	-0.8099	-12.0433	-9.7410	-10.6305
-6.0	-10.4025	-5.9995	-0.8949	-12.7168	-10.2513	-11.2175
-6.5	-10.9153	-6.4997	-0.9796	-13.3892	-10.7599	-11.8039
-7.0	-11.4260	-6.9998	-1.0641	-14.0606	-11.2670	-12.3898
-7.5	-11.9348	-7.4998	-1.1484	-14.7312	-11.7729	-12.9752
-8.0	-12.4422	-7.9999	-1.2325	-15.4011	-12.2777	-13.5603
-8.5	-12.9482	-8.4999	-1.3165	-16.0705	-12.7818	-14.1450
-9.0	-13.4533	-8.9999	-1.4004	-16.7394	-13.2851	-14.7295
-9.5	-13.9574	-9.5000	-1.4842	-17.4079	-13.7879	-15.3138
-10.0	-14.4608	-10.0000	-1.5679	-18.0761	-14.2902	-15.8980
-10.5	-14.9637	-10.5000	-1.6515	-18.7440	-14.7921	-16.4820
-11.0	-15.4660	-11.0000	-1.7351	-19.4117	-15.2936	-17.0659
-11.5	-15.9682	-11.5000	-1.8187	-20.0793	-15.7951	-17.6497

Table 3.4.7 Reference Curve of a Fixed Head Pile on the C-type Ground

$\log T_s$	$\log y_{t,s}$	$\log M_{\mathrm{top},s}$	$\log l_{m1,s}$	$\log y_{0,s}$	$\log M_{\max,s}$	$\log i_{0,s}$
13.0	15.2757	15.0193	2.7926	15.2757	14.4285	11.0171
12.5	14.4766	14.4199	2.6925	14.4765	13.8292	10.4172
12.0	13.6775	13.8205	2.5924	13.6774	13.2298	9.8172
11.5	12.8787	13.2215	2.4923	12.8786	12.6308	9.2173
11.0	12.0803	12.6226	2.3921	12.0802	12.0319	8.6174
10.5	11.2822	12.0240	2.2919	11.2821	11.4334	8.0175
10.0	10.4847	11.4259	2.1917	10.4845	10.8353	7.4176
9.5	9.6878	10.8281	2.0913	9.6875	10.2376	6.8178
9.0	8.8918	10.2310	1.9909	8.8913	9.6406	6.2180
8.5	8.0968	9.6345	1.8904	8.0961	9.0443	5.6183
8.0	7.3031	9.0389	1.7898	7.3020	8.4491	5.0186
7.5	6.5111	8.4445	1.6890	6.5093	7.8551	4.4190
7.0	5.7213	7.8513	1.5880	5.7185	7.2627	3.8196
6.5	4.9340	7.2597	1.4867	4.9297	6.6723	3.2202
6.0	4.1502	6.6701	1.3852	4.1435	6.0844	2.6210
5.5	3.3708	6.0828	1.2833	3.3605	5.4998	2.0220
5.0	2.5969	5.4983	1.1810	2.5811	4.9194	1.4233
4.5	1.8299	4.9171	1.0783	1.8061	4.3440	0.8248
4.0	1.0717	4.3395	0.9750	1.0361	3.7751	0.2266
3.5	0.3240	3.7662	0.8713	0.2716	3.2139	-0.3711
3.0	-0.4109	3.1975	0.7671	-0.4867	2.6617	-0.9684
2.5	-1.1304	2.6339	0.6626	-1.2381	2.1200	-1.5650
2.0	-1.8327	2.0756	0.5580	-1.9825	1.5893	-2.1610
1.5	-2.5157	1.5230	0.4538	-2.7193	1.0699	-2.7562
1.0	-3.1789	0.9760	0.3504	-3.4487	0.5606	-3.3506
0.5	-3.8217	0.4344	0.2481	-4.1707	0.0598	-3.9439
0.0	-4.4450	-0.1019	0.1475	-4.8858	-0.4345	-4.5362
-0.5	-5.0501	-0.6332	0.0487	-5.5943	-0.9246	-5.1275
-1.0	-5.6390	-1.1602	-0.0483	-6.2968	-1.4124	-5.7177
-1.5	-6.2135	-1.6831	-0.1433	-6.9941	-1.8994	-6.3069
-2.0	-6.7758	-2.2025	-0.2367	-7.6866	-2.3866	-6.8952
-2.5	-7.3277	-2.7189	-0.3285	-8.3752	-2.8746	-7.4827
-3.0	-7.8708	-3.2327	-0.4189	-9.0602	-3.3637	-8.0696
-3.5	-8.4066	-3.7442	-0.5081	-9.7423	-3.8541	-8.6558
-4.0	-8.9363	-4.2539	-0.5963	-10.4218	-4.3457	-9.2416
-4.5	-9.4609	-4.7619	-0.6838	-11.0992	-4.8385	-9.8269
-5.0	-9.9813	-5.2686	-0.7705	-11.7748	-5.3323	-10.4119
-5.5	-10.4983	-5.7742	-0.8566	-12.4489	-5.8271	-10.9966
-6.0	-11.0123	-6.2788	-0.9422	-13.1217	-6.3227	-11.5810
-6.5	-11.5239	-6.7826	-1.0275	-13.7935	-6.8191	-12.1653
-7.0	-12.0335	-7.2858	-1.1124	-14.4644	-7.3160	-12.7494
-7.5	-12.5414	-7.7885	-1.1970	-15.1346	-7.8134	-13.3334
-8.0	-13.0480	-8.2906	-1.2814	-15.8042	-8.3113	-13.9172
-8.5	-13.5534	-8.7924	-1.3656	-16.4732	-8.8095	-14.5010
-9.0	-14.0579	-9.2939	-1.4497	-17.1419	-9.3081	-15.0847
-9.5	-14.5616	-9.7952	-1.5336	-17.8102	-9.8069	-15.6683
-10.0	-15.0647	-10.2962	-1.6175	-18.4782	-10.3058	-16.2519
-10.5	-15.5672	-10.7970	-1.7012	-19.1460	-10.8050	-16.8354
-11.0	-16.0693	-11.2977	-1.7849	-19.8136	-11.3043	-17.4189
-11.5	-16.5711	-11.7983	-1.8685	-20.4811	-11.8037	-18.0024

(4) Several values for a ground level loading model pile the head of which coincides with the ground level (pile protrusion length = 0) can be calculated by **squations (3.4.44)** to **(3.4.47)** [53)] .

S-type ground, free head pile

$$\left.\begin{aligned}\log y_0 &= 0.38958 - \frac{4}{7}\log EI - \frac{6}{7}\log Bk_S + \frac{10}{7}\log T\\ \log M_{\max} &= -0.05825 + \frac{1}{7}\log EI - \frac{2}{7}\log Bk_S + \frac{8}{7}\log T\\ \log i_0 &= 0.22539 - \frac{5}{7}\log EI - \frac{4}{7}\log Bk_S + \frac{9}{7}\log T\\ \log l_{m1} &= 0.53473 + \frac{1}{7}\log EI - \frac{2}{7}\log Bk_S + \frac{1}{7}\log T\end{aligned}\right\} \quad \textbf{(3.4.44)}$$

S-type ground, fixed head pile

$$\left.\begin{aligned}\log y_0 &= -0.16047 - \frac{4}{7}\log EI - \frac{6}{7}\log Bk_S + \frac{10}{7}\log T\\ \log M_0 &= -0.05787 + \frac{1}{7}\log EI - \frac{2}{7}\log Bk_S + \frac{8}{7}\log T\\ \log M_{\max} &= -0.53703 + \frac{1}{7}\log EI - \frac{2}{7}\log Bk_S + \frac{8}{7}\log T\\ \log l_{m1} &= 0.54689 + \frac{1}{7}\log EI - \frac{2}{7}\log Bk_S + \frac{1}{7}\log T\end{aligned}\right\} \quad \textbf{(3.4.45)}$$

C-type ground, free head pile

$$\left.\begin{aligned}\log y_0 &= 0.11328 - \frac{2}{5}\log EI - \frac{6}{5}\log Bk_C + \frac{8}{5}\log T\\ \log M_{\max} &= -0.28846 + \frac{1}{5}\log EI - \frac{2}{5}\log Bk_C + \frac{6}{5}\log T\\ \log i_0 &= -0.00634 - \frac{3}{5}\log EI - \frac{4}{5}\log Bk_C + \frac{7}{5}\log T\\ \log l_{m1} &= 0.55205 + \frac{1}{5}\log EI - \frac{2}{5}\log Bk_C + \frac{1}{5}\log T\end{aligned}\right\} \quad \textbf{(3.4.46)}$$

C-type ground, fixed head pile

$$\left.\begin{aligned}\log y_0 &= -0.32731 - \frac{2}{5}\log EI - \frac{6}{5}\log Bk_C + \frac{8}{5}\log T\\ \log M_0 &= -0.18301 + \frac{1}{5}\log EI - \frac{2}{5}\log Bk_C + \frac{6}{5}\log T\\ \log M_{\max} &= -0.77377 + \frac{1}{5}\log EI - \frac{2}{5}\log Bk_C + \frac{6}{5}\log T\\ \log l_{m1} &= 0.59296 + \frac{1}{5}\log EI - \frac{2}{5}\log Bk_C + \frac{1}{5}\log T\end{aligned}\right\} \quad \textbf{(3.4.47)}$$

where

y_0 : displacement of a pile on the ground level (m)

M_0 : bending moment caused in the pile body on the ground level (kN·m)

i_0 : angle of deflection of a pile on the ground level (rad)

M_{max} : maximum bending moment below the ground level (kN·m)

l_{m1} : depth of bending moment first zero point of a free head pile or depth of bending moment second zero point of a fixed head pile (m)

EI : bending stiffness of a pile (kN·m^2)

B : width of a pile (m)

k_S : lateral resistance coefficient in S-type ground ($kN/m^{3.5}$)

k_C : lateral resistance coefficient in C-type ground ($kN/m^{2.5}$)

T :force in the direction perpendicular to the axis acting on the pile head (kN)

(5) Relation between the lateral resistance coefficient in the S-type ground and the increment of SPT-N values per unit depth as in **Fig. 3.4.6** has been shown [54)]. Here, the increment of SPT-N values per unit depth means the inclination of a line approximating the distribution of SPT-N values in depth direction obtained from a ground exploration. The increment of SPT-N values in the range from the ground level to $(0.5–1.0)l_{m1}$ which greatly influence the lateral resistance of piles is generally used. Even when the distribution of SPT-N values in depth direction is not 0 on the ground level, the inclination approximating with a line passing 0 on the ground level may be used.

The line in **Fig. 3.4.6** is a regression line obtained by the least squares method and is expressed by **equation (3.4.48)** [55)].

$$k_S = 592\,\bar{N}^{0.654} \qquad \textbf{(3.4.48)}$$

where

$\bar{N}$: increment of SPT-N values per unit depth (m^{-1})

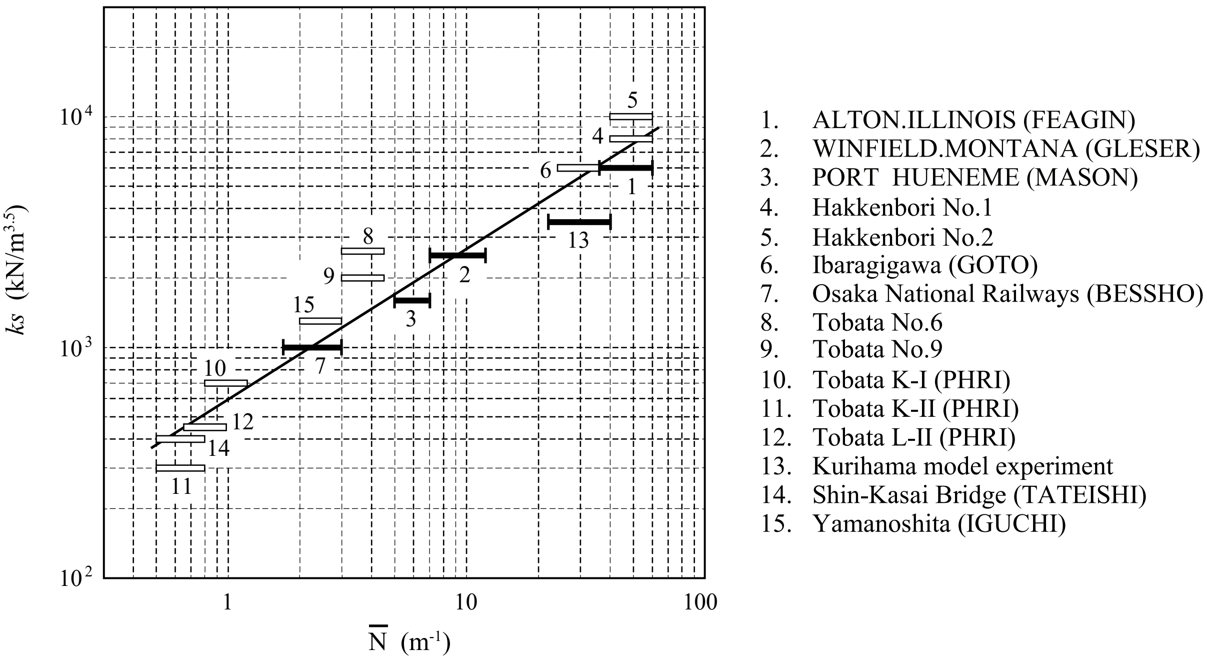


Fig. 3.4.6 Relation between lateral resistance coefficient in S-type ground and increment of SPT-N values $\bar{N}$ per Unit Depth (Kubo [54)], added and altered)

(6) Relation between the lateral resistance coefficient in the C-type ground and SPT-N value of the ground as in **Fig. 3.4.7** has been shown [56)]. Mean SPT-N values in the range from the ground level to $(0.5–1.0)l_{m1}$ which greatly influence the lateral resistance of piles are generally used for SPT-N values.

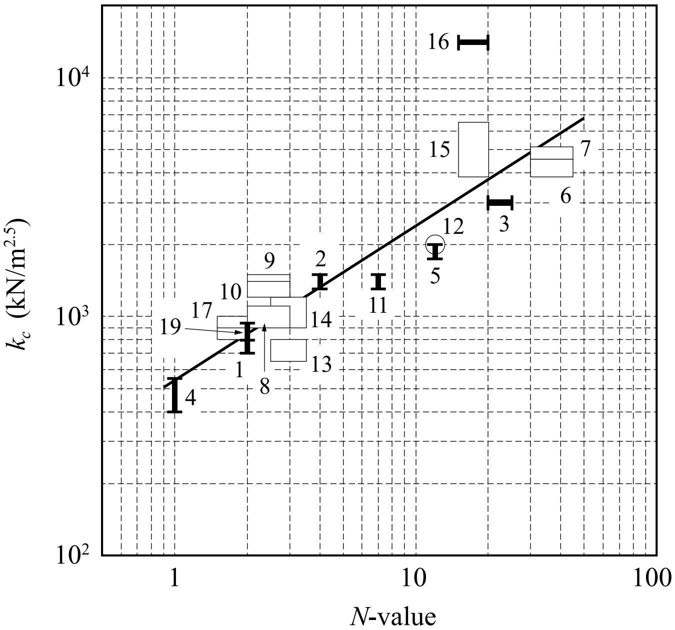
The line in **Fig. 3.4.7** is a regression line obtained by the least squares method and is expressed by **Equation (3.4.49)** [55)].

$$k_C = 540\,N^{0.648} \qquad \textbf{(3.4.49)}$$

where

N : mean SPT-N value in a dominant range of lateral resistant of piles.

If no SPT-N value of clayey ground is obtained from a ground exploration, it is difficult to estimate an SPT-N value from the strength of unconfined compression. Experimental conversion formula and others are often used for estimation of strength of unconfined compression from SPT-N values. Care needs to be taken in using the conversion formula in reverse direction for estimation of SPT-N values from the strength of unconfined compression since it lacks sufficient reliability and leads to estimation on the dangerous side. **Fig. 3.4.7** was prepared by using the data which directly estimated SPT-N values on the clayey ground by ground exploration and others.



1. Tobata K-I (TTRI)
2. Tobata K-III (TTRI)
3. Tobata K-IV (TTRI)
4. Tobata L-II (TTRI)
5. Tobata L-IV (TTRI)
6. Hakkenbori No.1
7. Hakkenbori No.2
8. Osaka National Railways
9. Yahata Seitetsu No.6
10. Yahata Seitetsu No.9
11. Tobata preliminary test-1 (TTRI)-1
12. Tobata preliminary test-2 (TTRI)-2
13. WAGNER (Calif.) No. 15
14. WAGNER (Calif.) No. 25
15. WAGNER (Alaska) -1
16. WAGNER (Alaska) -2
17. Tokyo National Railways b
18. Tokyo National Railways A4
19. Tokyo National Railways B

Fig. 3.4.7 Relation between lateral resistance coefficient in the C-type ground and SPT-N values (Port and Harbour Technical Research Institute and Yawata Iron & Steel Co., Ltd. [56]), added and altered)

(7) The following relation expressed by **equations (3.4.50)** and **(3.4.51)** has been reported between cohesion and lateral resistance coefficient of clayey ground from the result of field experiment in clayey ground and others [57]).

$$k_S = \alpha \sqrt{\frac{2\pi}{\varepsilon_{50} B}} \quad \textbf{(3.4.50)}$$

$$k_C = c_u \sqrt{\frac{2\pi}{\varepsilon_{50} B}} \quad \textbf{(3.4.51)}$$

where

α : increment of c_u per unit depth (kN/m^3)

c_u : undrained shear strength obtained from unconfined compression test (kN/m^2)
$c_u = q_u/2$

q_u : unconfined compression strength (kN/m^2)

ε_{50} : strain at $q_u/2$ in unconfined compression test

B : width of a pile (m)
The upper limit of B is 0.3 m.

(8) The behavior of piles is classified by embedment length as shown in **Table 3.4.8** as a result of examination of the variance in behavior of piles based on model experiment [58]). As seen from the behavioral characteristics shown in

Table 3.4.8, the effective length of piles is considered $1.5l_{m1}$. l_{m1} is the depth of bending moment first zero point of a free head pile or depth of bending moment second zero point of a fixed head pile.

l_{m1} generally tends to increase as the stiffness of a pile increases and tends to decrease as the lateral resistance coefficient of the ground increases. On the other hand, the height of load or securing conditions of pile heads do not influence much. Here, care needs to be taken that the value of l_{m1} increases as load increases. In other words, behavioral characteristic of the same pile may vary as the load changes.

Table 3.4.8 shows that the behavior of piles, even if they are short, is not different from that of long piles provided that their embedment length is $1.0lm1$ or more. However, it has been noted that more residual displacement accumulates than in the case of long piles when repeated load acts on short piles. Moreover, it is known that short piles are prone to be affected by soil creep and others. Therefore, the embedment length of piles should ensure $1.5lm1$ or more.

Table 3.4.8 Classification of the Behavior of Piles by Embedment Length

Classification		Embedment Length	Behavior characteristics
Long pile		$1.5lm1$ or more	The lower edge of a pile is fixed in the ground. The embedment length of a pile is irrelevant to its behavior.
Short pile	First transient area	$1.0lm1$–$1.5lm1$	The lower edge of a pile is incompletely fixed, but the behavior of a pile is same as that of a long pile.
	Second transient area	$0.6lm1$–$1.0lm1$	Displacement and inclination are significantly larger than those of a long pile. The pile bends extremely.
	Stiff pile	Less than $0.6lm1$	Bend of a pile is negligible. The pile moves almost rotationally.

(9) The relation between the lateral resistance coefficient and the width of piles in sandy S-type ground found in a model experiment is shown in **Fig. 3.4.8** [59) 60)]. **Fig. 3.4.8** shows that the lateral resistance coefficient reduces as the width of piles increases while the width of piles is small. On the other hand, when the width of piles exceeds 0.3 m, the lateral resistance coefficient seems independent to the width of piles. Therefore, the influence of the width of piles is generally not considered when estimating the deflection of piles by the PHRI method.

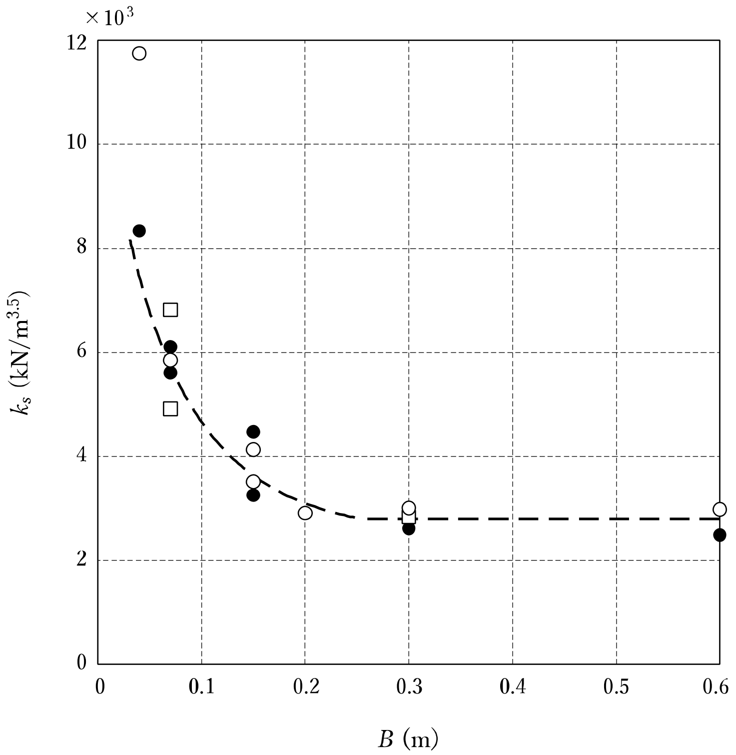


Fig. 3.4.8 Relation between lateral resistance coefficient in the S-type ground and the width of piles (Sawaguchi [60)], added and altered)

(10) The resistance force of a batter pile in the direction perpendicular to the axis varies according to the angle of a pile's inclination. The resistance force of a pile in the direction perpendicular to the axis generally tends to become smaller when load acts in the direction to raise a pile. Conversely, it tends to become larger when load acts in the direction to push a pile to the ground. Therefore, the influence of the angle of a pile's inclination shall be considered by correcting the lateral resistance coefficient of the ground in calculating the deflection of batter piles. **Fig. 3.4.9** shows the relation between the angle of a pile's inclination and the ratio of the lateral resistance coefficient of the ground. Here, the ratio of lateral resistance coefficient of the ground means that of the lateral resistance coefficient used for calculation of batter piles to the lateral resistance coefficient used for calculation of vertical piles.

When the surrounding ground is reclaimed after construction of batter piles or in other occasions, and the ground around the piles has not been sufficiently compacted yet, care needs to be taken since the resistance force of a pile in the direction perpendicular to the axis does not grow even if the angle of inclination is negative. For the case of coupled piles, see **Part III, Chapter 2, 3.4.9 (3)**.

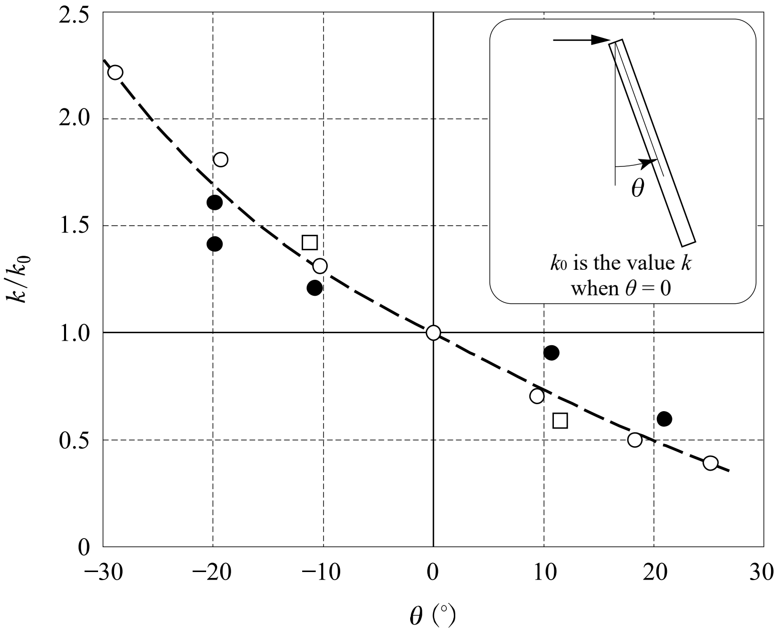


Fig. 3.4.9 Ratio of angle of pile's inclination to the lateral resistance coefficient of the ground (Kubo [61]), added and altered)

3.4.9 Bearing Capacity of Coupled Piles

(1) The bearing capacity of coupled piles is examined by resolving the vertical load and the horizontal load acting on the head of coupled piles into two elements in the axial direction of two piles composing coupled piles or into four elements in the axial direction and the direction perpendicular to the axis.

It is known that when the displacement of pile heads of coupled piles is small, most load acting on the pile heads of the coupled pile acts as the force in the axial direction of two piles composing the coupled pile [62]). Therefore, it is reasonable to verify in the method to resolve the load to two elements for coupled piles designed considering enough safety margin. On the other hand, when assuming a certain amount of displacement in structures like in examination of L2 earthquake, care needs to be taken since resolving the load into two elements may underestimate the bearing capacity of coupled piles.

(2) When resolving the load acting on the pile heads of coupled piles into two elements in the axial direction of the two piles composing the coupled piles, the force in the axial direction acting on the head of each pile can be calculated by **equation (3.4.52)** (see **Fig. 3.4.10**).

$$\left.\begin{aligned} P_1 &= \frac{V_i \sin\theta_2 + H_i \cos\theta_2}{\sin(\theta_1+\theta_2)} \\ P_2 &= \frac{V_i \sin\theta_1 - H_i \cos\theta_1}{\sin(\theta_1+\theta_2)} \end{aligned}\right\} \qquad \textbf{(3.4.52)}$$

where

P_1, P_2 : force in the axial direction acting on the head of each pile (pushing force is defined as positive) (kN)

θ_1, θ_2 : angle of inclination of each pile (°)

V_i : vertical force acting on coupled piles (kN)

H_i : horizontal force acting on coupled piles (kN)

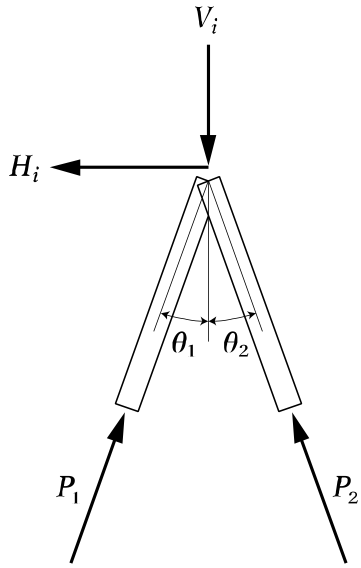


Fig. 3.4.10 Resolution of load acting on coupled piles into elements in the axial direction

The bearing capacity of coupled piles shall be verified by confirming if each of the two piles composing the coupled piles has enough bearing capacity for force in the axial direction acting on the pile head. For the verification method, see **3.4.2 (6)** in this Chapter.

It is difficult to determine the displacement of heads of coupled piles in the method to resolve and examine the load into two elements. Sufficient examination is necessary when the displacement of pile heads becomes an important issue in the design of structures. However, the displacement of coupled piles is generally much smaller than that of single piles, and it may become a problem in a few cases.

(3) When resolving the load into four elements, the behavior (displacement of pile heads, deflection of each pile, etc.) of each pile greatly influences the bearing capacity of coupled piles as in the case of single piles on which the force in the direction perpendicular to the axis acts. Therefore, verification needs to be performed after analytically estimating the behavior of each pile.

Analysis methods such as ① to analyze under the condition that displacement of each pile coincides at the intersection of coupled piles assuming that the spring of pile heads in the axial direction and the direction perpendicular to the axis are elastic [63)], ② to determine the ultimate resistance of coupled piles assuming that the resistance force of piles in the axial direction and the direction perpendicular to the axis shows elasto-plastic nature [64)], ③ to calculate the load, displacement, settlement, or upward displacement by pulling of pile heads with an empirical formula assuming that the pile shows nonlinear behavior in both axial direction and the direction perpendicular to the axis [65)], and ④ to utilize the result of loading test of single piles [66)] have been proposed and can be referred to.

The setting method of constants, such as modulus of subgrade reaction, becomes an important issue in analysis using these methods. Specifically, the effect of evaluation of the ground properties between piles to the result of analysis needs to be fully confirmed since the subgrade reaction in the ground between two piles composing the coupled piles may not be fully expected. Moreover, since the connection condition of two pile heads composing the coupled piles is known to significantly influence, it is also necessary to pay attention to its handling. Moreover, these analysis methods assume that displacement occurs in pile heads. Multifaceted verification is required whether the displacement of pile heads of coupled piles analytically calculated is reasonable for the displacement of the entire pile foundation, whether it conforms to the displacement of pile heads of other piles composing the pile foundation, etc.

(4) The method to increase the bearing capacity of coupled piles is under research by improving the ground between piles since the behavior of the ground between piles greatly influences the bearing capacity of coupled piles [67)].

3.4.10 Bearing Capacity of a Pile Group

(1) When arranging several piles densely in close interval, they tend to behave as one group of piles because of overlapping of stress transmitted from each pile to the ground, and others. The behavior as a group of piles represents a different characteristic from the bearing capacity, settlement, and others of a single pile. These phenomena are called pile group effects. A group of piles arranged to exert pile group effects is called a pile group in contrast to a single pile. When there are pile group effects, it is necessary to consider the behavior of a pile group separately from the one of a single pile when verifying the pile foundations.

Note that a group of piles may generally and simply be called a pile group even if it has no pile group effect.

(2) The pile group effect is largely affected by the ground condition around and below piles. Specifically, when the ground condition changes abruptly around and below piles, the pile group effect may become noticeable.

For example, as shown in **Fig. 3.4.11**, consider a situation where piles are embedded in sandy layer of good quality but there is a soft clayey layer below it. In the case of a single pile, the range of stress caused in the ground by load acted on a pile and transmitted to the ground remains within the sandy layer of good quality. Then, the clayey layer existing below the sandy layer does not greatly affect the behavior of piles. Whereas, even if the load acting on a pile in a pile group is comparative with the case of a single pile, the range of stress caused in the ground extends much deeper. Thus, the behavior of the pile group is dominated by the clayey layer below and will show a tendency substantially different from the case of a single pile.

This situation requires the ground exploration in the preliminary survey in both sufficiently broad horizontal range and the depth direction against the dimension of structures and careful examination of the pile group effect.

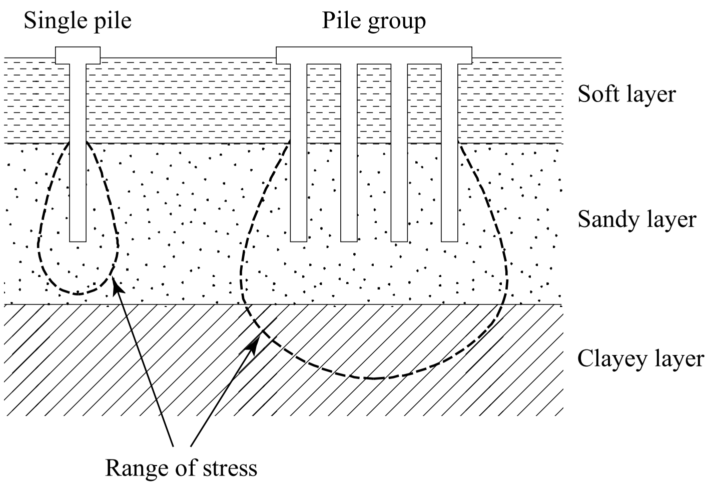


Fig. 3.4.11 Comparison of Ranges Where Stress in the Ground Is Caused in a Single Pile and a Pile Group

(3) If the ground condition around and below piles is relatively homogeneous, the pile group effect against the pushing resistance force of a pile in its axial direction can be considered as follows.

If the interval of bearing piles is normal (see **Part III, Chapter 2, 3.4.12 (1)**), the stress concentration in the bottom bearing stratum is not a problem. Therefore, the pile group effect is not generally considered when determining the pushing resistance force of bearing piles in their axial direction.

The pushing resistance force of friction piles in the axial direction embedded in sandy ground per pile in a group of piles is prone to become greater than that of a single pile due to the compaction effect of the ground by pile driving [68)]. Therefore, the pile group effect becomes an important issue in just a few cases.

On the other hand, in the case of friction piles in the clayey ground, the pushing resistance force of a group of piles in the axial direction may become smaller than the value calculated based on that of a single pile due to the pile group effect. In this case, the behavior as a pile group needs to be fully considered along with the behavior as a single pile.

(4) When a pile subjected to the pulling force in the axial direction is used as a group of piles, the pulling resistance force of a pile group in the axial direction, together with the pulling resistance force of each pile composing a group of piles in the axial direction as a single pile shall be verified.

(5) Although various research has been done so far about the pile group effect for resistance force in the direction perpendicular to the axis, many uncertainties still remain. As the heterogeneity in the ground or a slight difference in conditions such as fixing of pile heads greatly influence the behavior of piles on which large force in the direction perpendicular to the axis acts, it is more difficult to introduce the pile group effect against the resistance force in the direction perpendicular to the axis to performance verification. Therefore, it is desirable to arrange piles on which the force in the direction perpendicular to the axis acts to ensure enough separation so that no pile group effect becomes effective.

If the ground condition around and below piles is relatively stable, the pile group effect against the resistance force in the direction perpendicular to the axis may not be considered provided that the center interval between piles ensures the values shown in **Table 3.4.9**. The values for sandy soil in **Table 3.4.9** have been determined considering the result of model experiment or compaction effect of the ground by pile driving. Larger values than for sandy soil are set for clayey soil to assure safety as insufficient data is available for clayey soil.

Table 3.4.9 Minimum Interval between Piles for Which No Pile Group Effect on Resistance Force in the Direction Perpendicular to the Axis May Be Considered

	Direction in which the force in the direction perpendicular to the axis acts	Direction perpendicular to action
Sandy soil	2.5 times of the pile diameter	1.5 times of the pile diameter
Clayey soil	4.0 times of the pile diameter	3.0 times of the pile diameter

(6) A concept in which the soil and the pile shown in the shaded area in **Fig. 3.4.12** behave as one united block has been proposed as a method to evaluate the pile group effect [69]. According to this concept, the characteristic value of the pushing resistance force of a pile group in the axial direction can be calculated by **equation (3.4.53)**.

$$R_{gk} = q_{u1k}\, A_g + s_k\, U_g\, L - \gamma'_g\, A_g\, L \tag{3.4.53}$$

where

R_{gk} : characteristic value of the pushing resistance force of a pile group in the axial direction (kN)

q_{u1k} : characteristic value of the bearing capacity on the bottom of a block (kN/m^2)

A_g : base area of a block (m^2)

s_k : mean shear strength of soil contacting a block (kN/m^2)

U_g : circumference of a block (m)

L : embedment length of a pile (m)

γ'_g : mean unit volume weight of an entire block containing pile and soil; calculated from the submerged unit volume weight at or below the groundwater level, from the wet unit volume weight at or above the groundwater level (kN/m^3)

Equation (3.4.53) calculates the pushing resistance force of a pile group in the axial direction by adding the bearing capacity of the bottom of a block and the skin resistance force and then subtracting the self-weight of block. For the bearing capacity on the bottom of a block, see **Part III, Chapter 2, 3.2.2 Bearing Capacity of Foundations on Sandy Ground** and **Part III, Chapter 2, 3.2.3 Bearing Capacity of Foundation on Clayey Ground**.

When verifying the pile foundation and if the pile group effect on the pushing resistance force of a pile in its axial direction needs to be considered, verify both of the pushing resistance force in the axial direction of a pile group and each pile composing a pile group as a single pile. When verifying the pushing resistance force of a pile group in its axial direction, use the design value determined by considering the safety margin based on the characteristic value calculated by **equation (3.4.53)**. For the safety margin to allow for, see **Part III, Chapter 2, 3.4.3 (4)** .

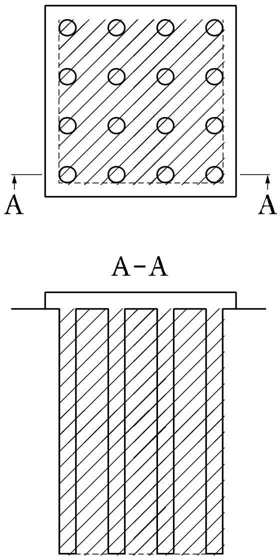


Fig. 3.4.12 A block Comprising a Group of Piles and Soil between the Piles (Shaded Part in the Figure)

(7) The pulling resistance force of a pile group in its axial direction shall be calculated by **equation (3.4.54)** considering that the group of piles and the soil between the piles behave as a block same as in **(6)**.

$$R_{\mathrm{pull},gk} = s_k U_g L + \gamma'_g A_g L \tag{3.4.54}$$

where

$R_{\mathrm{pull},gk}$: characteristic value of the pushing resistance force of a pile group in the axial direction (kN)

The pile foundation shall be verified for both of the pulling resistance force in the axial direction of a pile group and each pile composing a pile group as a single pile. When verifying the pulling resistance force of a pile group in its axial direction, use the design value determined by considering safety margin based on the characteristic value calculated by **equation (3.4.54)**. For the safety margin to allow for, see **Part III, Chapter 2, 3.4.4 (4)**. For the part of the pulling resistance force in the axial direction deriving from the self-weight of a block, the safety factor is commonly reduced to about 2/3 of a normal value as the variance of the resistance force is assumed small.

When a structure borne by a group of piles is subjected to eccentric moment, pulling force will act on some piles. The kind of resistant force that acts on each pile at this time is not well known. A simple method is to verify by defining the piles the pulling force acts on as a pile group when assuming a linear resistant force distribution, as shown in **Fig. 3.4.13** and considering that the resultant of pulling force acts on the pile group [70].

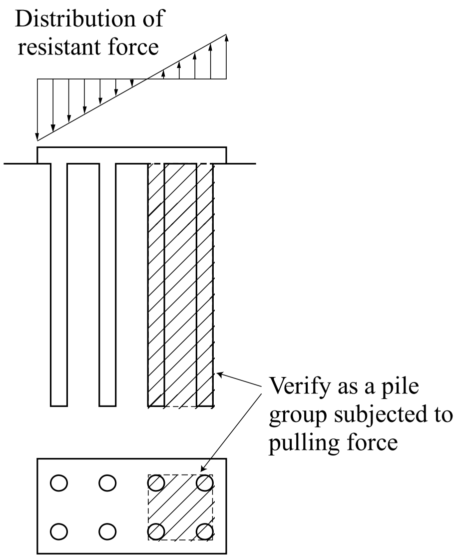


Fig. 3.4.13 Verification Method of Pulling Resistance Force on Foundation Subjected to Eccentric Moment

3.4.11 Negative Skin Friction Force

(1) When a bearing pile penetrates ground that may consolidate, the effect of negative skin friction force needs to be considered when examining the resistance force of a pile in its axial direction.

Consider a case where a weak layer consolidates and settles at a bearing pile embedded into a bearing stratum through a weak layer. As piles are borne by a bearing stratum and hardly settle, the friction force to the direction to push the piles is exerted in weak layer (see **Fig. 3.4.14**). As seen above, the downward friction force exerted on the skin of piles is called negative skin friction force (negative friction).

At this time, sandy layer sandwiched by consolidated and settled weak layers, sandy layer existing above the weak layer or other layer also settles relatively to the piles. As such, care needs to be taken that negative skin friction force may act irrespective of soil type in the soil layer shallower than the lower edge of the weak layer.

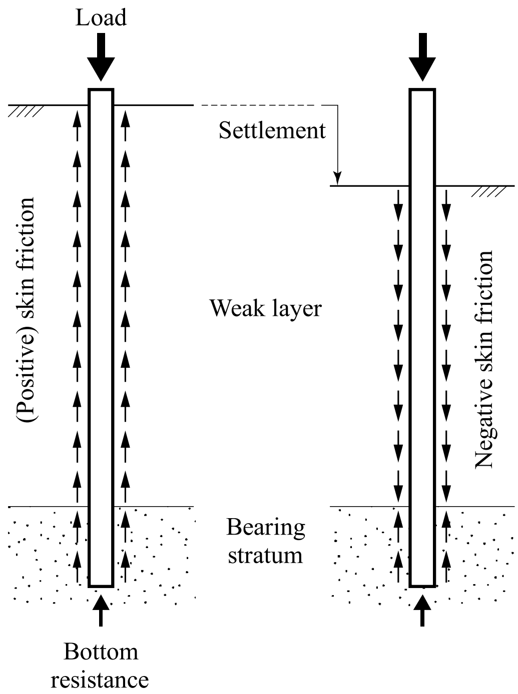


Fig. 3.4.14 Negative Skin Friction Force

(2) The negative skin friction force is caused by the relative displacement of settling ground and a pile. The negative skin friction force generally originates around the ground level where ground settles a lot and the action range of the negative skin friction force extends to the depth direction as ground subsidence proceeds. The boundary point of the range where negative skin friction force acts and its lower range where positive skin friction force acts is called the neutral point.

When the pile bottom is embedded in extremely rigid rock just below the weak layer, or in other cases, the neutral point is located at the lower edge of the weak layer (upper edge of rigid rock stratum) as the pile bottom is hardly displaced (settled) even if a large load acts on the pile. However, in normal cases, as the pile bottom penetrates the ground as the axial force due to negative skin friction force increases, the situation where the negative skin friction force is caused in the whole range from the ground level to the lower edge of the weak layer is not reached even the ground subsides to the maximum degree, and the neutral point settles somewhere between the ground level and the pile bottom. The depth of the neutral point differs according to the property of the ground, but past actual measurements give values on the order of (0.70–0.95)L_a [71]. Here, L_a means the depth of the lower edge of weak layer. However, the depth of neutral point can be considered L_a unless special result of investigation and others exist.

(3) When examining the bearing capacity of piles, many unclear aspects still exist about the way to consider the negative skin friction force. However, the characteristic value of the negative skin friction force acting on a single pile may normally be obtained by **equation (3.4.55)**.

$$R_{nfk} = \left(2\bar{N} L_S + \frac{\overline{q_u}}{2} L_C \right) \varphi \qquad \textbf{(3.4.55)}$$

where

R_{nfk} : characteristic value of the negative skin friction force acting on a single pile (kN)

L_S : thickness of sandy layer contained down to depth L_a (m)

L_C : thickness of clayey layer contained down to depth L_a (m)

L_a : depth of lower edge of a weak layer (m)
$L_a = L_S + L_C$

$\bar{N}$: mean SPT-N value of sandy layer contained down to depth L_a

$\bar{qu}$: mean unconfined compression strength of clayey layer contained down to depth L_a (kN/m^2)

φ : circumference of a pile (closed circumference for H-shaped steel piles) (m)

In the case of a pile group, the characteristic value of the negative skin friction per pile acting on a pile group may be obtained by **equation (3.4.56)** considering a block comprising a group of piles and the soil between the piles (see **Part III, Chapter 2, 3.4.10 (6)**).

$$R_{nfgk} = \frac{s_k U_g L_a + \gamma A'_g L_a}{n} \qquad \textbf{(3.4.56)}$$

where

R_{nfgk} : characteristic value of the negative skin friction per pile acting on a pile group (kN)

s_k : mean shear strength of soil contacting a block (kN/m^2)

γ : mean unit volume weight of soil contained in a block (kN/m^3)

U_g : circumference of a block (m)

A'_g : bottom area of a block (except pile portion) (m^2)

n : number of piles composing a pile group (piece)

In the case of a pile group, the negative skin friction force is considered to act on the block shaft same as a single pile. This corresponds to the first term of the numerator in **equation (3.4.56)**. On the other hand, soil in a block is borne by piles, and no relative displacement is assumed between soil and piles. At this time, a value corresponding to the self-weight of soil may be considered as the negative skin friction force caused inside of a block. This corresponds to the second term of the numerator in **equation (3.4.56)**.

Short distance between piles in a pile group makes the weight of soil inside of a block that should be borne by each pile of the pile group small, and piles bear the soil inside of the block. Then, the negative skin friction force is calculated by **equation (3.4.56)**. The longer the distance between piles becomes, the more weight of soil each pile needs to bear, and the more the negative skin friction force per pile calculated by **equation (3.4.56)** becomes. More separated piles cannot bear the soil inside of the block, and a relative displacement appears between piles and soil. Then, the negative skin friction force acting on the pile is calculated by **equation (3.4.55)**. This is the reason why it is considered reasonable to set whichever is smaller of the values calculated by **equation (3.4.55)** or **(3.4.56)** as the characteristic value of the negative skin friction force actually acting on piles. The actually acting negative skin friction force largely varies with the amount of consolidation settlement, the rate of consolidation, the creep characteristics of weak layer, and the deformation characteristics of bearing stratum. The characteristic value of the negative skin friction force calculated here is the maximum value of the negative skin friction force possible to be actually caused.

In the examination of bearing capacity of piles, as verification of the negative skin friction force, confirm that the sum of the skin friction force of a pile deeper than the neutral point (generally the lower edge of weak layer) and the characteristic value of the base resistance is greater than the sum of pushing force in the axial direction acting on the pile head and the characteristic value of the negative skin friction force (see **Fig. 3.4.14**). The design value of the pushing resistance force of a pile in its axial direction is generally used as the force in the axial direction acting on a pile head. For characteristic values of pushing resistance force of a pile in its axial direction, skin friction force and base resistance, see **Part III, Chapter 2, 3.4.3 Pushing Resistance Force of a Pile in Its Axial Direction**. Safety margin needs to be allowed for in verification. A safety factor on the order of 1.2 was used as a safety margin in the past, which may be referred to.

Verification of a pile body failure shall be performed. In the verification of the pile body failure, it is common to calculate the axial force acting on the neutral point from the sum of pushing force in the axial direction acting on the pile head and the characteristic value of the negative skin friction force and confirm that the stress caused in the pile body does not exceed the characteristic value of yield stress of material of the pile.

(4) A large expected negative skin friction force makes it important to perform detailed verification such as full consideration of the depth of neutral point. An analysis method assuming an elasto-plastic relation between the relative displacement caused between piles and the ground and the skin friction force has been proposed as a method [72]. Another simple calculation method has been suggested to examine the negative skin friction force considering the neutral point by ignoring the compression of a pile based on the above method [73]. However, as these methods are quite simplified, they need to be applied after closely examining the ground constant used for calculation, etc.

(5) Various construction methods have been proposed as countermeasures against negative skin friction force. Such methods include a method to apply thin film of asphalt and others on the skin of piles, a method to use a double pipe so that the negative skin friction force does not act on the pile's main body (inner pipe) bearing the structure, and a method to drive a dummy pile outside of the pile foundation considering the pile group effect [74]. It is desirable to use these countermeasure construction methods after fully confirming the applicability based on the past performance.

(6) If the negative skin friction act on batter piles, care needs to be taken for large bending of piles induced by it. The negative skin friction force acting on batter piles may be examined according to vertical piles, but bend is difficult to treat. A bend analysis method of batter piles based on the experimental study has been proposed [75], which may be referred to.

(7) Although not enough knowledge is available about the generation status of negative skin friction force when seismic force or others act, the negative skin friction force is not generally considered.

3.4.12 Details

(1) When determining the driving center distance between piles, effects of constructability, deformation behavior of surrounding ground, pile group effect, and so on need to be considered. Large distance between piles is generally advantageous in that each pile can fulfill a function as an individual pile. However, note that too much distance requires increase in bearing capacity for each pile and consequently in pile diameter and wall thickness, sometimes

making the facility uneconomical as a whole. The following items shall be fully considered in determining the distance between piles.

① the closest distance a pile driver can approach

② possibility of collision with neighboring piles due to errors in piles' center location, inclination of piles, and so on in driving construction

③ mound of soil or pressing of neighboring piles by soil removed when driving piles

④ effect of soil disturbance by driving into clayey ground on bearing capacity of neighboring piles

⑤ effect of soil compaction by driving into sandy ground on driving efficiency of neighboring piles

⑥ bearing capacity or negative skin friction force as a pile group

Many constructions restrict the minimum pile driving distance to on the order of 2.5–3 times of the pile diameter. When driving bearing piles into bearing stratum of rigid clayey layer, the distance between pile centers is normally 3–3.5 times or more in order to reduce the disturbance of bearing stratum by pile driving.

What is described here targets piles used as a foundation of structures and is not applied to cases using pile-shaped members such as steel pipe sheet pile wall and box-type sheet pile wall.

(2) When verifying performance of piles, actions in construction such as transport, erection, and driving shall also be examined. Although these examinations are often performed in the construction stage, it is desirable to perform examination corresponding to construction conditions along with the performance verification of pile foundations.

Piles are generally transported horizontally by supporting 2–4 points. Bending moment and shear due to the self-weight of piles are caused at this time. Care should be taken for the cases where load greater than the self-weight may act by application of impact and others during transportation. As the self-weight of a steel pipe pile is small relative to its cross-section, the cross-section is seldom determined by the stress during transportation. However, care should be taken not to have the cross-section deformed when stacking large-diameter thin-wall piles during transportation or temporary placement.

As to erection of piles, it is necessary to examine the tensile force in the axial direction caused by self-weight when hanging piles.

Massive dynamic compressive force and dynamic tensile force are exerted when driving piles. Specifically, massive dynamic tensile force may be exerted when a pile rapidly sinks into weak ground with an impact of hammer or in other cases. Examination is necessary so that dynamic tensile force does not disconnect or damage joints as the dynamic tensile force is said to indicate an absolute value on the order of dynamic compressive force. Also, care should be taken so that protruding part of a pile (above the ground level) does not buckle during driving.

(3) Joints shall be arranged to be completely safe against actions during construction and after completion of structures. The positions of joints shall be basically selected so as to ensure cross-sectional performance margin of piles. When examining the cross-sectional performance, positions of joints need to be selected according to the characteristic of their structures to avoid actions disadvantageous to their structures. Care needs to be taken as there is a case where a pile buckled at the joint part or wall thickness changing point below it by deformation behavior of the ground or other reasons even at deep portion where bending stress does not act under normal conditions. As the corrosion control performance of steel pile may be deteriorated by welding or other processing of joint part, joints shall be installed at a position not prone to the effect of corrosion, especially and hopefully avoiding where drying and wetting repeat due to fluctuation of the sea surface elevation.

The position of joints determines the single material allocation length of piles. Therefore, length of a single material determined with such constraint conditions as transportation, construction equipment, working space needs to be considered when examining the position of joints. Generally, it is structurally and economically advantageous to reduce the number of joints by using a single material as long as conditions permit.

Ensuring construction accuracy of joint part is harder than in shop fabrication because the construction accompanies field work. When selecting a joint structure, it is necessary to fully confirm its reliability. Although field welding is often used for joint part of steel pile, other methods have been developed in recent years, and they can be utilized after fully checking their safety. It is desirable not to install joints to wooden piles if horizontal force or pulling force acts on them.

(4) Sectional force caused in a pile body generally varies in depth direction and becomes smaller in deep underground locations. Therefore, plate thickness or steel type of a steel pipe pile is changed according to the depth even for one

pile from the economic point of view. The location where the plate thickness or steel type is changed is selected considering the distribution of sectional force of a pile and constructability. It should be noted that the plate thickness or steel type may not be changed when negative skin friction force acts. The portion where the plate thickness or steel type is changed is generally bonded by factory circumferential welding.

(5) The structure of the pile bottom is determined considering the ground condition and the construction method. Although steel piles are often used as open-ended piles with the bottom opened, they may be used as closed ended piles (closed-ended piles) attaching flat steel bottoms or pointed shoes. Closed-ended piles can expect large base resistance but on the other hand have less penetrability into the ground than open-ended piles. Moreover, closed-ended piles may rise when driving into weak ground. On the other hand, open-ended piles can be more accurately driven in terms of displacement, rotation, and others and have better constructability as a whole. Additionally, open-ended piles less vibrate ground and remove less soil when driving and are advantageous also from the viewpoint of influence to adjacent structures. Care needs to be taken when using hollow closed-ended piles since the soil pressure may buckle them in radial direction.

(6) Thickness of steel piles shall be set considering reduction due to corrosion. For the amount of corrosion of steel members, see **Part II, Chapter 11, 2.3 Corrosion of Steel Members**. Actions to which piles are subjected during construction may be verified assuming that the whole cross-section works effectively, without considering corrosion.

(7) When piles become hollow, e.g., closed-ended piles, or open-ended piles when soil inside of them is to be removed to fill the inside with concrete or for other reasons, verify concerning buckling in radial direction due to soil or water pressure acting on the side wall of piles. Special care needs to be taken when the wall of piles is extremely thin compared to their diameter or when embedment length of piles is very long. The external pressure causing buckling can be expressed by **equation (3.4.57)** when assuming the steel pile is subjected to uniform external pressure [76].

$$P_k = \frac{E}{4\left(1-\nu^2\right)}\left(\frac{t}{r}\right)^3 \quad \textbf{(3.4.57)}$$

where

P_k : external pressure to cause buckling (kN/m^2)

E : modulus of elasticity of a steel member (kN/m^2)

ν : Poisson ratio of a steel member

t : wall thickness of a steel pipe (mm)

r : radius of a steel pipe (mm)

Moreover, steel pipe piles of thin wall compared to their diameter may cause local buckling due to load in the axial direction. Examination based on static compression test of steel pipes shows the relation as in **equation (3.4.58)** between the buckling stress and the tensile yield stress in the axial direction [77].

$$\frac{\sigma_{cr}}{\sigma_y} = 0.86 + 2.7\frac{t}{r} \quad \left(0.01 \le \frac{t}{r} \le 0.10\right) \quad \textbf{(3.4.58)}$$

where

σ_{cr} : buckling stress of a steel pipe (kN/m^2)

σ_y : tensile yield stress of a steel member (kN/m^2)

Thin wall piles the radius thickness ratio (ratio of pile diameter to wall thickness) of which exceed 100 should not be used in normal cases.

(8) For detailed specifications and others of piles, see **Part II, Chapter 11, 2 Steel Members** and **Part II, Chapter 11, 3.6 Materials of Concrete Pile**.

(9) **Specifications for Highway Bridges, IV Substructures** [78], **Design Recommendations for Foundations of Buildings**[79] may be referred to according to kind, structural type, and others of facilities. **Pile Design Handbook**

for Highway Bridge Foundation [80] and **Pile Construction Handbook for Highway Bridge Foundation** [81] may also be referred to. When referring to these reference books, it is necessary to carefully take stock of assumed type of structures, size of structures, foundation conditions, action external force, and others and examine the applicability of the description. Moreover, take care not to easily or partially quote the content of these reference books, as it may be inadequate to combine a quotation from a book with one from the other books.

3.5 Settlement of Foundations

3.5.1 Stress in the Ground

(1) The stress in the ground induced by load of a foundation can be estimated by assuming that the ground is an elastic material. For uniformly distributed load, the underground stress may also be estimated by a simple method assuming linear stress dispersion.

(2) A reasonable approximate solution of stress induced in the ground when a structure having enough stability against shear failure of ground exists on the ground can be obtained even when assuming the soil to be an elastic material. The elastic solution used for calculation of stress in the ground is mainly Boussinesq's solution, which is based on the solution in the case where a vertically concentrated load acts on the surface of an isotropic and homogeneous semi-infinite elastic body. The stress in the ground for a line load and a surface load can be obtained by integrating this. In addition to the elastic solution, the Kögler method assuming linear dispersion of the stress can be used for estimating the stress in the ground for a strip load or a rectangular load [82].

(3) Note that the following solution of stress in the ground is used to obtain only the increment of stress in the ground due to applied load and that stress due to the self-weight of soil is not contained.

① **Stress in the Ground due to Concentrated Load**

When the ground is assumed to be a semi-infinite elastic body without self-weight, the stress in the ground induced by the concentrated load P applied on its surface is given in **equation (3.5.1)** by Boussinesq.

$$\sigma_z = \frac{P}{z^2} I_\sigma \qquad \textbf{(3.5.1)}$$

where

σ_z : vertical stress in the ground (kN/m^2)

P : concentrated load (kN)

z : depth from the ground level (m)

I_σ : influence value of vertical stress in the ground (see **Fig. 3.5.1**)

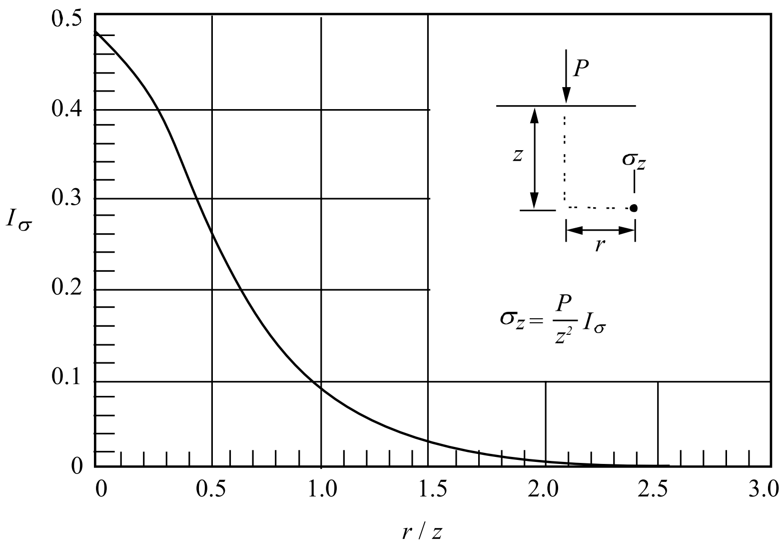


Fig. 3.5.1 Influence Value of Vertical Stress in the Ground due to Vertical Concentrated Load

② **Stress in the Ground due to Line Load**

When an infinitely long line load (p per unit length) is applied vertically, the underground vertical stress σ_z at depth z from the ground level is expressed by **equation (3.5.2)**.

$$\sigma_z = \frac{p}{z} I_\sigma \tag{3.5.2}$$

where

σ_z : vertical stress in the ground (kN/m^2)

z : depth from the ground level (m)

p : line load per unit length (kN/m)

I_σ : influence value (see **Fig. 3.5.2**)

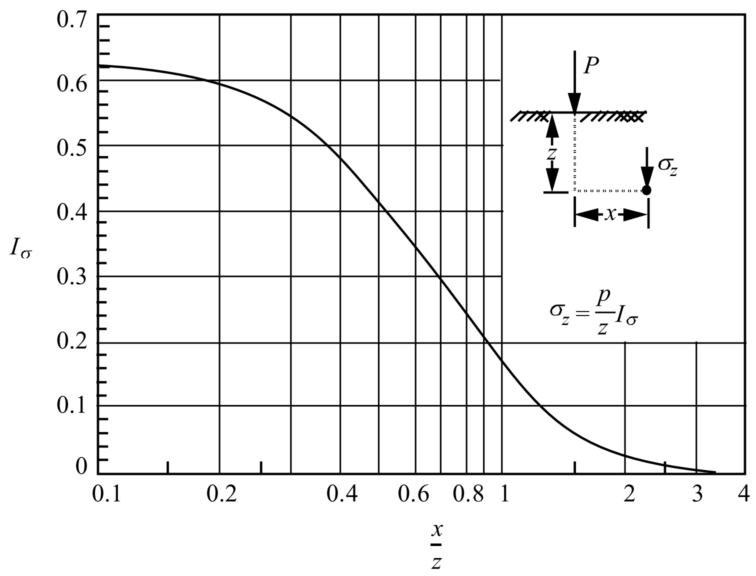


Fig. 3.5.2 Influence Value of Vertical Stress in the Ground due to Vertical Line Load

③ **Stress in the Ground due to Strip Load**

(a) Uniformly Distributed Strip Load

The stress in the ground induced by uniformly distributed strip load (width of load application: B (m)) is given by **equation (3.5.3)**.

$$\sigma_z = pI_\sigma \tag{3.5.3}$$

where

σ_z : vertical stress in the ground (kN/m^2)

p : strength of the load (kN/m^2)

I_σ : influence value (see **Fig. 3.5.3**)

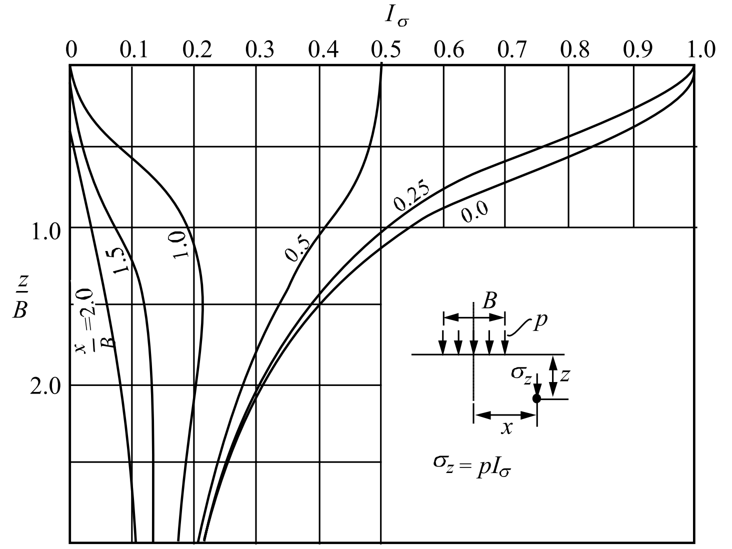


Fig. 3.5.3 Influence Value of Vertical Stress in the Ground due to Uniformly Distributed Strip Load

Not only the above elasticity solution, but the Kögler method assuming linear dispersion of the stress may also be used for uniformly distributed strip load. There are two methods, namely so-called the Boston Code Method, which was Kögler's first proposal but was so named after the building code in Boston City to which this was adopted, and the modified Kögler's method, as shown in **Fig. 3.5.4 (a)** and **Fig. 3.5.4 (b)**.

The Boston Code Method assumes that the vertical load on the ground level uniformly disperses at a certain angle α ($\alpha \geq 30°$). The vertical stress in the ground on a surface at any depth due to uniformly distributed strip load (width: B (m)) can be obtained by **equation (3.5.4)** from **Fig. 3.5.4**.

$$\sigma_z = \frac{p}{1+2\left(\dfrac{z}{B}\right)\tan\alpha} \tag{3.5.4}$$

where

σ_z : vertical stress in the ground (kN/m^2)

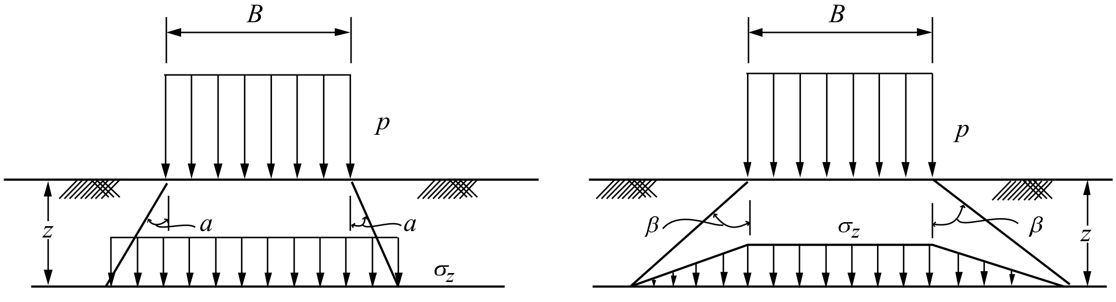
p : strength of the load (kN/m^2)

B : application width of uniformly distributed load (m)

z : depth from the ground level (m)

α : load dispersion angle (°), normally, $\alpha = 30°$

The modified Kögler's method was advocated to avoid irrationality of discontinuous stress in the ground when overlapping with the Boston Code method in **Fig. 3.5.4** [83]. As shown in **Fig. 3.5.4**, this assumes that the ground stress is trapezoidally distributed extending at an angle of β (normally, $\beta = 55°$) and the vertical ground stress in this case is given by **equation (3.5.5)**.



(a) Boston Code method (b) Modified Koegler method

Fig. 3.5.4 Linear Stress Dispersion

$$\sigma_z = \frac{p}{1+\left(\frac{z}{B}\right)\tan\beta} \qquad \textbf{(3.5.5)}$$

where

σ_z : vertical stress in the ground (kN/m^2)

p : strength of the load (kN/m^2)

B : application width of uniformly distributed load (m)

z : depth from the ground level (m)

β : load dispersion angle (°), normally, $\beta = 55°$

(b) Strip Load

The vertical stress in the ground due to strip load can be obtained by **equation (3.5.6)** using **Fig. 3.5.5**.

$$\sigma_z = pI_\sigma \qquad \textbf{(3.5.6)}$$

where

σ_z : vertical stress in the ground due to strip load (kN/m^2)

p : strength of the load (kN/m^2)

I_σ : influence value (see **Fig. 3.5.5**)

The vertical stress in the ground due to strip load can be obtained as in **Fig. 3.5.6 (b)** by algebraically overlapping triangular loads shown in **Fig. 3.5.6 (a)**. **Fig. 3.5.5** is the influence value obtained by Osterburg [84)] by such method.

The stress in the ground σ_z obtained by **equation (3.5.6)** from the influence value given in **Fig. 3.5.5** is within one vertical cross-section perpendicular to the normal line of infinitely continuing dam bodies and the influence value assuming trapezoidally distributed load (embankment load) on one side. Therefore, when the location the stress in the ground of which is going to be calculated is below the center line of symmetric embankment, double the influence value. Moreover, because influence value can be added or subtracted as they are obtained by assuming a linear elastic body, influence values corresponding to strip loads of various distribution profiles can be obtained.

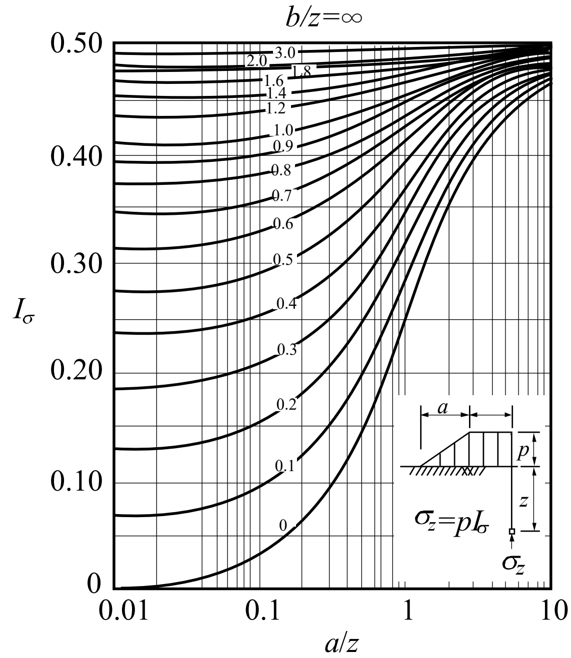


Fig. 3.5.5 Influence Value of Vertical Stress in the Ground due to Strip Load

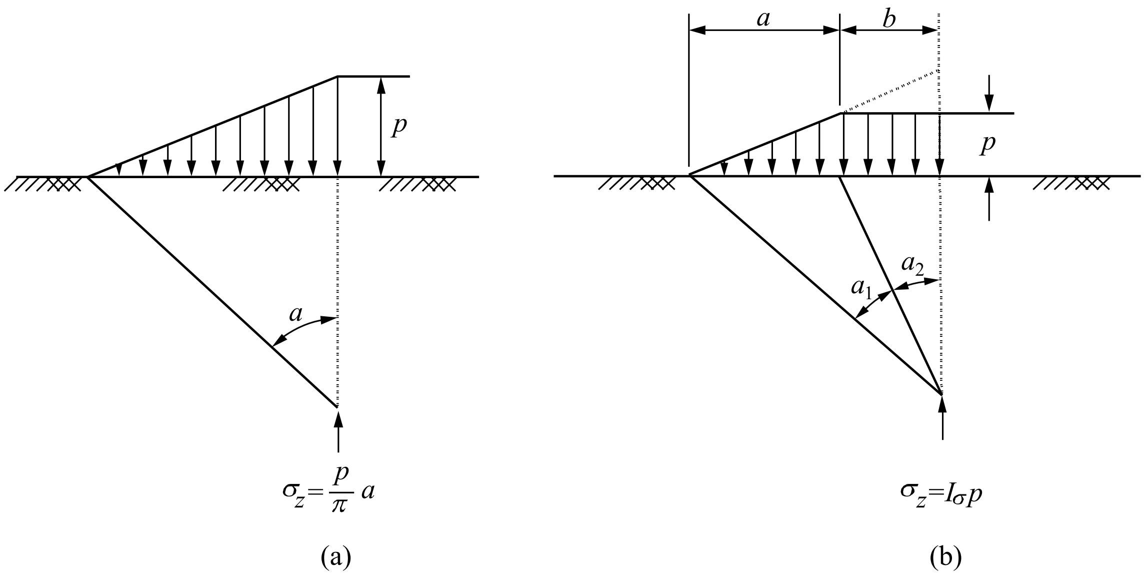


Fig. 3.5.6 Strip Load Consisting of Overlapped Triangular Loads

④ Stress in the Ground due to Surface Load

(a) Uniformly Distributed Load in a Circle Shape

The vertical stress in the ground when uniformly distributed load is applied on a circle shape of radius R can be obtained by **equation (3.5.7)**.

$$\sigma_z = pI_\sigma \tag{3.5.7}$$

where

σ_z : vertical stress in the ground due to uniformly distributed load in a circle shape (kN/m^2)

p : strength of the load (kN/m^2)

R : radius of the loading surface (m)

I_σ : influence value (see **Fig. 3.5.7**)

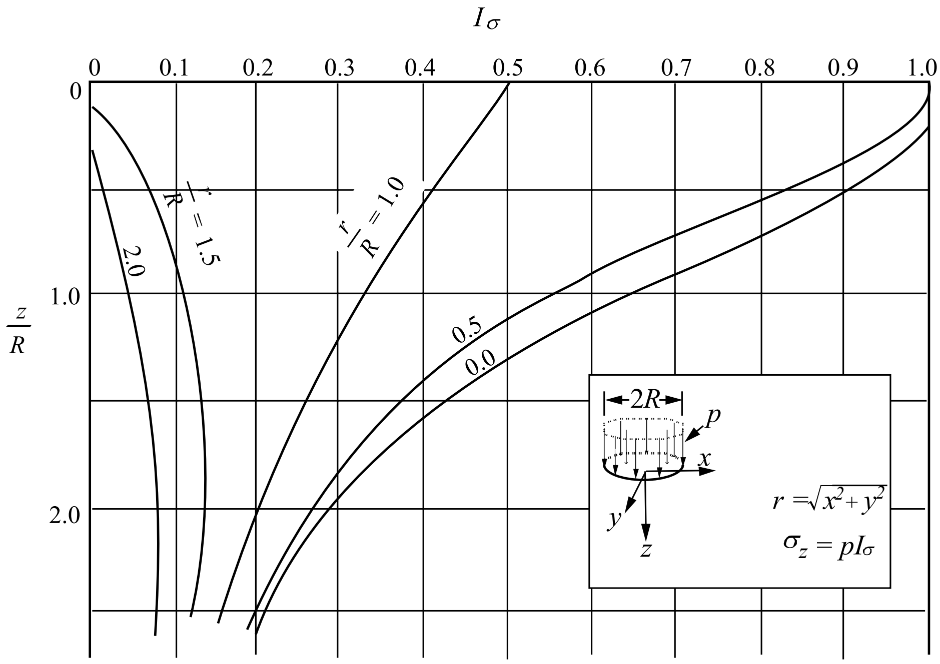


Fig. 3.5.7 Influence Value of Vertical Stress in the Ground due to Uniformly Distributed Load in a Circle Shape

(b) Uniformly Distributed Load in a Rectangular Shape

The vertical stress in the ground at any depth below a rectangular corner point when uniformly distributed load is applied to a rectangular loading surface (B (m) × L (m)) can be obtained by **equation (3.5.8)** using **Fig. 3.5.8**.

$$\sigma_z = pI_\sigma \qquad \textbf{(3.5.8)}$$

where

σ_z : vertical stress in the ground due to uniformly distributed load in a rectangular shape (kN/m^2)

p : strength of the load (kN/m^2)

I_σ : influence value (see **Fig. 3.5.8**)

The stress in the ground at a point other than rectangular corner points can be obtained by separating to several rectangles having the point as a corner point and algebraically summing the influence values of each rectangle.

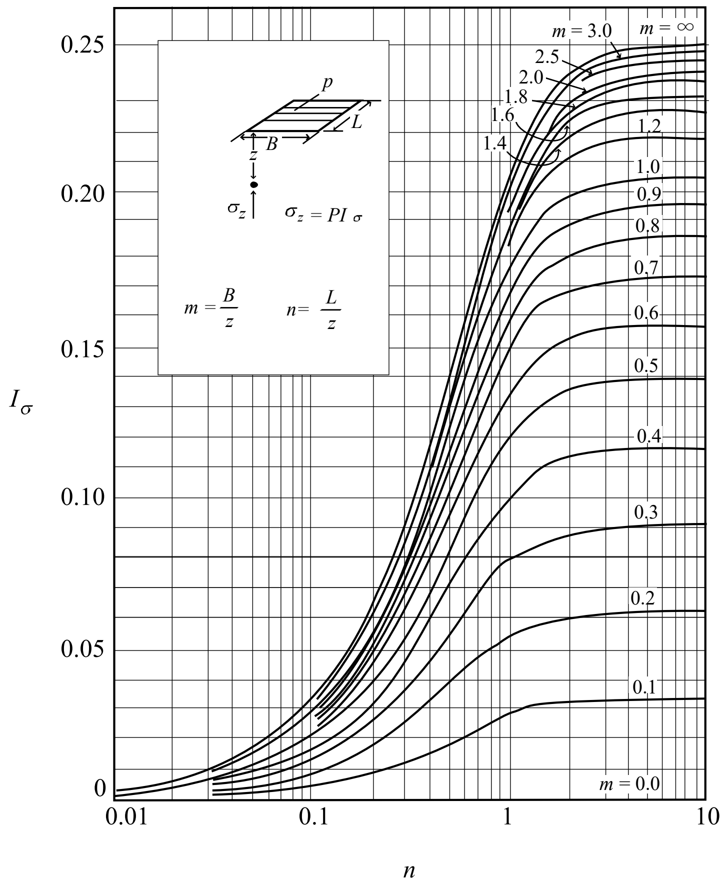


Fig. 3.5.8 Influence Value of Vertical Stress in the Ground due to Uniformly Distributed Load in a Rectangular Shape

⑤ **Westergaard's Formula**

Thin coarse-grained lenticular soil layer may exist in a clayey layer. If this type of soil layer exists, occurrence of lateral strain is considered to be avoided. Thus, Westergaard derived an elasticity solution assuming an elastic material where countless densely spaced elastic sheets continue infinitely, and no lateral strain is caused as a whole. Note that Boussinesq's formula is given irrespective of Poisson ratio, whereas Westergaard's solution contains Poisson ratio.

It cannot be easily determined which formula is better when applying to actual ground. Although Westergaard's formula seems to be closer to the condition of sedimentary soil than the isotropic condition Boussinesq assumed in that Westergaard derived an elastic formula considering the bedding condition of the ground, but Westergaard's formula is inconvenient because it needs to give a Poisson ratio.

3.5.2 Immediate Settlement

(1) In estimation of immediate settlement, it is preferable to apply the theory of elasticity by appropriately setting the modulus of elasticity of the ground.

(2) Immediate settlement, unlike consolidation settlement, which will be described in the following, is caused by shear deformation and occurs simultaneously with loading. Because sandy ground does not undergo long-term consolidation settlement like that in clayey ground, immediate settlement in sandy ground, as described here, can be considered to be total settlement. On the other hand, the immediate settlement of clayey ground is a phenomenon which is caused by settlement due to undrained shear deformation in the lateral direction. In soft clayey ground, there are cases in which immediate settlement may be ignored in performance verification because it is smaller than the consolidation settlement described below.

In calculations of immediate settlement, the ground is usually assumed to be an elastic body, and the theory of elasticity and the modulus of elasticity E and Poisson's ratio v are used. As the modulus of elasticity of soil varies greatly depending on the strain level, it is important to make calculations using a modulus of elasticity that corresponds to the actual strain level. For example, the strain in soft ground with a small safety factor is on the order of 0.5% to 1.5%, whereas that in excavation of hard ground and deformation of foundations is no more than 0.1%. The relationship between the strain level and the elastic modulus shall follow **Part II, Chapter 3, 2.3.1 Elastic Constants**.

(3) Settlement due to Vertical Concentrated Load

The settlement of the ground surface S subjected to the vertical concentrated load P as an action is given by **equation (3.5.9)** (see **Fig. 3.5.1**).

$$S = \frac{P\left(1-\nu^2\right)}{\pi\, r\, E} \tag{3.5.9}$$

where

S : settlement (m)

P : concentrated load (kN)

ν : Poisson's ratio

E : modulus of elasticity of soil (kN/m^2)

r : horizontal distance from the load action point (m)

(4) Settlement due to Vertical Line Load

The settlement of the ground surface S in this case is expressed by **equation (3.5.10)** (see **Fig. 3.5.2**).

$$S = \frac{2\,p\left(1-\nu^2\right)}{\pi E}\ln\left(\frac{d}{r}\right) \qquad (r \le d) \tag{3.5.10}$$

where

d : horizontal distance (m) between the point where the settlement becomes 0 on the ground surface and the loading location of the line load, which should be appropriately estimated

p : vertical line load (kN/m)

S : settlement (m)

ν : Poisson's ratio

E : modulus of elasticity of soil (kN/m^2)

r : horizontal distance from the load action point (m)

(5) Settlement due to Uniformly Distributed Load in a Circle Shape

The settlement of the ground surface S at the center of a circle is given by **equation (3.5.11)** (see **Fig. 3.5.7**).

$$S = \frac{2\,p\left(1-\nu^2\right)}{E}R \tag{3.5.11}$$

where

S : settlement (m)

R : radius of a circular load (m)

p : uniformly distributed load (kN/m^2)

ν : Poisson's ratio

E : modulus of elasticity of soil (kN/m^2)

(6) Settlement due to Uniformly Distributed Load in a Rectangular Shape

The settlement of the ground surface *S* at the corner point *N'* of a rectangle is given by **equation (3.5.12)** (see **Fig. 3.5.8**).

$$S = p\,B\,\frac{\left(1-\nu^2\right)}{E} I_S \tag{3.5.12}$$

where

S : settlement (m)

I_S : influence value against settlement. I_S is a function of proportion of a rectangle (L/B), which is shown in **Fig. 3.5.9**.

p : uniformly distributed load (kN/m^2)

ν : Poisson's ratio

E : modulus of elasticity of soil (kN/m^2)

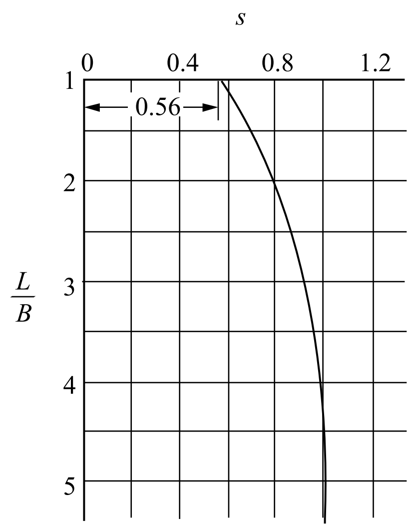


Fig. 3.5.9 Influence Value to Settlement of Corner Points due to Uniformly Distributed Load in a Rectangular Shape

3.5.3 Consolidation Settlement

(1) Time-dependent changes in the final consolidation settlement and the consolidation settlement of a foundation shall be examined in accordance with **Part II, Chapter 3, 2.3.2 Compression Consolidation Characteristics**. Consolidation-related physical properties for the ground can be set by using an appropriate method based on the results of consolidation tests.

(2) Calculations of settlements due to consolidation can be performed based on the results of consolidation tests on undisturbed samples of clayey soils. The final consolidation settlement, which is the amount of soil settlement when consolidation settlement caused by a certain surcharge has finally completed, depends on by the compressibility properties of the soil skeleton structure and can be calculated directly from the results of consolidation tests. Time-dependent changes in settlement up to the final consolidation settlement of a foundation can be calculated based on the theory of consolidation.

(3) Calculation Methods of Final Consolidation Settlement of Foundation

Final consolidation settlement of a foundation can be calculated by using the following equations as described in **Part II, Chapter 3, 2.3.2 Compression Consolidation Characteristics**.

① **When using a compression curve (*e*-log *p* curve):**

$$S = h\frac{\Delta e}{1+e_0} \tag{3.5.13}$$

where

S : final consolidation settlement due to pressure increment Δp (m)

h : layer thickness (m)

Δe : change in void ratio for pressure increment Δp (read from a compression curve)

e_0 : initial void ratio

② **When obtained from C_c:**

Application of this method is limited mainly to the cases in which consolidation of the normal consolidation area is considered.

$$S = h\frac{C_c}{1+e_0}\log_{10}\frac{\sigma'_{v0}+\Delta p}{\sigma'_{v0}} \tag{3.5.14}$$

where

S : final consolidation settlement due to pressure increment Δp (m)

h : layer thickness (m)

C_c : compression index

e_0 : initial void ratio

σ'_{v0} : effective overburden pressure before loading (kN/m^2)

Δp : pressure increment (kN/m^2)

③ **when obtained from m_v:**

Application of this method is limited to cases in which the increment of consolidation pressure is sufficiently small that m_v can be considered constant.

$$S = m_v \Delta p h \tag{3.5.15}$$

where

S : final consolidation settlement due to pressure increment Δp (m)

m_v : coefficient of volume compressibility when consolidation load is $\sqrt{\sigma'_{v0}(\sigma'_{v0}+\Delta p)}$ (m^2/kN)

σ'_{v0} : effective overburden pressure before loading (kN/m^2)

Δp : pressure increment (kN/m^2)

h : layer thickness (m)

(4) Calculation Method of Time-Settlement Relationship

The rate of consolidation settlement can be calculated from the relationship between the average degree of consolidation U and the time factor T_v that is obtained from Terzaghi's consolidation theory, where the dissipation of excess pore water pressure is expressed as a differential equation of thermal conductivity type. The amount of settlement $s(t)$ at a given time t can be calculated by multiplying the final settlement S with the average degree of consolidation $U(t)$ by the following equation:

$$s(t) = SU(t) \tag{3.5.16}$$

The finite element analysis with visco-elasto-plasticity model for clayey soil is desirable to be utilized for accurate analysis of the consolidation settlement that takes account of inhomogeneity on compression consolidation characteristics of the ground, the effect of self-weight of clayey layer and time-related changes in consolidation load.

(5) Division of Clayey Layer subject to Consolidation

When calculating the final consolidation settlement, the clayey layer is divided into a number of segments as shown in **Fig. 3.5.10**. This is because the consolidation pressure increment $\Delta\sigma_z$, the consolidation yield stress p_c, and the coefficient of volume compressibility m_v vary with depth. The final consolidation settlement S_0 of foundation can be calculated using **equation (3.5.17)** as a sum of settlement S of segments calculated by **equation (3.5.13)**, **equation (3.5.14)** or **equation (3.5.15)** when assuming the layer thickness h is the thickness of each segment.

$$S_0 = \sum S \tag{3.5.17}$$

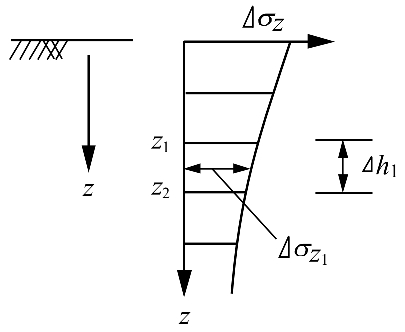


Fig. 3.5.10 Division of the Clayey Layer in Calculating the Consolidation Settlement

The thickness of segments Δh is usually set at 3 to 5 m. It should be noted that the consolidation settlement of soft clayey layer will be underestimated when Δh is taken too large because the initial void ratio e_0 of the surface layer is very large and it governs the total settlement.

The increment of consolidation pressure $\Delta\sigma_z$ in each segment is calculated at the center of each segment using the distribution with depth of the vertical stress in the ground, which is described in **Part III, Chapter 2, 3.5.1 Stress in the Gound**. The term $\Delta\sigma_z$ is the increment in consolidation pressure due to loading. In the natural ground, it is usually assumed that consolidation due to the overburden pressure has finished.

Although the distribution of subgrade reaction at the bottom of rigid loading plate is not uniform, the highly rigid loading plate settles uniformly, and the distribution of stress in the ground at a certain depth practically becomes irrelevant to the distribution of subgrade reaction immediately below the loading plate. Therefore, the distribution of vertical stress in the ground may be determined considering only the load distribution form on rigid loading plates.

(6) Vertical Coefficient of Consolidation c_v and Horizontal Coefficient of Consolidation c_h

When pore water of soil flows vertically, the coefficient of consolidation c_v is used in calculation. But when vertical drains are installed and consolidated, drained water flows mainly to the horizontal direction and the horizontal coefficient of consolidation c_h should be used. The value of c_h obtained from experiments on the clay in Japanese port areas ranges from 1.0 to 2.0 times the value of c_v [85]. However, in performance verification $c_h \approx c_v$ is acceptable, considering a decrease in c_h due to disturbance caused by installation of drains, inhomogeneous consolidation constants in the ground, and others.

(7) Coefficient of Consolidation c_v of Overconsolidated Clay [86]

The coefficient of consolidation of clayey soil in overconsolidated state is generally larger than that in normally consolidated state. When the clayey soil seems to be clearly in overconsolidated state, the value of c_v used should be the one at the mean consolidation pressure between the effective overburden pressure before loading and the final consolidation pressure based on the result of a consolidation test. However, rather than simply calculating c_v at the mean stress, it would be better to consider the settlement and use a weighted c_v, as it were a mean amount of settlement.

(8) Rate of Consolidation Settlement in Inhomogeneous Ground

When layers with different c_v are alternate, the rate of consolidation settlement is analyzed using the equivalent-layer thickness method [87], numerical solution using the finite difference method [88], or the analysis method using the finite element method [89), 90), 91]. The equivalent-layer thickness method is used as a simplified method, but it sometimes yields significant errors. When the ground is inhomogeneous to a large extent, or when accuracy is required, it is recommended to use the finite element method.

(9) Settlement due to Secondary Consolidation

The shape of the settlement-time curve in long-term consolidation tests on clayey soil is well consistent with Terzaghi's consolidation theory up to the degree of consolidation of around 80%. When the degree of consolidation exceeds this level, the settlement increases linearly with logarithm of time. This is due to the secondary consolidation that arises with the time-dependent compression properties of soil skeleton (easily understood by imagining viscosity), besides the primary consolidation that causes the settlement accompanying dissipation of excess pore water pressure induced in the clayey soil due to consolidation load.

The settlement due to secondary consolidation is particularly significant in peat and other organic soils. In ordinary Holocene clay grounds, the consolidation pressure caused by loading is often several times greater than the consolidation yield pressure of the ground. Under such conditions, the settlement due to secondary consolidation is smaller than that due to the primary consolidation and is not significant in the performance verification. But when the pressure acting on the ground due to loading does not greatly exceed consolidation yield stress, the settlement due to secondary consolidation tends to continue over a long time, even though the settlement due to primary consolidation may be small. In this case, the secondary consolidation settlement needs to be fully taken into account in the performance verification. When a large-scale reclamation is conducted on the seabed containing thick Pleistocene clay layer accumulating in deep portion, the ratio of secondary consolidation becomes large due to consolidation settlement of Pleistocene clay layer with reclamation load and others.

The settlement due to secondary consolidation can be calculated using the following equation. But the time when secondary consolidation started is generally unclear, and comprehensive consideration is required when applying the following equation.

$$S_s = \frac{C_\alpha}{1+e_0} h \log_{10}\left(\frac{t}{t_0}\right) \tag{3.5.18}$$

where

S_s : settlement due to secondary consolidation (m)

C_α : secondary compression index (also called coefficient of secondary consolidation)

t : time (d) (d means day)

t_0 : start time of secondary consolidation (d)

h : clay layer thickness (m)

The secondary compression index C_α can be obtained from consolidation tests. However, in the long-term consolidation test conducted in laboratories, C_α and the compression index C_c empirically have the relation expressed by the following equation, and thus C_α may be estimated from C_c [92].

$$C_\alpha = (0.03 \sim 0.05) C_c \tag{3.5.19}$$

(10) Estimation of Long-term Settlement by Introduction of Isotache [93]

If the consolidation pressure slightly higher than the consolidation yield stress acts in the ground by construction of a facility, the settlement caused by secondary consolidation may become bigger than that caused by primary consolidation. This case includes development of a huge artificial island on thick Pleistocene clay seabed grounds. Estimation of long-term settlement in design and maintenance stages becomes important in this case since residual settlement occurs continuously even after service has started. As a result of the study aiming at improvement of prediction accuracy of the long-term consolidation settlement, a chart to simplify prediction of long-term settlement has been shown. This chart formulates the concept of isotache [94] as a creep model, which was advocated focusing on the dependability of consolidation settlement behavior on the rate of strain. The more detailed description can be found in **Part II, Chapter 3, 2.3.2 Compression Consolidation Characteristics**. For details, see Reference [93].

This method assumes that the compression curve obtained from the **Stage Loading Consolidation Test (JIS A 1217)** is the one corresponding to the rate of strain on the order of 1.0×10^{-7} s^{-1}, expresses in an equation that the amount of strain increases as the rate of strain *in-situ* is smaller than this and schematizes as a chart. Increment in strain corresponding to a set rate of strain can simply be obtained such as the increment from the consolidation settlement strain directly predicted from the result of stage loading consolidation test with the traditional calculation method to the final consolidation settlement strain corresponding to the point where the rate of strain finally reached zero and the rate of strain corresponding to the permissible rate of settlement when considering performance and working life of structures.

3.5.4 Lateral Displacement

(1) In wharfs or revetments constructed on soft clayey ground, countermeasures are preferable when lateral displacements due to shear deformation of the ground have an effect on structures.

(2) In wharfs or revetments on soft ground, there are cases in which it is necessary to estimate lateral displacements caused by shear deformation of the ground. Lateral displacements include displacement accompanying immediate settlement occurring immediately after loading, and displacement which occurs continuously over time thereafter. In cases where the imposed load is significantly smaller than the ultimate bearing capacity of the ground, lateral displacement accompanying immediate settlement can be predicted by analyzing the ground as an elastic body.

(3) Lateral displacement, which becomes problematic with soft ground, is the phenomenon where there is no margin in stability and creep deformation caused by shear occurs in addition to consolidation. A method to determine whether this kind of lateral displacement will occur or not using a simple constant based on past experience has been proposed [148]. When making a more detailed analysis, programs which obtain changes over time in settlement and lateral displacement by finite element analysis are widely used, by applying an elasto-plastic model or an elasto-visco-plastic model to clayey ground. Because the importance of lateral displacement differs greatly depending on the functions of the facilities, it is necessary to select an appropriate calculation method considering these functions.

3.5.5 Differential Settlements

(1) When constructing structures, uneven settlements of the ground surface (this is called differential settlement) caused by inhomogeneous settlement of ground shall be taken into account and countermeasures as appropriate are preferable when differential settlements have an effect on structures. Differential settlements are specifically predominant in soft clayey ground.

(2) Causes and Types of Differential Settlements

Differential settlements that cause problems in port structures are as follows:

① Differential settlements occurring between foundations of structures and reclaimed land

Ex. Differential settlements occurring between buildings borne by piles and reclaimed ground, settlement occurring between pile-supported type bridges and their attaching portions

② Differential settlements occurring between improved ground portions and intact portions

Ex. Differential settlements occurring between grounds improved with drains or deep layer mixed processing and intact grounds

③ Differential settlements occurring by difference in the amount of load acting on grounds or the history of construction

Ex. Settlement of fill and accompanying settlement in its vicinity, settlement around the buried structures

④ Differential settlements caused by inhomogeneous compressibility or consolidation characteristics of grounds

①, ②, and ③ out of the above four items should be considered in performance verification of structures or ground improvement, and the prediction of differential settlements becomes important. Differential settlements in ④ can also be predicted to some extent by numerical analysis considering inhomogeneous nature of grounds [95] [96].

(3) Countermeasures against Differential Settlements

Countermeasures against differential settlements are as follows:

① Avoid damages due to differential settlements by installing flexible joints between structures and buried structures.

② Use light materials so that surcharge acting on grounds can change smoothly, or heavy materials to adjust load.

③ Install a runoff section of improved ground area and intact area.

(4) A method to easily estimate differential settlement in reclaimed land in port areas has been proposed. This method classifies the ground of reclaimed land into the following four types:

① Extremely inhomogeneous ground
② Inhomogeneous ground
③ Ordinary ground
④ Homogeneous ground

Fig. 3.5.11 shows the mean differential settlement ratios for each type of ground. The mean differential settlement ratio means the ratio of the difference in the average settlement occurring between two arbitrary points to the total settlement. For example, because the mean differential settlement ratio for two points separated by a distance of 50 m in inhomogeneous ground ② can be read as 0.11, when settlement of ΔS occurs from a certain reference time, the average differential settlement occurring in the distance of 50 m can be calculated as $0.11\Delta S$. When applying this method to actual problems, it is preferable to correct the values in **Fig. 3.5.11** for the reference time and the depth of the ground, which is the object to settlement [97), 98)].

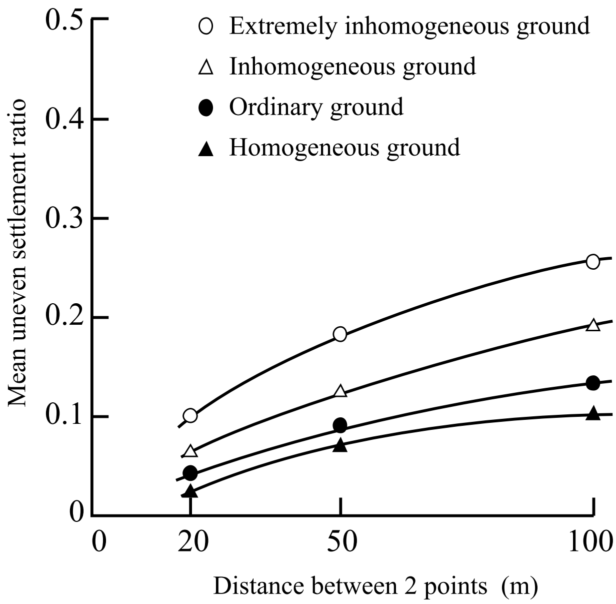


Fig. 3.5.11 Relationship between Distance and Differential Settlement Ratio in Reclaimed Land

3.5.6 Ground Subsidence in Wide Area

(1) In ground subsidence areas, it is desirable to take appropriate measures by investigating the situation and mechanism of subsidence in detail and estimating subsidence in future.

(2) Causes of Ground Subsidence

Several causes of ground subsidence may be noted. The previous analysis results of subsidence phenomenon in ground subsidence areas show that main causes are the contraction and consolidation of soil layer caused by increase in effective stress due to drawdown of ground water by rapid pumping of groundwater.

Pumping of massive groundwater for industrial water and irrigation water, oil and natural gas mining, snow melting, and other purposes decreases the water pressure in water-bearing strata. When the water pressure in water-bearing strata, which are often gravel bed, decreases, the gravel bed contracts by increased effective stress in the gravel bed. Decreased water pressure in water-bearing strata causes the hydraulic gradient near the boundary with their neighboring clayey layer, and the water in clayey layer is drained to water-bearing strata to promote consolidation. The rate or amount of ground subsidence confirmed in Japan shows evident correlation with reduction in pumpage of groundwater or groundwater pressure. These are the main causes of the ground subsidence.

(3) Countermeasures against Ground Subsidence

Current technology does not make it possible to raise subsided ground surface to its original height. Therefore, a feasible countermeasure is to reduce the rate of subsidence and the amount of subsidence in future. Thorough investigation of the situation of ground subsidence and the mechanism of subsidence is required in order to take such countermeasures. Major items to investigate are as follows:

① Amount and rate for the whole ground subsidence area
② Strata, geology, and soil properties of the subsidence area
③ Changes over time in groundwater pressure in each water-bearing stratum
④ Amount of compression per layer

The results of the above investigation enable estimation of the amount of consolidation and compression assuming reduction in water pressure in future. The regulation of groundwater pumping has been fruitful as a specific countermeasure against ground subsidence. Pouring water in the ground successfully stopped subsidence in Long Beach, US. The countermeasures described in **Part III, Chapter2, 3.5.5 Differential Settlements** are required for performance verification of facilities installed in ground subsidence area

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4 Stability of Slopes

4.1 General

(1) Stability of slopes against slip failure caused by self-weight of soil or surcharge may be analyzed as a two-dimensional problem, assuming a circular arc slip surface or a straight sliding surface.

(2) It is necessary to perform slope stability analysis for the case in which a slope becomes least stable.

(3) In slope stability analysis, the stability of the soil mass comprising a slope against the self-weight of the soil or surcharge is verified by the ultimate equilibrium state method. It is necessary to confirm that the design value of shearing resistance exceeds the design value of shearing force caused by actions. Calculation methods used in the slope stability analysis can also be used to calculate the bearing capacity of foundations as these calculation methods are used to examine the stability of soil masses. The method described below can be used in verification of stability against variable situations in respect of Level 1 earthquake ground motion in addition to the permanent situation.

(4) Shapes of Slip Surface

① **Types of shapes of slip surfaces**

Theoretically, shapes of slip surfaces in slope stability analysis may be combinations of linear, logarithmic spiral, and/or circular arc shapes [1]. In practice, however, linear or circular arc slip surfaces are assumed. When there is a particularly weak layer, and a slip surface is expected to pass over it, that slip surface or other appropriate slip surfaces may sometimes be assumed. An assumed slip surface, in general, should be the one along which the slip of the soil mass smoothly takes place. Thus, a slip surface with sharp bends or curves that seems to be kinematically unnatural should not be used.

② **Slip failure of slope on sandy soil ground**

Slip failure of slopes of dry sand or saturated sand usually takes a form in which the slope collapses, and as a result, its inclination decreases. Therefore, it is more appropriate to consider failure surface of a slope of these types as a straight sliding surface than as a circular slip failure surface. Even when considering a circular slip failure surface, the form is close to a straight line passing through the vicinity of the surface layer. The inclination of a sandy slope when the slope is in a state of equilibrium is termed the angle of repose. This angle of repose is equivalent to the angle of shear resistance, which corresponds to the void ratio of the sand comprising the slope. In the case of unsaturated sand, the slope possesses apparent cohesion resistance caused by the suction due to the surface tension of the pore water. As a result, its angle of repose is far larger than in the cases of dry sand and saturated sand. However, saturation may increase due to infiltration of rainwater or a rise in the groundwater level, causing a sudden decrease in apparent cohesion resistance, or angle of repose. Therefore, adequate consideration is necessary so that enough stability can be secured under the supposed conditions.

③ **Slope failure of cohesive soil ground**

The actual slip failure surface of cohesive soil ground is close to a circular arc, and a deep slip called the base failure often takes place, whereas a shallow slip appears near the surface layer in sandy slope.

Slope stability analysis is often treated as a two-dimensional problem. Although actual slip surface in slopes with long extension takes the form of three-dimensional curved surfaces, a two-dimensional analysis gives a solution on the safer side. When the stability is expected to decrease due to surcharge over a finite extension, however, the resistance of both sides of a cylindrical failure surface may be taken into account.

(5) Actions in Slope Stability Analysis

Important causes of slip failures are self-weight of soil, surcharge, water pressure, and others. Beside them, repeated actions such as seismic force, wave force, and others may be included. Resistance against the slip is given by shear resistance of soil and counterweight.

Because the shear strength of soil is related with time, the stability problems on soil mass are classified into two cases; loading on the ground in normally consolidated state and unloading by excavation. The former is referred to as a short-period stability problem and the latter a long-period. It is preferable to set shear strength appropriate to each case (see **Part II, Chapter 3, 2.3.3 Shear Characteristics**).

(6) Stability verification in slope stability problems can be performed by confirming that the ratio of the design value of shear stress to the design value of the shear strength of soil in an assumed slip surface is equal to or smaller than

1.0. The value of the obtained ratio will differ depending on the assumed slip surface. The result with the largest ratio of "shearing force"/"shearing resistance" among combinations of the shearing resistance and shearing force obtained assuming several slip surfaces based on the given conditions shall be regarded as the limit state for slip failure of the slope under study.

(7) Partial Factors

In examination of the stability of slopes, the partial factors for each structural type of facilities, or partial factors by type of improved soil can generally be used. The parts to be referenced on partial factors are as shown in **Table 4.1.1**.

Because the position of the slip surface will differ depending on how the partial factors are determined, caution is necessary when the range of soil improvement is to be determined based on the stability verification. For example, if the partial factor that multiplies the resistance term is set small, the range of slip failure, which is the limit state, will be narrow. This means that the necessary range of soil improvement will be underestimated.

Table 4.1.1 Parts to be Referenced on Partial Factors for Use in Verification of Slip Failure

Applicable facilities for partial factors	Parts to be referenced	Applicable facilities
Composite breakwater	**Part III, Chapter 4 Protective Facilities for Harbors 3.1 Gravity-type Breakwaters (Composite Breakwaters), Table 3.1.1**	Upright breakwater, sloping caisson breakwater, upright wave-dissipating block type breakwater, wave-dissipating caisson type breakwater
Breakwater armored with wave-dissipating blocks		Sloping top caisson breakwater armored with wave-dissipating blocks
Gravity-type quaywall	**Part III, Chapter 5 Mooring Facilities 2.2 Gravity-type Quaywalls, Table 2.2.1**	Gravity-type revetment, placement-type cellular-bulkhead quaywall
Sheet pile quaywall		Sheet pile revetment, cantilevered sheet pile quaywall
SCP improved soil	**Part III, Chapter 2, 5 Soil Improvement Methods 5.10 Sand Compaction Pile Method for Cohesive Soil Ground, Table 4.10.2**	Gravity-type quaywall or sheet pile quaywall applying SCP improvement
Others	In accordance with this section (**4 Stability of Slopes**)	Sloping breakwater and other similar facilities

4.2 Examination of Stability

4.2.1 Stability Analysis by Circular Slip Failure Surface

(1) Examination of the stability of slopes can be performed by circular slip failure analysis with the modified Fellenius method, which is given by the following equation, or by an appropriate method equivalent to the bearing force in **Part III, Chapter 2, 3.2.5 Bearing Capacity for Eccentric and Inclined Actions**, depending on the characteristics of the ground. For partial factor γ_S that multiplies the action term, partial factor γ_R that multiplies the resistance term, and adjustment factor *m* in **equation (4.2.1)**, those for each structural type of facility or those by type of improved soil should be used. The conventional design, using the safety factor method, is equivalent to the design where both γ_S and γ_R are 1.00: Factor *m,* that is, equivalent to the safety factor, was set at 1.30 or higher for permanent situations, but in cases where the reliability of the constants used in verification can be considered high, based on actual data for the same ground, and monitoring work is carried out by observing the displacement and stress of the ground during construction, factor m could be set at 1.10 or more for the same situations. [1)] In line with these rules, when partial factors γ_S and γ_R have not been determined, they can be set as 1.00, in accordance with the conventional method, and the adjustment factor *m* can be set to a value equivalent to the conventional safety factor to verify stability.

$$m \cdot \frac{\gamma_S \cdot \left[\sum \left\{x\left(W_k + q_k\right) + aP_{Hk}\right\}\right]}{\gamma_R \cdot R \sum \left\{c_k \ell + \left(W'_k + q_k\right)\cos\theta \tan\phi_k\right\}} \le 1 \quad \textbf{(4.2.1a)}$$

$$m \cdot \frac{\gamma_S \cdot \sum \left\{ (W_k + q_k) \sin\theta + \frac{1}{R} a P_{Hk} \right\}}{\gamma_R \cdot \sum \left\{ c_k s + (W'_k + q_k) \cos^2\theta \tan\phi_k \right\} \sec\theta} \leq 1 \qquad \textbf{(4.2.1b)}$$

where

R : radius of circular slip failure (m)

c_k : in case of cohesion soil ground, characteristic value of undrained shearing strength, and in case of sandy ground, characteristic value of apparent cohesion in drained condition (kN/m^2)

l : length of bottom of slice segment (m)

W'_k : characteristic value of effective weight of slice segment per unit of length (weight of soil. When submerged, unit weight in water) (kN/m)

q_k : characteristic value of vertical action from top of slice segment (kN/m)

θ : angle of bottom of slice segment to horizontal (º)

ϕ_k : in case of cohesion soil ground, 0, and in case of sandy ground, characteristic value of angle of shearing resistance in drained condition (º)

W_k : characteristic value of total weight of slice segment per unit of length, total weight of soil and water (kN/m)

x : horizontal distance between center of gravity of slice segment and center of circular slip failure (m)

P_H : horizontal action on slice segment of soil mass per unit of length in circular slip (kN/m)

a : length of arm from center of circular slip failure at position of action of P_H (m)

s : width of slice segment (m)

γ_S : partial factor multiplying the action term

γ_R : partial factor multiplying the resistance term

m : adjustment factor

The slice segment contains water mass(es) where no soil (including a structure) exists; in other words, it includes water from the water surface to the ground surface. When a circular arc does not reach the water surface, even when it has reached the ground surface, hydrostatic pressure should be applied to the vertical plane of the slice segment at the edge. Details for how to determine a slice segment and its weight (W_k and W'_k) in circular slip failure analysis are shown in **Part III, Chapter 2, 3.2 Shallow Spread Foundations**.

In **equation (4.2.1a)**, the length of the base of the slice segment (l) is used. In **equation (4.2.1b)**, the width (s) is used. The notation is different, but they are essentially same.

(2) In slope stability analysis, the causes of slip failure include the self-weight of the soil, surcharge, water pressure, wave pressure, and action due to seismic ground motion. Elements that resist slip failure include the shearing resistance of the soil and counterweight. Verification of safety against slip failure of slopes is performed assuming that the shearing force in the assumed slip surface falls below the expected shearing resistance of the soil. When assuming a circular slip failure surface, this is equivalent to the moments that cause slip falling below the moments that work to resist slip for the center of the circle.

(3) In the slice method used in circular slip failure analysis, the soil mass inside the slip circle is divided into a number of slices by vertical planes, the shearing stress at the bottom surface of each slice segment and the resistant stress of the soil estimated based on the failure criterion of the soil are calculated considering the balance of forces in each slice segment. The fact that the shearing resistance obtained by adding the stresses for all of the slice segments exceeds the shearing force along the slip line is then verified. In order to solve the inter-slice segment balance of forces in the slice method, it is necessary to assume statically the determinate conditions. Various methods have been proposed, which vary depending on the assumptions used. In general, the modified Fellenius method and the simplified Bishop method are used.

(4) Stability Analysis Method using Modified Fellenius Method[2)3)4)]

Various calculation methods have been proposed for the slice method, depending on how the forces acting on the vertical planes between the slice segments are assumed. The modified Fellenius method assumes that the direction of the resultant force acting on vertical planes between slice segments is parallel to the base of the slice segments. This method is also referred to as the simplified method or Tschbotarioff method. When a circular arc and a slice segment are as shown in **Fig. 4.2.1**, **equation (4.2.1)** according to the modified Fellenius method is applicable. In performing slope stability analysis, first, the center of the slip circle is assumed. Of the slip circles that take this point as their center, the one with the largest ratio of S_d (value obtained by multiplying S_k characteristic value of the shearing force or action moment caused by action) by partial factor γ_S to R_d (value obtained by multiplying R_k characteristic value of shearing resistance or resistant moment by partial factor γ_R) is obtained. Its value is used as the maximum ratio for that center point. The maximum ratio of S_d/R_d (shearing force (action moment)/shearing resistance (resistance moment) for other center points is then obtained by the same method. Verification can be performed for the limit state for slip failure of the slope by confirming that the value obtained by multiplying the maximum value of the maximum ratios obtained by the contour for the maximum ratios by adjustment factor m is 1 or smaller.

The equations below show the basic form for verification.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad S_d = \gamma_S S_k \qquad R_d = \gamma_R R_k \tag{4.2.2}$$

where

S_k : characteristic value for the action term

S_d : value used to design the action term

R_k : characteristic value for the resistance term

S_k : value expected in designing the resistance term

The equations below are obtained by converting **equation (4.2.1b)** in line with the conditions above.

$$m \cdot \frac{S_d}{R_d} \leq 1$$

$$S_d = \gamma_S S_k = \gamma_S \cdot \sum \left\{ (W_k + q_k)\sin\theta + \frac{1}{R} a P_{Hk} \right\} \tag{4.2.3}$$

$$R_d = \gamma_R R_k = \gamma_R \cdot \sum \left\{ c_k s + (W'_k + q_k)\cos^2\theta \tan\phi_k \right\} \sec\theta$$

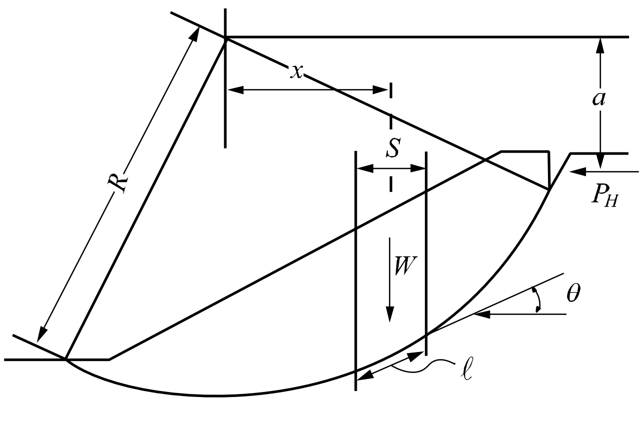

Fig. 4.2.1 Circular Slip Failure Analysis using Modified Fellenius Method

(5) Stability Analysis by Simplified Bishop Method [3)5)]

Bishop[5)] proposed an equation that considers the vertical shearing force and horizontal force acting in the vertical plane of a slice segment. In actual calculations, a calculation method assuming that the vertical shearing forces are in balance is often used, which is called the simplified Bishop method. In the simplified Bishop method, stability can be verified by (1) determining partial factors γ_S and γ_R (usually, $\gamma_S = \gamma_R = 1.00$), (2) calculating the minimum value of adjustment factor m that satisfies **equation (4.2.4)**[5)] with repeated convergent calculation, and (3) confirming that the value is larger than the standard lower limit of the adjustment factor.

$$\frac{m}{\gamma_R}\cdot\frac{\gamma_S\cdot\sum\left\{(W_k+q_k)\sin\theta+aP_{Hk}/R\right\}}{\sum\left[\left\{c_k s+(W'_k+q_k)\tan\phi_k\right\}\dfrac{\sec\theta}{1+\tan\theta\tan\phi_k/(m/\gamma_R)}\right]}\le 1 \tag{4.2.4}$$

where

R : radius of circular slip failure (m)

c_k : in case of cohesion soil ground, characteristic value of undrained shearing strength, and in case of sandy ground, characteristic value of apparent cohesion in drained condition (kN/m^2)

W'_k : characteristic value of effective weight of slice segment per unit of length (weight of soil. When submerged, effective weight in water) (kN/m)

q_k : characteristic value of vertical action from top of slice segment (kN/m)

θ : angle of bottom of slice segment to horizontal (º)

ϕ_k : in case of cohesion soil ground, 0, and in case of sandy ground, characteristic value of angle of shearing resistance in drained condition (º)

W_k : characteristic value of total weight of slice segment per unit of length, total weight of soil and water (kN/m)

P_{Hk} : characteristic value of horizontal action on soil mass of slice segment (kN/m)

a : length of arm from center of circular slip failure at position of action of P_H (m)

s : width of slice segment (m)

γ_S : partial factor multiplying the action term

γ_R : partial factor multiplying the resistance term

m : adjustment factor

The slice segment contains water mass(es) where no soil (including a structure) exists; in other words, it includes water from the water surface to the ground surface. When a circular arc does not reach the water surface even when it has reached the ground surface, hydrostatic pressure should be applied to the vertical plane of the slice segment at the edge. Details for how to think a slice segment and its weight (W_k and W'_k) in circular slip failure analysis are shown in **Part III, Chapter 2, 3.2 Shallow Spread Foundations**.

For details of the simplified Bishop method, **Part III, Chapter 2, 3.2 Shallow Spread Foundations** can be referred to.

(6) Applicability of Stability Analysis Methods[6)7)]

Verification results in stability analysis by the modified Fellenius method and the simplified Bishop method are in agreement for cohesive soil in which $\phi = 0$, when both partial factors γ_S and γ_R are 1.00, but differ when the circular arc passes through sandy ground. In Japan, circular slip failure analysis by the modified Fellenius method is widely used. This is because it has been reported that the modified Fellenius method reasonably explains the actual behaviors of slope failure based on the results of analysis of case histories of slip failures for banks in port areas in Japan,[4)] and also gives a safety side solution for sandy ground.

However, when a slip circle cuts through the foundation ground consisting entirely of sandy soil layers, or when a slip circle cuts through ground consisting of an upper thick sandy layer and lower cohesive soil layer, it is known that the modified Fellenius method tends to underestimate stability.[7)] From the viewpoint of the basic principles of the stability calculation method, evaluation by the simplified Bishop method is more accurate than that by the modified Fellenius method under such conditions. Therefore, the simplified Bishop method is generally used in case of eccentric and inclined loads, which are particularly a problem when examining the bearing capacity of mounds. It should be noted that the simplified Bishop method has the problem of overestimating adjustment factor m when actions on near-horizontal sandy ground apply vertical loads. In such cases, a method of stability calculation can be used which assumes that the ratio of the vertical to the horizontal forces between slice segments is 1/3.5 of the angle of slice segment inclination.[8)] In stability verification in this case, calculations are made using the following equation.

$$\frac{m}{\gamma_R}\cdot\frac{\gamma_S\cdot\sum\left\{(W_k+q_k)\sin\theta+aP_{Hk}/R\right\}}{\sum\left[\left\{nc_k s+(W'_k+q_k)\tan\phi_k\right\}\times\dfrac{\sec\theta}{n+\{\tan\theta-\tan(\beta\theta)\}\tan\phi_k/(m/\gamma_R)}\right]}\leq 1 \qquad \textbf{(4.2.5)}$$

Stability can be verified by (1) determining partial factors γ_S and γ_R (usually, $\gamma_S = \gamma_R = 1.00$), (2) calculating the minimum value of adjustment factor m that satisfies the **equation (4.2.5)** with repeated convergent calculation, and (3) confirming that the value is larger than the standard lower limit of the adjustment factor. The calculation procedures are the same as those for the simplified Bishop method.

Where $n = 1 + \tan\theta \tan(\beta\theta)$, β is a parameter that provides the ratio of the vertical force to the horizontal force acting on the sides of the slice segment, and can be assumed to be $\beta = 1/3.5$. The other symbols are the same as those in **equation (4.2.4)**.

4.2.2 Stability Analysis Assuming Slip Surfaces other than Circular Slip Surface

(1) Despite the provisions stated in the previous sections, a linear or a compounded slip surface shall be assumed in stability analysis when it is more appropriate to assume a slip surface other than a circular arc slip surfaces according to the ground conditions.

(2) When linear slip is assumed, examination of stability against slip failure of a slope with a straight sliding surface is calculated using the following equation.

$$m\cdot\frac{\gamma_S\cdot\sum\left\{(W_k+q_k)\sin\theta+P_{Hk}\cos\theta\right\}}{\gamma_R\cdot\sum\left[c_k\ell+\left\{(W'_k+q_k)\cos\theta-P_{Hk}\sin\theta\right\}\tan\phi_k\right]}\leq 1 \qquad \textbf{(4.2.6)}$$

where

c_k : characteristic value of cohesion of soil (kN/m^2)

ϕ_k : characteristic value of angle of shearing resistance of soil (º)

l : length of base of slice segment (m)

W'_k : characteristic value of effective weight of slice segment per unit of length (weight of soil. When submerged, effective weight in water) (kN/m)

W_k : characteristic value of total weight of slice segment per unit of length, total weight of soil and water (kN/m)

θ : inclination of base of slice segment, assumed to be positive in the case shown in **Fig. 4.2.2** (º)

P_{Hk} : characteristic value of horizontal action to soil mass of slice segment (kN/m)

γ_S : partial factor multiplying the action term

γ_R : partial factor multiplying the resistance term

m : adjustment factor

The slice segment contains water mass(es) where no soil (including a structure) exists; in other words, it includes water from the water surface to the ground surface. When a circular arc does not reach the water surface, even when it has reached the ground surface, hydrostatic pressure should be applied to the vertical plane of the slice segment at the edge. Details for how to determine a slice segment and its weight (W_k and W'_k) in circular slip failure analysis are shown in **Part III, Chapter 2, 3.2 Shallow Spread Foundations**.

When partial factors γ_S and γ_R have not been determined, they can be set as 1.00 in accordance with the conventional method and adjustment factor m can be set to a value equivalent to the conventional safety factor to verify the stability. In this case, adjustment factor m for slip failure can be 1.2 or more in the permanent situation and 1.00 or more for variable situations in respect of Level 1 earthquake ground motion.

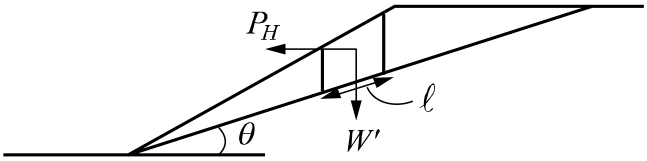

Fig. 4.2.2 Examination of Slope Stability Analysis using Linear Sliding Surface

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5 Soil Improvement Methods

5.1 General

(1) When facilities to be constructed are unstable under the given actions and original ground conditions, or the facilities are expected to be unable to fulfill the desired functions due to unacceptably large ground deformation during and after construction, the ground concerned is called soft ground. That is, whether or not the ground is defined as soft ground is not only determined by the ground conditions but is inseparably linked to the types and sizes of the facilities to be constructed, the construction speeds and the expected functions of the facilities. Soft ground requires countermeasures to cope with several problems, not only in the design and construction stages, but also in the post-construction stage. These countermeasures may be taken for coping with stability and deformation issues facing the facilities to be constructed, stabilizing the facilities during the temporary stage, treating groundwater during and after construction, alleviating adverse impacts on neighboring existing structures, and reinforcing existing structures.

The countermeasures against soft ground can be largely classified into the following four approaches.

① Approach to change the design of the facilities according to the ground conditions

(a) Reduction in loads or moment (use of lightweight materials and counterweight fill, etc.)

(b) Reduction in ground stresses by expanding the base areas of the facilities

(c) Avoidance of soft layers by adopting pile foundations

(d) Other measures (for example, the acceptance of settlement of the facilities concerned in synchronization with the surrounding ground so as to alleviate relative deformation in the case of widespread subsidence regions)

② Approach to replace soft soils with quality materials

③ Approach to temporarily or permanently improve soft materials so as to be suitable for the intended facilities

④ Approach to develop ground conditions suitable for the intended facilities by supplementing the soft ground with materials (supplemental materials) with property lacking in the soft ground
The soil improvement methods described in this section are ② to ④ above.

(2) The basic principles of soil improvement methods are (a) replacement, (b) consolidation and drainage, (c) compaction, (d) chemical and thermal stabilization, and (e) reinforcement. The soil improvement methods can be further classified into dozens of types (refer to **Table 5.1.1**). However, there are no soil improvement methods which can be applied to all cases. Thus, it is preferable to carefully select the soil improvement methods with due consideration to an accurate understanding of the physical and mechanical properties of the soft ground to be improved; clarification of the purposes of soil improvement in relation to several conditions such as the types, functions, importance and sizes of the facilities, and the implementation difficulties, construction periods, economic efficiency and environment impacts of the respective methods1), 2), 3).

(3) Targets when selecting soil improvement methods

The optimal soil improvement methods shall be selected in a manner that compares the economic efficiency and goal attainment levels of a few candidate methods selected based on the goals of soil improvement (types, sizes and required performance of facilities), the characteristics of the object soil, implementation difficulties, construction periods and the impacts on the surrounding environments[2)].

(4) Monitoring the states of ground during construction

Some soil improvement methods require certain periods until the development of the targeted ground strength with the original ground strength lost or significantly reduced during construction. When implementing such soil improvement methods, it is necessary to monitor the ground stability and earth pressure of the improved ground on the facilities, not only after but also during the construction of the facilities, and to pay attention to the procedures of facility construction and the time to commence the subsequent construction work.

(5) Examination of the impacts on the environment

Because there have been reports that some soil improvement methods using cement and cement-based binders pose a risk of causing the elution of hexavalent chrome from improved soil, depending on the conditions, with the concentration exceeding the environmental quality standards for soil, the Ministries of Construction and Transport (at the time) issued a notice in March 2000 for **immediate measures and their operation with respect to the use**

of cement and cement-based hardeners for soil improvement and the recycling of improved soil. Thus, when implementing soil improvement methods using cement and cement-based binders and recycling soil improved with such methods, it is necessary to conduct hexavalent chrome elution tests based on the **Guidelines for Hexavalent Chrome Elution Tests on Improved Soil Using Cement and Cement-Based Hardeners (Draft)** (Directors of Engineering Affairs Division and Government Buildings Department, Minister's Secretariat, the Ministry of Land, Infrastructure, Transport and Tourism, No. 16 and No. 1 of April 20, 2001) [4)].

Recently, slag has been recycled as soil improvement materials. When using slag as a recycled material, it is necessary to give due consideration to the measures to prevent environmental problems on the basis of the provisions in the related laws such as the Waste Disposal and Public Cleansing Act, the Act for the Prevention of Marine Pollution and Maritime Disasters, and the Soil Contamination Countermeasures Act. For the basic concepts of using slag as a recycled material, refer to the **Recycling Guidelines for Port and Airport Development (Revised Version)** [5)].

Table 5.1.1 Classification of Soil Improvement Methods Based on the Basic Principles

Basic principle	Name of method	Remarks
Replacement	Replacement method	Including blasting replacement and forced displacement methods
Drainage	Preloading and surcharge methods Vertical drain method Vacuum consolidation method	Relying mainly on the consolidation effect by the drainage of cohesive soil
	Dewatering method (well point and deep well methods)	Used mainly for lowering water levels through the drainage of sandy soil but also used for increasing consolidation loads
	Pore water pressure dissipation method	Liquefaction countermeasure
Compression	Sand compaction pile method	Applicable to both sandy and cohesive soil
	Rod compaction method	Including the density increase and compaction of sandy soil
	Vibro-flotation method	
	Heavy tamping method	
	Compaction grouting method	
Chemical stabilization	Deep mixing method	Including the improvement of base course materials
	Shallow mixing method	Including the improvement of base course materials
	Premix method Lightweight treated soil method Pneumatic flow mixing method	Improvement of soil from borrow pits as quality ground improvement materials for reclamation and backfilling
	Jet grouting method	
	Chemical grouting method	
	Quicklime pile method	Relying on the stabilization of columns
Thermal treatment	Freezing method	Mainly for temporary stabilization
Reinforcement	Reinforcement methods (sheet and net methods, etc.)	Including spread fascine and rope nets

(6) Improvement of cohesive ground and cohesive soil

① **Replacement method**

The replacement method is to partially or entirely replace the soft layers with quality soil and is expected to be reliably implemented over a short period of time[6)] .However, recently, the replacement method has become impossible to be implemented in many cases because of problems with the generation of turbid water and the difficulty in disposing of excavated cohesive soil as well as procuring replacement sand. In addition, loosely compacted replacement sand may cause an insufficient bearing capacity for large facilities and leave the possibility of liquefaction. The performance verification of the replacement method can be carried out with reference to **Part III, Chapter 2, 5.3 Replacement Methods**. The variations of the replacement method use granulated blast furnace slag as a backfill material for mooring and revetment facilities (refer to **Part III, Chapter 2, 5.7 Blast Furnace Granulated Slag Replacement Method**). Furthermore, one of the forced replacement methods uses sand compaction piles installed at high replacement area ratios.

② **Preloading and surcharge methods**

The preloading and surcharge methods are to achieve an expected increase in ground strength due to consolidation or reduction in settlement with pressure equivalent to the ground contact pressure of the facilities or higher applied to the foundation ground in advance of the construction of the facilities[7]. The preloading method expedites most of the consolidation settlement with fill having weight larger than the facilities to be constructed and is placed on ground surfaces, and enables the facilities to be finally constructed after removing the fill. The surcharge method is based on the same principle as the preloading method and enables the final facilities, such as the fill for roads and railroads, to be constructed in a manner that removes part of the fill used as preloads.

Generally, it is impossible to apply all the fill loads necessary for achieving the predetermined effects to the ground from the beginning without impairing its stability. Therefore, the fill loads are applied in stages while confirming the increases in ground strength. In addition, the preloading and surcharge methods are generally implemented in combination with the vertical drain methods for the purpose of accelerateing the consolidation periods. When implemented without combination with the vertical drain methods, because the consolidation of ground is close to primary consolidation where the degrees of consolidation in depth directions significantly vary, it is necessary to pay attention to the fact that the preloading and surcharge methods do not allow the distribution of the increment of ground strength in depth directions to be appropriately evaluated only by the average degrees of consolidation available through the settlement measured at ground surfaces. For the performance verification of consolidation settlement and the increases in ground strength through the preloading and surcharge methods, refer to **Part II, Chapter 3, 2.2 Physical Properties of Soil**, **Part II, Chapter 3, 2.3 Mechanical Properties of Soil**, and **Part III, Chapter 2, 3.5 Foundation Settlement.**

③ **Vertical drain method**

The vertical drain method is to artificially install vertical drainage layers (vertical drains) in cohesive soil ground so as to accelerate the consolidation periods[8]. The vertical drain method is generally implemented in combination with the preloading, surcharge or vacuum consolidation methods as the means to apply surcharges necessary to generate consolidation. Although vertical drains are effective to significantly reduce the construction periods, the vertical drain method still requires overall soil improvement periods of about one year in general and relatively cumbersome construction management.

The sand drain method, which uses sand piles as drainage layers, is one of the general variations of the vertical drain method, and a method using drain (prefabricated drain) materials made of synthetic resin or nonwoven fabrics in place of sand is also frequently used[8] . Sand piles made of bags filled with sand (packed drains) are also used for the purpose of facilitating construction management of the sand drain method and ensuring the continuity of sand piles in soft ground[9, 10, 11]. For the performance verification of the vertical drain method, refer to **Part III, Chapter 2, 5.4 Vertical Drain Method**.

④ **Vacuum consolidation method (neutral stress reduction method)**

The vacuum consolidation method is to increase consolidation effective stress by reducing the pore water pressure in the soil instead of applying surcharge to the ground as consolidation loads[12]. The vacuum consolidation method is generally implemented in combination with the vertical drain methods to accelerate consolidation. One of the characteristics of the vacuum consolidation method is that the method does not have stability problems because it does not require using surcharge (no increases in shear stresses associated with loading)[13]. Thus, the vacuum consolidation method can accelerate construction periods by eliminating the staged loading required for the preloading and surcharge methods. Furthermore, the vacuum consolidation method is advantageous when improving soil below deep seafloors because of the availability of large consolidation loads.

However, it has been pointed out that the vacuum consolidation method has lower ratios of consolidation than the preloading and surcharge methods, and the increase in ground strength in the early stages of consolidation is later in the vacuum consolidation method than in the preloading and surcharge methods[14] .

⑤ **Quicklime pile method**

The quicklime pile method is to improve ground with pore water in cohesive soil absorbed by the slaking of quicklime and has been used in many construction works on land such as reclamation[15]. Although there are some cases where soil improvement is expected to be achieved by an increase in the strength of quicklime piles stabilized through the slaking reaction, the increase in the strength of quicklime piles is subjected to the effects of the characteristics and the quantity of quicklime and the constraint conditions of the original ground. Thus, it is reasonable to limit the effects of the quicklime pile methods to the reduction in the water content of original ground through the slaking reaction of quicklime and the capillary water absorption power of hydrate lime. A method for estimating settlement and strength increase by assuming a surcharge equivalent to the reduction in water content is proposed for use in the performance verification of the quicklime pile method[16].

⑥ **Sand compaction pile method (for the improvement of cohesive ground)**

The sand compaction pile method is to improve ground with well-compacted, large-diameter sand piles constructed in cohesive ground[17, 18]. The method can reduce settlement amounts because of the concentration of surcharge loads on compacted sand piles having large stiffness and improve the stability of original ground as composite ground capable of resisting shear force with the undrained shear resistance of cohesive soil and friction resistance of sand piles. The performance verification of the sand compaction pile method can be carried out with reference to **Part III, Chapter 2, 5.10 Sand Compaction Pile Method (for the Improvement of Cohesive Ground)**.

⑦ **Deep mixing method**

The deep mixing method is to solidify ground with chemical reactions between the binders and soft ground in a manner that supplies chemical binders such as quicklime or cement deep in the ground, and forcibly mixes in-situ soft soil with the binders. The method is considered to be one of the most effective chemical ground improvement methods[19] and has several variations, with the most popular including those using slurry stabilization materials[20] applied to large-scale offshore and on land projects, and pneumatically transportable powdered binders[21] applied to small-scale on land projects. The performance verification of the deep mixing method can be carried out with reference to **Chapter 2, 5.5 Deep Mixing Method**.

⑧ **Jet grouting method**

The jet grouting method is to improve ground in a manner that cuts the ground with an injection of highly pressurized fluid and mixes the soil with stabilization materials[22] . There are several variations of the jet grouting method depending on the types of high pressure fluid (water and stabilization materials), intensity of pressure, flow rates and construction specifications. Generally, the jet grouting method can be implemented with compact facilities suitable for narrow construction sites and has large improvement strength. The performance verification of the jet grouting method can be carried out with reference to **Part III, Chapter 2, 5.19 Jet Grouting Method**.

⑨ **Lightweight treated soil method**

The lightweight treated soil method is to develop light and stable ground by mixing dredged cohesive soil or construction waste soil with lightweight materials (foam or expanded beads) and stabilization materials such as cement[23] . The types of treated soil with foam and expanded beads used as lightweight materials are called foam treated soil and expanded bead treated soil, respectively. The lightweight treated soil enables earthquake-proof facilities and reclamation land to be developed because of its characteristics of being lighter than normal earth and soil useful for reducing settlement when used as reclamation or backfill materials, and having large strength useful for reducing earth pressure on the occurrence of earthquakes. The performance verification of the lightweight treated soil method can be carried out with reference to **Part III, Chapter 2, 5.6 Lightweight Treated Soil Methods**.

⑩ **Pneumatic flow mixing method**

The pneumatic flow mixing method is a technology which adds chemical stabilization materials such as cement to dredged soil pneumatically transported from the seafloor with pump dredgers and mixes the dredged soil with the stabilization materials using the turbulence effect of plug flows generated inside pressure pipes[24, 25] . The characteristics of the pneumatic flow mixing method include the ability to mix materials while they are transported, thereby simplifying the stabilization facilities, low initial investment costs, and the availability of rapid large-scale construction with the use of large pump dredgers. The pneumatic flow mixing method has been used for reclamation, reduction in earth pressure, earthquake reinforcement, surface treatment and the widening of revetments. The performance verification of the pneumatic flow mixing method can be carried out with reference to **Part III, Chapter 2, 5.17 Pneumatic Flow Mixing Method**.

⑪ **Reinforcement methods**

Unlike the concept of normal soil improvement methods, which aim at improving the characteristics of the soil itself, reinforcement methods are for supplementing the characteristics lacking in original ground using reinforcement materials with a stronger resistance against tensile, shear and compressive forces than the original ground, and are installed on the surface of or in the original ground so as to enable the original ground and the reinforcement materials to jointly behave as compound ground. When defined in this way, the spread fascine and mattress methods can be considered the precursors of the reinforcement methods, and the sheet net, geo-textile and soil nailing methods, which have been practically used as auxiliary methods for earth covering works on soft ground since the 1960's, can also be classified as types of reinforcement methods[26)].

(7) Improvement of sandy ground and sandy soil

① **Pore water pressure dissipation method**

The pore water pressure dissipation method is to prevent the accumulation of excessive pore water pressure in a manner that quickly dissipates the amount generated during an earthquake from the ground through drains made of artificial materials or gravel with high permeability and built in ground with a risk of liquefaction, thereby alleviating the degree of liquefaction[27)] . The drains are normally constructed in the form of piles but there are cases of wall-type drains or continuous-type drains that are placed around the structures. The performance verification of the pore water pressure dissipation method can be carried out with reference to the **Reference 28)**.

② **Sand compaction pile method (for the improvement of sandy ground)**

The sand compaction pile method is to drive or install sand piles in the ground using vibration or impulsive loads, and has been the most widely used soil improvement method for sandy ground[18)] . Vibration hammers are generally used for driving and compacting sand piles, and recently, variations of the sand compaction method for statically compacting sand piles have been developed[29), 30), 31), 32)] .The performance verification of the sand compaction pile method can be carried out with reference to **Part III, Chapter 2, 5.9 Sand Compaction Pile Method (for the Improvement of Sandy Ground)**.

③ **Rod compaction method (vibrating rod method)**

The rod compaction method is to compact ground in a manner that inserts special rods to a predetermined depth in the ground using vibration hammers and supplies sand through the rods while making them vibrate[33), 34)] . This method has many variations depending on the types of vibration rods. Generally, there has been an increasing number of cases using the rod compaction method in soil improvement because it can be easily implemented compared to the sand compaction pile and vibro-flotation methods mentioned below. The performance verification of the rod compaction method can be carried out with reference to **Part III, Chapter 2, 5.11 Rod Compaction Method**.

④ **Vibro-flotation method**

The vibro-flotation method is to improve sandy ground by inserting rods (vibroflots), which vibrate in a horizontal direction, into the ground with water injected through the lower nozzles of the rods, then compacting the ground using the vibration and filling the gaps around the rods created by the compaction of the ground with gravel, crushed stones, sand and slag so as to enhance the vibration transmission and press-fit effects[35), 36), 37)] . This method has been the second-most popular method for compacting sandy ground in Japan, next to the sand compaction pile method. The applicable depth of the method is considered to be limited to approximately 14 m from ground surfaces for enabling filling materials to be properly installed[35)]. The performance verification of the vibro-flotation method can be carried out with reference to **Part III, Chapter 2, 5.12 Vibro-Flotation Method**.

⑤ **Heavy tamping method (dynamic consolidation method)**

The heavy tamping method is to compact the ground at a depth range of 5 to 15 m using weights having a mass of 10 to 25 tons, which are repeatedly dropped from a height of approximately 20 m to the surface of the ground[38)] . It is impossible to compact identical locations repeatedly because the surface of the ground is dented every time the weights are dropped. Therefore, the method is implemented in a manner that compacts previously arranged compaction locations in a reticular pattern in the areas necessary for the operation over several days and covers the areas with sand, then repeats the above processes. Although the improvement effects and depths of the method are depend on the ground conditions, such as the properties and thicknesses of the object sandy soil as well as groundwater levels, and the construction conditions such as the potential energy of the weights per drop, in addition to the numbers and intervals of drops, it is necessary to confirm the optimal conditions through field test. Because the principle of the method is the impact load applied to the ground surfaces, the existence of obstacles such as rocks with diameters exceeding 1 m in the ground do not affect the applicability of the method. Therefore, the method can be effectively used for ground such as waste landfill sites consisting of miscellaneous materials including bulk waste[39)].

⑥ **Deep mixing method**

Although having been conventionally used for the improvement of cohesive soil ground, the deep mixing method has started to be widely used as a liquefaction countermeasure for sandy ground in recent years. Examples of the deep mixing method implemented as a liquefaction countermeasure include cases where the method is applied to entire liquefaction layers and cases where the method is partially applied to locations arranged in a grid pattern to restrict the shear deformation of sandy ground[40)]. The performance verification of the deep mixing method can be carried out with reference to **Part III, Chapter 2, 5.5 Deep Mixing Method**.

⑦ **Jet grouting method**

The jet grouting method can be used for not only cohesive ground but also sandy ground. Recently, there have been cases of new methods developed with improved economic efficiency achieved through the optimization of construction specifications and the use of materials suitable for soil improvement that requires lower strength such as liquefaction countermeasures[41)]. The performance verification of the jet grouting method can be carried out with reference to **Part III, Chapter 2, 5.19 Jet Grouting Method**.

⑧ **Chemical grouting method**

The chemical grouting method is used for stabilizing ground or shutting off water flow using cement, cohesive soil, asphalt or several types of synthetic resin injected into the voids among sand particles[42)]. The method has been widely used for local improvement of sandy ground and liquefaction countermeasures[43), 44), 45)]. The performance verification of the chemical grouting method as a liquefaction countermeasure can be carried out with reference to **Part III, Chapter 2, 5.16 Liquefaction Countermeasures through Chemical Grouting**.

⑨ **Compaction grouting method**

The compaction grouting method is meant to compact the ground with injection materials with extremely low fluidity such as mortar, fluidized sand or plastic grout forcibly injected into the ground. Although it has been conventionally used for remedying settled buildings or filling voids in the ground, the compaction grouting method has been frequently used for liquefaction countermeasures, starting with the 1995 Great Hanshin earthquake. The method has also been applied to the improvement of ground immediately below existing structures and narrow places to which large construction machines are not accessible.

⑩ **Premix method**

The premix method is to develop ground with high seismic resistance through underwater reclamation using treated soil with stabilization materials such as cement and segregation preventive agents which are preliminarily added to and mixed with sandy soil used for reclamation[46)]. The method is characterized by the applicability to the reclamation of new ground or the backfill of excavated ground, and the abilities to utilize dredged soil, to complete soil improvement concurrently with ground development, and to reduce vibration and noise during construction. The scope of application of the method was originally liquefaction prevention; however, currently, it has been expanded to the reduction of earth pressure. The performance verification of the premix method can be carried out with reference to **Part III, Chapter 2, 5.8 Premix Method**.

⑪ **Reinforcement methods**

One typical reinforcement method applicable to sandy soil is a method for constructing retaining walls using reinforcement materials (galvanized steel plates) laid in backfill soil and simple wall surface materials in a manner that enables the reinforcement materials to add virtual cohesion to sandy soil, thereby reducing the earth pressure applied to the retaining walls. The method was first adopted successively by the ???? then Japan Highway Public Corporation and the Japan National Railway in 1972, and has been used in many projects since that time[47)]. In addition, variations of reinforcement methods include shallow treatment methods such as the sheet net, geotextile and soil nailing methods.

(8) Liquefaction countermeasures

① Because the settlement and deformation of ground as a result of liquefaction impairs the functions of facilities, it is preferable to implement liquefaction countermeasures if facilities include ground at risk for liquefaction.

② Liquefaction countermeasures shall be implemented with due consideration to the purposes of the facilities and the effects on existing facilities as well as the surrounding areas.

③ When implementing liquefaction countermeasures, it is preferable to give due consideration to the following items:

(a) The types of countermeasure work;

(b) The area of countermeasure work (planar area and depths); and

(c) Specific performance verification of countermeasure work.

④ The types of liquefaction countermeasure work are as listed in (a) to (c) below.

(a) Prevention of the generation of pore water pressure

1) Replacement method (replacement of existing soil with easily compactable sand)

2) Compaction methods (sand compaction pile method, rod compaction method, vibro-flotation method, heavy tamping method, static press-in compaction method, etc.)

3) Stabilization methods (deep mixing method, premix method, chemical grouting method, jet grouting method, etc.)

(b) Dissipation of excess pore water pressure

1) Replacement method (replacement of existing soil with coarse sand and gravel)

2) Pore water pressure dissipation method

(c) Combination of (a) and (b)

1) Simple combination of (a) and (b)

2) Combination of (a) and (b) in relation to facilities

⑤ The area of soil improvement as liquefaction countermeasures shall be determined for the purpose of maintaining the function of the facilities. It is preferable to implement soil improvement for ground expected to undergo liquefaction.

⑥ The area of soil improvement necessary for maintaining the functions of facilities shall be determined in consideration of the following items.

(a) Gravity-type quaywalls

1) Stability with respect to bearing capacity

2) Stability with respect to earth pressure at the back of quaywalls

3) Settlement of aprons

(b) Sheet pile quay quaywalls

1) Stability of sheet piles

2) Stability of anchorage work

3) Settlement of aprons

(c) Vertical piled piers

1) Stability of piled pier bodies

2) Stability of earth retaining sections

3) Settlement of aprons

⑦ When ground adjacent to soil improvement areas is expected to undergo liquefaction, buffer improvement areas shall be constructed to alleviate the effects of the ground subjected to liquefaction on the adjacent ground. For the determination of the necessary soil improvement areas, studies on the soil improvement areas through the deterioration and damage of regions affected by the propagation of excess pore water presssure[48] and studies on soil improvement areas through the finite element method and laboratory vibration tests[49] can be used as references.

⑧ In addition to the stability and other items listed in ⑥ above, there may be cases requiring the examination of ground stability with respect to slip circle failures and the necessity for implementing soil improvement for areas subject to slip circle failures for the purpose of ensuring stability of the areas. For the analysis of slip circle failures in these cases, refer to **Part III, Chapter 2, 3.2.5 Bearing Capacity with Respect to Eccentrically Inclined Actions**.

⑨ When using compaction methods as liquefaction countermeasures, compaction shall be applied to the ground until the *N*-value after compaction reaches a satisfactory level for liquefaction prevention as determined in **Part II, Chapter 7, 2 Prediction and Determination of Liquefaction**. The target *N*-value can also be obtained by using the results of cyclic triaxial tests of object soil layers.

⑩ When using stabilization methods as liquefaction countermeasures, because the stabilized bodies through stabilization methods reduce flexibility to cope with the deformation of the surrounding ground, there may be cases where the stabilized bodies undergo fractures associated with cracks due to the uneven settlement of lower layers, thereby causing differences in levels or cave-ins on the ground surfaces. Thus, it is necessary to pay attention to the behavior of the ground around the soil improvement areas.

(9) Temporary soil improvement

① **Dewatering method**

The dewatering method is used to take care of spring water for the safe implementation of excavation work when constructing mainly underground structures, and is classified as a temporary soil improvement method. It is necessary to select the appropriate drainage methods (deep well or well point methods) depending on the properties of the object soil[50]. There are cases of lowering groundwater levels as a kind of preloading to increase loads effective for expediting consolidation of the soil in deep layers. Recently, it has been pointed out that the over-consolidation effect of the dewatering method is effective for liquefaction countermeasures. The performance verification of the dewatering method can be carried out with reference to **Part III, Chapter 2, 5.14 Well Point Method**.

② **Freezing method**

The freezing method is to construct reinforcement walls or inpermeabile walls by stabilizing soil with water inside frozen soil. The method can be applied to both sandy and cohesive soil, allowing for stronger soil and a greater water sealing effect.

③ **Shallow mixing method**

The shallow mixing method is used for treating the surfaces of soft ground filled with cohesive soil in a manner that constructs slabs with treated soil prepared by mixing several binders. The treated surfaces are covered with sheets, nets or rope nets as auxiliary means of earth covers. The performance verification of the shallow mixing method can be carried out with reference to **Part III, Chapter 2, 5.15 Shallow mixing Method**.

5.2 Ground Investigations for Performance Verification of Soil Improvement

5.2.1 General

(1) In general, common preliminary investigations for performance verification shall be carried out for all facilities regardless of whether or not the facilities are associated with soil improvement. For the significance and contents of the preliminary investigations, refer to **Reference (Part II), Chapter 1, 3 Investigations and Tests Related to Ground**.

(2) Whether the ground is stable or requires soil improvement to cope with possible settlement or liquefaction shall be determined based on the results of preliminary investigations of the depths of bearing layers, stratification conditions, and strength as well as consolidation characteristics of the respective layers. Then, possible combinations of facility structures and soil improvement methods shall be selected as possible countermeasures. In this regard, reference can be made to the examples of several port facilities having similar ground conditions, particularly those of damaged facilities[51) to 61)].

(3) The parameters necessary for the performance verification shall be determined after selecting the optimal combination of a facility structure and a soil improvement method, as well as a performance verification method,

and conducting additional ground investigation as needed. In the performance verification stage, however, it is necessary to set hypothetical soil improvement characteristics as target values of soil improvement. Thus, it is important to implement soil improvement with proper quality management of the materials to be used, construction management and a post investigation to confirm the actual soil improvement characteristics.

5.2.2 Ground Investigations Related to Vertical Drain Method

(1) The improvement effect of the vertical drain method cannot be obtained until effective stresses are increased by the preloading, surcharge or vacuum consolidation methods. The improvement effect is gradually exerted as the progress of consolidation due to the increase in the effective stresses. Thus, the periods subject to investigations, tests and behavior monitoring for soil improvement through the vertical drain method shall be all the construction periods from the installation of sand mats and vertical drains to the completion of structures on the improved ground. In cases where the structures are expected to undergo settlement after their completion, it is preferable to continuously implement behavior monitoring.

(2) Investigations and tests for performance verification

Almost all information necessary in the performance verification stage can be obtained through the general investigations and tests described in **Reference (Part II), Chapter 1, 3 Investigations and Tests Related to Ground**. In the case of directly confirming the increase rates of strength through soil tests using specimens largely affected by the stress release when sampled from intermediate soil or deep layers, in addition to the estimation of the increase rates of strength based on the strength distribution in the depth direction of the cohesive layers and consolidation yield stresses, it is preferable to conduct triaxial compression tests using the recompression method as described in **Part II, Chapter 3, 2.3 Mechanical Properties of Soil**.

In addition, the continuity and permeability of sand layers are important items to investigate when expecting drainage performance of sand layers below vertical drains. If it is difficult to evaluate the continuity and permeability of sand layers based on the existing investigation results, additional boring surveys and supplemental sounding tests shall be conducted to confirm the continuity of sand layers and additional surveys shall be conducted to obtain permeability as needed.

(3) Quality of drains and sand mats

Sand with high permeability shall be used for sand drains and sand mats. It is important to ensure the quality of the sand through grain size tests to be conducted for each borrow pit before starting construction, and every time after the completion of predetermined completion quantities during the construction (together with permeability tests as needed). The frequency of the grain size tests is preferably once every completed quantity of 2,000 m^3. Although prefabricated drains are considered to have stable quality, it is preferable to confirm the quality of the products through field test results or performance records. According to the performance records of prefabricated drains, they have been installed mostly at intervals of 0.5 to 2.5 m, and those installed at intervals of 1.0 to 1.5 m account for about 80% of the total[62)]. Furthermore, prefabricated drains have been used for soil improvement to a depth of up to about 45 m[62)]. There may be cases of conducting special permeability tests when it is necessary to apply prefabricated drains to the soil improvement beyond the scope of the performance records[63)].

As for sand mats, the majority have been constructed with thicknesses ranging from 1.0 to 1.5 m for offshore sites and 0.5 to 1.0 m for on land sites. Every vertical drain shall be subjected to quality control and any defective drains shall be reinstalled. In cases where sand layers below vertical drains are expected to function as drainage layers, the vertical drains shall be reliably connected to the sand layers.

Table 5.2.1 Quality and Work Progress Control Items for Sand Mats and Vertical Drains

Quality and work progress	Control method
Sand material	Grain size tests for the respective borrow pits and every time after predetermined complete quantities with permeability tests as needed
Drain products	Manufacturers' performance test results and special permeability tests as needed
Sand mats	Control of thicknesses
Drain installation intervals	Surveying

Quality and work progress	Control method
Installation depth and continuity	For sand drains, the ratios of casing depths to the readings of sand level sensors. For prefabricated drains, the confirmation of the prevention of drain materials from being lifted up together with the casings, and the extended lengths of drain materials through construction management and records. When sand layers below vertical drains are expected to function as drainage layers, connection of drain materials to the sand layers.

(4) Investigations, tests and behavior monitoring during construction

When using fill for vertical loading, the fill is preferably constructed in stages. Furthermore, while applying vertical loading through the staged construction of fill, it is necessary to confirm that vertical loading has been implemented as planned, monitor the behavior of the ground, and verify the increase in strength in every stage through boring surveys because the stability of fill at each stage and the deformation of ground, as well as the dissipation of pore water pressure during the consolidation time with fill, are not always as predicted in the design stage. Recently, there have been cases of installing sand drains using sand compaction barges. In general, because sand compaction barges enable sand piles with larger diameters to be installed than sand drain barges, the intervals of sand piles required for achieving an identical improvement rate are larger when using sand compaction barges than when using sand drain barges. Although whether sand piles are installed with sand compaction barges or sand drain barges must not cause any difference in the performance verification results of improvement effects in theory, it is necessary to confirm the actual improvement effects through behavior monitoring because there have not been enough performance records to verify the theory.

The investigations, tests and behavior monitoring in each construction stage shall be implemented not for simple confirmation but for the correction of the information on the ground and inaccuracies in the predictions, as well as the revisions of construction plans (loading rates and consolidation time) when carrying out the performance verification, thereby achieving safe implementation and reliable improvement effects.

Measuring devices such as settlement plates are obstacles from the viewpoint of construction, during which they have a risk of being broken. Thus, it is preferable to use the measuring devices flexibly with the mechanisms according to the behavior of the ground. It is also necessary to use not only one device for each measuring item but also multiple devices as backups in case of failures, with different devices arranged in a manner that enables measurement of one device to be cross-checked with measurements from other devices. For measuring methods, measuring devices, points of caution during measurements, and methods for organizing and analyzing the measuring results, refer to the **Reference 64)**.

In addition, the arrangement and structural types of those measuring devices continuously used for maintenance after the completion of improvement work shall be determined in consideration of the plans of the structures constructed on the ground concerned and the measuring periods.

Table 5.2.2 Measuring and Control Items during Fill Loading

Measuring and control item	Purpose	Measuring method
Settlement at the center of fill	Stability and consolidation (settlement) control	Settlement plates and settlement gauges by layer
Settlement by layer at the center of fill	Consolidation (settlement) control	Settlement gauges by layer
Horizontal displacement in the ground at slope and slope toe	Stability control	Inclinometers
Vertical and horizontal displacement of the ground at slope toes	Stability control	Displacement piles and inclinometers
Vertical and horizontal displacement in the ground at slope toes	Detection of adverse effects on neighboring structures	Displacement piles and inclinometers
Increment of load and loading rate	Stability and settlement control	Earth pressure gauges, layer thickness control data, field density tests and measurements using RI
Water levels in fill	Measurement of changes in effective load due to settlement	Water level gauges

Measuring and control item	Purpose	Measuring method
Pore water pressure	Consolidation control	Pore water pressure gauges
Increase in ground strength	Consolidation control and confirmation of the stability of the loading for the next stage	Verification boring (sampling + soil tests) and sounding

① **Stability control**

Stability control is implemented to ensure the stability of fill at each stage. The stability of fill on soft ground is largely affected by the increments of stress at each stage, slope gradient of fill, ground strength before staged fill loading and loading rates. The stability of fill is generally examined for the final shapes of fill at the respective stages using slip circle analyses. However, the slip circle analyses are not enough for daily stability control because such calculations do not provide information on ground deformation. Thus, there have been proposals for methods qualitatively evaluating whether or not the fill has been destabilized by visualizing the displacement in progress and loads. For the organization method of measured data and control values, refer to the **References 64)** to **67)**.

The cross sections subject to measurement for stability control shall be selected by comprehensively evaluating the stability calculation results and stratification conditions as well as the inclination of the ground. When fill shows signs of a significant increase in instability, it is necessary to immediately take measures such as lowering the fill loading rates or observing its behavior when fill loading is temporarily stopped. Thus, the interval of measurements shall be determined in relation to the loading rate so as to enable changes in the behavior of the ground to be detected at each stage of fill.

② **Control during consolidation time (settlement control + ground investigations and tests)**

The purpose of stability control during the consolidation time is the confirmation and correction of the appropriateness of the initially predicted settlement and increases in strength with respect to actual chronological changes. There are several factors which cause discrepancies between the initial predictions and actual measurements including errors in the information on the ground used for initial predictions such as soil layer compositions, consolidation characteristics and thicknesses of the respective layers, drainage conditions and groundwater levels, and construction performance such as errors in loading, delays in consolidation at the depth of the ground due to defective drainage work, and the effects of shear deformation and lateral flows on actual settlement. Thus, stability during the consolidation time shall be comprehensively determined by not only measurement of the settlement of ground surfaces, but also the settlement of the respective layers, lateral displacement, and the investigation and test results in advance of each stage of fill.

The methods frequently used for settlement control include those estimating the final settlement focusing only on the settlement of the layers to be improved (such as the hyperbolic, Hoshino, Asaoka and Kadota methods)[67)] . It has been said that these methods require actual measurement data when the consolidation degrees are 60 to 80% or more for the prediction of settlement, with an error range of 10% or less[65), 67), 68)] . The positions of measuring the settlement of each layer shall correspond to the classification of the layers in the design stage and are preferably set in the appropriate ranges.

③ **Evaluation of the influences on the surrounding environments**

In cases of possible influences on the surrounding structures such as the lateral displacement of the surrounding ground during fill loading and the dragging down of the areas around the ground subjected to settlement due to fill loading, it is necessary to monitor the behavior of the surrounding areas with displacement piles and inclinometers installed around fill.

(5) Long-term measurements for maintenance

Settlement due to unfinished primary consolidation and secondary consolidation after completion of the structures is the subject of long-term measurements for the maintenance of the vertical drain method. For long-term measurements, some of the measuring devices used during the construction are continuously used. In cases where uneven settlement is more critical than the absolute values of the residual settlement, it is preferable to conduct measurements once a year or more using settlement plates installed at appropriate positions determined by referring to the trends of settlement during the construction periods.

5.2.3 Ground Investigation Related to the Sand Compaction Pile Method (for Improvement of Cohesive Ground)

(1) In the implementation of the sand compaction pile method, the installation of sand piles causes the heaving of ground surfaces in a manner that reduces the strength of the existing cohesive soil ground due to the disturbances and discharges which push the cohesive soil in lateral and upward directions. The heights and shapes of the heaving ground surfaces depend on the areas subjected to soil improvement, the lengths of sand piles, replacement area ratios and the directions of the sand piles. There are cases where the heaving of ground surfaces reaches several meters and causes difficulties in continuing soil improvement in shallow water areas. Conventionally, fill has been removed after the completion of soil improvement, but there have been increasing numbers of cases of continuously using fill as part of the foundation ground. Thus, it is necessary to preliminarily evaluate not only the behavior of the ground, but also the changes in ground shapes during construction. Countermeasures against the lateral displacement of the ground due to the installation of sand piles include a method which installs displacement absorbing holes[69)].

It is also necessary to examine the availability of the required quantities of sand and soil disposal sites in the event that it becomes necessary to dispose of the heaved soil.

(2) Investigations and tests for performance verification

Almost all the information necessary for the performance verification stage can be obtained through the general investigations and tests described in **Reference (Part II), Chapter 1, 3 Investigations and Tests Related to Ground**. Changes in the shapes and behavior of the ground during construction vary depending on the methods for installing sand piles, and, therefore, it is necessary to utilize the information from past construction works which are similar to the ones being planned. There has been a proposal of an empirical equation based on abundant performance records, which can be used as reference for the method of sand pile formation by vibro-driving and vibro-removal[70)].

(3) Quality of materials used for sand compaction piles and sand mats

Sand mats are laid on the ground before installing sand compaction piles. Sand mats have various functions including acting as horizontal drainage layers, ensuring the workability of construction machines for on land work, and increasing overburden pressure for controlling the disturbance and lateral displacement of original ground as well as to prevent turbidity while installing sand piles in the case of offshore work. The thicknesses of the sand mats are mostly in the range of 1.0 to 2.0 m for offshore work, and 1.0 m for on land work. The sand used for sand compaction piles needs to have the appropriate strength and permeability to function as sand piles and drainage layers, respectively. For the quality of the sand to be used for sand compaction piles, reference can be made to the grain size distribution curves of sand materials used in past construction in **Part III, Chapter 2, 5.10 Sand Compaction Pile Method (for Improvement of Cohesive Ground)**. The sand used for sand mats needs to have the same quality as that used for sand compaction piles. It is important to ensure the quality of the sand through grain size tests to be conducted for each borrow pit before starting construction and every time after the completion of predetermined completion quantities during the construction (together with permeability tests as needed). The frequency of the grain size tests is preferably once every completed quantity of 2,000 m^3.

When using slag as a recycled material, it is necessary to give due consideration to measures to prevent environmental problems based on the provisions of the related laws. For the basic concepts of using slag as a recycled material, refer to the **Recycling Guidelines for Port and Airport Development (Revised Version)**[5)].

(4) Investigations, tests and behavior monitoring during construction

It is necessary to investigate and confirm the appropriateness of the expected changes in the shape of the ground and the strength of original ground which are set in the design stage of soil improvement through the sand compaction pile method based on empirical equations and the existing construction information. In the event of discrepancies between the designed and actual changes, there may be a necessity to revise the initial design and take countermeasures.

When applying a surcharge to ground to be improved in expectation of the consolidation acceleration effect, stability and settlement control are required during construction of the structures as in **Part III, Chapter 2, 5.2.2 Ground Investigation Related to Vertical Drain Method**.

① **Records of pile installation work**

The work progress of sand compaction piles (installation positions, the depths at the lower ends of the sand compaction piles, crown heights and input quantities of sand indicating the replacement area ratios) shall be recorded and controlled for each sand pile. When using the sand pile formation by vibro-driving and vibro-removal, it is necessary to confirm whether or not the predetermined quantities of sand are input when pulling out the casings by comparing the pull-out lengths with the measurement results of sand level sensors in the casing pipes and to reinstall the casings so as to expand the sand piles to achieve the predetermined diameters. When improving heaved soil, the elevations of the original ground shall be measured every time a sand compaction pile is installed and the soil shall be improved up to the crowns. The records of the sand pile formation by vibro-driving and vibro-removal for the respective sand piles are of importance in the control of work progress and in the facilitation of quality control of the sand piles.

② **Investigations of the changes in ground shapes and behavior of the ground due to pile installation**

The thickness of the sand mats and the heaving states of the ground due to the installation of sand compaction piles shall be investigated in a manner that surveys the elevations of the original ground, the ground surfaces after laying sand mats and the ground surface after installing sand piles in areas including fringes with widths 1.5 to 2.0 times the pile lengths. In the event of large discrepancies in swell heights or shapes between the predictions and actual values, the initial design needs to be revised.

It is thought that the cohesive soil between sand piles and near the improved ground loses strength when disturbed by the installation of sand piles but then gradually restores its strength. In cases where a reduction in the strength of disturbed cohesive soil is thought to affect the stability of the ground with a surcharge applied to it, the restoration states of the strength shall be confirmed through ground investigations to be carried out for the soil between the sand piles and outside the improvement areas.

③ **Confirmation of the quality of sand compaction piles**

Standard penetration tests shall be carried out at pile center positions to confirm the strength and continuity of the sand compaction piles. The frequency of the standard penetration tests shall be determined in accordance with the complexity of the object ground, the importance of the structures, and the number of sand piles and construction machines. The frequency in past examples is once in 50 to 100 sand piles for soil improvement work with the total number of sand compaction piles at approximately 500. The frequency is likely to decrease with an increase in the total number of piles. (Refer to **Fig. 5.2.1**)[71)].

The strength parameters of sand compaction piles used in the performance verification are determined based on past examples. Thus, supplemental installation shall be considered in cases where the strength parameters obtained through the *N*-values at pile centers are smaller than those set in the performance verification.

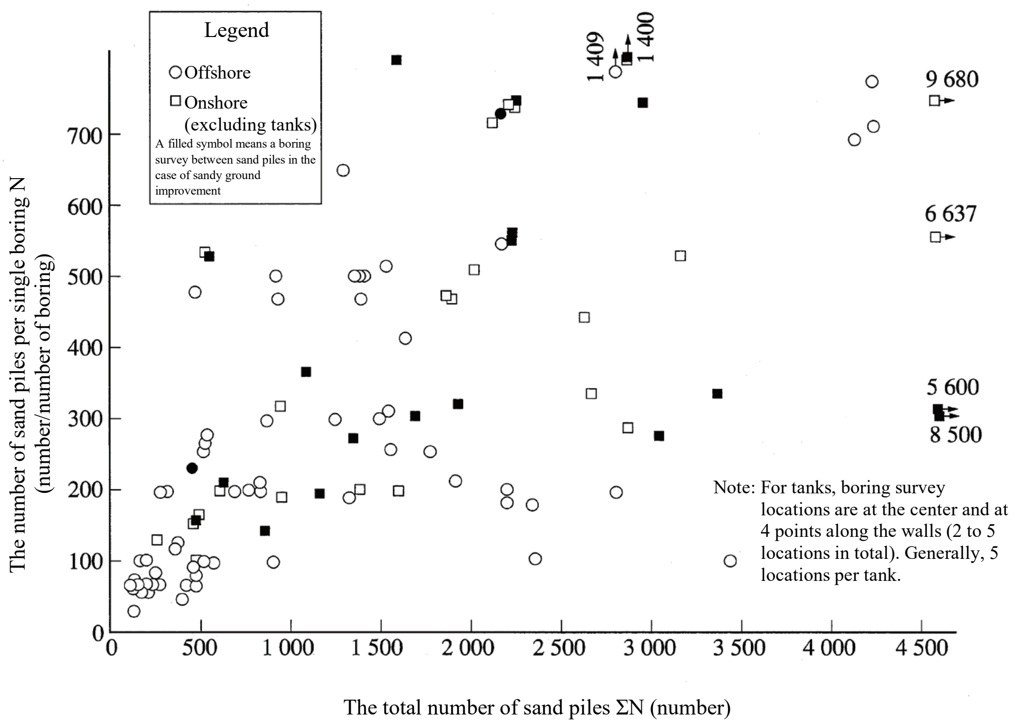


Fig. 5.2.1 Examples of the Relationship between the Total Number of Sand Piles and the Number of Sand Piles per Single Boring[71)]

Table 5.2.3 Quality and Work Progress Control Items for Sand Compaction Piles for the Improvement of Cohesive Ground

Quality and work progress	Control method
Sand material	Grain size tests for respective borrow pits and every time after predetermined complete quantities
Sand mats	Control of thicknesses
Sand Pile installation position	Surveying
Work progress (length, diameter and continuity) of sand piles	Comparison between casing depths and sand level and confirmation through construction management and records
Shape of heaved soil	Surveying before, during and after construction (bathymetric surveying)
Quality of sand piles (strength and continuity)	Standard penetration tests
Reduction and restoration of the strength of original ground	Unconfined compression tests of samples taken from points between the piles and fringes of improvement ground and sounding as needed

(5) Investigations, tests and behavior monitoring during the construction of superstructures

In cases of cohesive ground improvement with low replacement area ratios and a reliance on an increase in strength due to consolidation, it is of importance to monitor the behavior of superstructures such as rubble mounds placed on the ground to be improved as surcharges. The methods for behavior monitoring, investigations and tests shall be determined in accordance with **Part III, Chapter 2, 5.2.2 Ground Investigation Related to the Vertical Drain Method**.

In cases of cohesive ground improvement with high replacement area ratios and no expectation of an increase in strength due to consolidation without a reliance on an increase in strength due to consolidation, it is also preferable to measure settlement for the prediction of settlement in the future, although the importance of measuring settlement during construction is low compared to cases of soil improvement with small replacement area ratios. Furthermore, an increase in replacement area ratios causes the original ground to have larger changes in shape due to heaving, and causes the ground between the piles and around the improvement areas to have a larger reduction in strength. Ground around the improvement areas requires a particularly long period of time to restore its strength. When investigation results of these control items show large influences on the superstructures, it is necessary to take careful countermeasures including revisions of the initial design.

5.2.4 Ground Investigations Related to the Deep Mixing Method

(1) In the deep mixing method, soft soil is mixed with binders in situ.

The quality of stabilized soil obtained by mixing soft soil with binders is generally evaluated with the average strength of in situ treatment soil and a variation coefficient as the acceptance criteria. Because of the difficulty in taking remedial measures for improved soil whose quality cannot satisfy the acceptance criteria, it is necessary to implement quality control with a particular focus on the factors that affect the improvement effects. The factors that affect the strength of improved soil are classified into the four items described below and listed in **Table 5.2.4**.

① **Characteristics of binders**

The binders shall be selected based on the suitability of their characteristics to the site conditions. When using slurry-type binders, ordinary Portland cement or Portland blast furnace slag cement conforming to the JIS Standards is normally used as the original material for the binders. Furthermore, the use of cement-based binder needs to be examined when preliminary mix proportion tests show the difficulty in achieving the desired improvement effects using cohesive or organic soil having high water contents. Cement-based binder, which are based on cement with additional special components, and grain sizes modified in accordance with the usage purposes, are not standardized products.

② **Characteristics of object soil for improvement**

In the deep mixing method, which improves soft soil in situ, taking remedial measures is impossible in many cases. The factors affecting the strength of stabilized soil include the water and organic contents of original soil, grain size distributions, types of clay minerals and pH values of water in the soil. In particular, many organic substances are hazardous to the chemical reactions of the binders, and the contents and types of organic substances largely affect the strength of the stabilized soil. When the improvement soil has high organic contents, it is necessary to take measures such as the use of special binders for high organic soil[72)].

③ **Degree of mixing**

The degree of mixing, which is affected by the mixing mechanisms and speeds of the selected machines and the amounts of binders, is one of the important control items at construction sites. In laboratory mix tests, the mixing of binders is standardized so as to ensure the reproducibility and versatility of the test results, thereby facilitating the comparative evaluation of actual data with the laboratory mix test results.

④ **Curing conditions**

Material age (curing period) is one of the curing conditions of the deep mixing method, and the strength of improved soil is proportional to the logarithm of the material ages[73)] . Humidity does not affect strength as long as the improvement soil is located below sea surfaces or groundwater levels. In addition, temperature affects the expression of strength but does not affect long-term strength[74)] .

Table 5.2.4 Factors Affecting the Improvement Effects of Cement and Cement-Based Binders

Control item	Influence factor	Remarks
① Characteristics of binders	Type of binder Quality of binder Quality of additive Quality of mixing water	Cement and cement-based binders conforming to JIS standards
② Characteristics of object soil for improvement	Physical, chemical and mineralogical characteristics Organic content pH of pore water Water content	In the deep mixing method, which improves soft soil in situ, it is impossible in many cases to take remedial measures
③ Degree of mixing	Degree of mixing Time of mixing and remixing Additive amounts of binders	Evaluation of the additive amounts of binders in laboratory mix tests Control of the degree of mixing at construction sites
④ Curing conditions	Material age Humidity and temperature Repetition of drying and humidification, freezing and thawing, and confining pressure	Humidity does not affect strength Temperature affects the expression state of strength but does not affect long-term strength

When binder slurry is injected into and mixed with soil, the ground undergoes lateral displacement or heaving. It is necessary to take appropriate measures for the lateral displacement of soil which may have adverse impacts on neighboring structures if any. Recently, construction machines which can alleviate lateral displacement of soil have been commercialized. Furthermore, it is necessary to examine the methods in advance for removing heaved soil if needed.

(2) Investigations and tests for performance verification

In the performance verification stage, the characteristic values of improvement bodies shall be appropriately set with reference to past examples. The characteristic values of ground outside the improvement areas can be obtained with reference to **Reference (Part II), Chapter 1, 3 General Ground Investigations and Tests**. In cases where the deep mixing method is one of the options for a soil improvement method, the pH values of pore water in the respective layers and organic contents shall be added to the items to be tested. Together with other ground characteristics (liquid limits, natural water contents, grain size distribution, etc.), the results of these investigations and tests provide useful information when determining the classifications of soil layers for which laboratory mix tests are carried out and when analyzing the laboratory mix test results.

(3) Strength and quality of improved bodies

① Laboratory mix test

The laboratory mix tests are carried out to determine the types and amounts of binder necessary to ensure the strength of stabilized soil as specified in the performance verification. For the implementation method of the laboratory mix tests, refer to the **Technical Manual for the Deep Mixing Method in Ports and Airports**[4)]. The preparation and curing of specimens for testing stabilized soil can be carried out with reference to the **Practice for Making and Curing Stabilized Soil Specimens without Compaction (JGS 0821-2009)**[75)] by the Japanese Geotechnical Society. In addition, strength tests can be carried out with reference to the **Method for Unconfined Compression Tests of Soil (JIS A 1216)**[76)] .

The information obtainable through laboratory mix tests is the strength of stabilized soil mixed with predetermined degrees of mixing. Thus, it is necessary to pay attention to the fact that the strength set in the performance verification needs to be corrected with an empirical correction coefficient while taking into consideration the effects of the characteristics of construction machines, ground conditions and variation coefficient for strength so as to be equivalent to the strength obtained through the laboratory mix tests to be used for determining the amount of binder . It shall also be noted that because the capacity of construction machines is set to ensure the intended mixing efficiency under the mix conditions with a large number of performance records, the discharge performance of grout pumps is limited. The construction barges generally used for the deep mixing method tend to have large variation coefficient for the strength of stabilized soil when slurry additive amounts become lower than 90 L/min[77)] . Thus, practical experience, including an understanding of the constraint conditions of the construction machines, is required for setting parameters for the laboratory mix tests.

② Hexavalent chrome elution test

When setting the mix conditions at construction sites, it is necessary to confirm that the rate of hexavalent chrome elution is equal to or less than the environmental quality standards for soil (0.05 mg/L). The specimens subjected to hexavalent chrome elution tests shall be selected from specimens (with a material age of seven days) which are used for laboratory unconfined compression tests and have mix proportions closest to the actual proportions for the respective soil layers or soil types. The implementation method for the hexavalent chrome elution tests shall conform to the **Guidelines for Hexavalent Chrome Elution Tests on Improved Soil Using Cement and Cement-Based Hardeners (Draft)** (the Directors of the Engineering Affairs Division and Government Buildings Department, Minister’s Secretariat, the Ministry of Land, Infrastructure, Transport and Tourism, No. 16 and No. 1 of April 20, 2001). Furthermore, the **Reference78)** can be used as a reference for implementation of the test.

(4) Investigations, tests and behavior monitoring during construction

The quality of heaved sections associated with the deep mixing method varies because the general scope of work progress control does not include heaved sections. Thus, it is necessary to fully understand the characteristics of heaved sections through, for example, topographic surveys for identifying their shapes and boring surveys for verifying their quality when utilizing heaved soil, and to revise the initial design if necessary.

Because the improvement effects of the deep mixing method are expressed without the deformation of the ground, unlike in the case of methods for improving the ground through consolidation, the deep mixing method does not require settlement and stability control after the expression of certain degrees of strength. However, because the improved ground immediately after construction has a loss of strength, in the cases of soil improvement work implemented with large daily construction volumes or those close to existing facilities, it is preferable to monitor the displacement of the existing facilities and the deformation of the surrounding ground.

Furthermore, the quality and work progress of the deep mixing method shall be ensured in a manner that confirms a reliable supply of binder with a predetermined quality and quantity and implementation of the necessary mixing, in addition to control of the installation positions and depths, verticality and the connection of improved bodies to bearing layers. The control items of the deep mixing method using slurry-type binders are shown in **Table 5.2.5**[79)]. The measured values of the control items shall be recorded for each improvement column for the enhancement of the quality control of improvement work. In addition, the strength and adverse effects on the surrounding environments if any shall be tested with specimens of predetermined material ages which are removed through boring and sampling.

Table 5.2.5 Control Items of the Deep Mixing Method Using Slurry-Type Binders[79)]

Factor affecting the strength of the improved body	Control item	Measuring equipment
Quality of binder	Composition ratios by weight of water, cement and additives for each batch (including the quality control of cement and additives and characteristic control of slurry)	Weight scales
Additive amounts of binders	Additive amounts of binder slurry per unit volume of original ground	Flow meters
Degree of mixing	Elevating speeds of improving machines and the brade rotation number	Elevating speed meters and rotation indicators
Work progress	Installation positions, installation depths (upper and lower ends of improvement columns) and verticality (inclinations of mixing shafts)	Surveying equipment, depth meters and inclinometers
Connection of improved body to bearing layer	Penetration rates, suspension loads, torque and installation depths (when improvement bodies are required to be connected to the bearing layers)	Load cells, hydraulic gauges, ammeters and depth indicators

① **Quality confirmation after installation**

Although soil stabilized through the deep mixing method expresses about 70% or more of the design strength within one week, the quality of stabilized soil is generally controlled with 28-day strength. The quality of stabilized soil is generally confirmed through the visual inspections of core samples of stabilized soil for the continuity and 28-day strength and variation coefficient of 28-day strength.

Dozens of test results are thought to be required to effectively evaluate the variation coefficient[79)] . The number of locations of boring surveys to obtain samples varies depending on the size of construction and site conditions, but the target number of locations in the case of offshore construction is one boring survey for every treated volume of 10,000 m^3[79)] . The specimens for unconfined compression tests are generally collected from three locations on the improved column bodies: the upper, middle and lower portions. The sampling frequency and locations above are for soil improvement to achieve a uniform strength in the depth direction (by changing the mixing conditions in accordance with the respective layers to be improved).

When allowing the respective layers to have different expressions of strength (in the case of an improvement with the mixing conditions fixed in the depth direction), it is preferable to take samples from the layers and evaluate their average strength and variation coefficient.

Because the evaluation in this method is dependent on the quality of the sampling techniques (the appropriateness of the selected sampling method and the skills of the operators), it is necessary to allow tests to be conducted with good quality samples. Sampling has often been implemented with rotary-type double tube samplers or triple tube samplers.

Furthermore, sampling shall be conducted immediately before the stabilized soil reaches the material ages for unconfined compression tests to the extent possible. Samples shall be cured with due consideration to keeping them from being damaged by impacts or drying.

② **Hexavalent chrome elution tests and tank leaching tests after installation**[78)]

When improving volcanic cohesive soil, it is necessary to conduct hexavalent chrome elution tests for the actual stabilized soil regardless of the results of these tests following the mix proportion tests. The specimens subjected to the hexavalent chrome elution tests shall be those used for the confirmation of field density or quality control in terms of unconfined compressive strength, or the specimens sampled concurrently with those for unconfined compressive strength tests (with a material age of 28 days). The frequency of the hexavalent chrome elution tests varies depending on the volume of stabilized soil in construction, and in cases of construction with a stabilization volume of 1,000 m^3 or less and a stabilization volume of 1,000 to 5,000 m^3, the required number of specimens is generally one and three, respectively. In the case of construction with a stabilization volume of 5,000 m^3 or more, the number of specimens is generally one for every stabilized soil of 1,000 m^3[78)].

In addition, construction with a stabilization volume of 5,000 m^3 or more, or with 500 or more stabilized columns, is subjected to tank leaching tests in addition to hexavalent chrome elution tests. The tank leaching test shall be conducted for one specimen sampled from the location showing the maximum elution values in the hexavalent chrome elution tests.

5.2.5 Ground Investigations Related to the Sand Compaction Pile Method (for the Improvement of Sandy Ground)

(1) In the sand compaction pile method, among the volume of the materials pressed into the ground, the portion which does not contribute to the compaction of surrounding ground occurs in the form of lateral displacement or heaving of ground surfaces. In the case of implementing the sand compaction pile method close to existing structures which place strict limitations on ground displacement, it is necessary to predict ground displacement and monitor the behavior of the ground during the implementation of the method.

(2) Investigations and tests for performance verification

The performance verification of improved loose sandy ground through the sand compaction pile method requires investigations of the layer compositions, N-values obtained through standard penetration tests, and grain size distribution. Details regarding the investigations of these factors are described as the standard investigation items in **Reference (Part II), Chapter 1, 3 Standard Investigation Results on Investigations and Tests Related to Ground**.

When seismic response analyses are necessary to verify the required performance, ground information corresponding to the analysis methods needs to be investigated. However, in the case of using the FLIP, there have been proposals of methods which allow the parameters to be simply set using N-values obtained through the standard penetration tests[80)] .

(3) Quality of sand compaction piles

The basic requirements of the sand used for sand compaction piles include having a strength suitable for sand pile materials, low contents of fine-grained particles and no grain fragmentation during construction. It is important to ensure the quality of the sand through grain size tests to be conducted for each borrow pit before starting construction and every time after the completion of predetermined completion quantities during the construction.

When using slag as a recycled material, it is necessary to give due consideration to measures in order to prevent environmental problems based on provisions in the related laws. For the basic concepts of using slag as a recycled material, refer to the **Recycle Guidelines for Port and Airport Development (Revised Version)**[5)] .

(4) Investigations, tests and behavior monitoring during construction

① **Records of pile installation work**

The work progress of sand compaction piles (installation positions, the depths at the lower ends of the sand compaction piles, crown heights and input quantities of sand indicating replacement area ratios) shall be recorded and controlled for each sand pile. When using the sand pile formation by vibro-driving and vibro-removal, it is necessary to confirm whether or not the predetermined quantities of sand are input when pulling out the casings by comparing the pullout lengths with the measurement results of the sand level sensors in the casing pipes, and to reinstall the casings so as to expand the sand piles to achieve the predetermined diameters. The records of sand pile formation by vibro-driving and vibro-removal for the respective sand piles are important for the control of work progress and in the facilitation of the quality control of sand piles.

② **Ground displacement associated with installation**

In cases of possible adverse effects on neighboring structures, it is necessary to measure ground displacement during installation of the sand compaction piles. For ground displacement associated with the sand pile formation by vibro-driving and vibro-removal, refer to the Examples of the Measurement of Horizontal Displacement during Construction[69) .

③ **Confirmation of the quality of sand compaction piles**

When *N*-values are used for the determination of liquefaction, the *N*-values of soil between the piles are the objects of quality control. In contrast, when verifying the performance of improved ground in consideration of the effects of composite ground, the *N*-values at the centers of the sand piles need to be included in the objects of quality control in addition to the *N*-values between the piles. The frequency of the standard penetration tests shall be determined in accordance with the complexity of the ground, the importance of the structures, and the number of sand piles and construction machines. According to past examples of the sand compaction pile method implemented for sandy ground, the required number of locations for the standard penetration test is one for every installation of 150 sand compaction piles in the case of small-scale construction with around 500 sand compaction piles. The frequency of the standard penetration test is likely to be reduced with an increase in the total number of sand compaction piles[81)].

5.2.6 Ground Investigations Related to the Improvement of Soils through Chemical Stabilization

(1) This section deals with ground investigations for the lightweight treated soil method, the pneumatic flow mixing method and the premix method. These methods are meant to improve the characteristics of soil as original material by mixing it with stabilization materials or binders such as cement and other necessary additives. As is the case with the deep mixing method, the principle of these soil improvement methods is classified as chemical stabilization. Thus, the factors affecting the improvement effects through stabilization as listed in **Table 5.2.4** are commonly applied to these methods.

(2) Investigations and tests for performance verification

The information necessary for the performance verification of improved ground can be obtained through the general ground investigations and tests introduced in **Reference (Part II), Chapter 1, 3 Standard Investigation Results on Investigations and Tests Related to Ground**. The characteristics of improved soil shall be appropriately set based on past examples in the performance verification stage.

① **Investigations of the characteristics of original material soil**

In the lightweight treated soil method, the pneumatic flow mixing method and the premix method, because the sources of original material soil are generally known in advance, investigations and tests are conducted for the soil in the designated areas and depths of the sources to determine whether or not the soil planned to be used is appropriate for the respective methods.

In the case of the lightweight treated soil method and the pneumatic flow mixing method, the investigation objects are soil particle density that affects the physical properties of the improved soil, water contents, grain size distribution, liquid and plastic limits, wet density, ignition loss, and, if necessary, pH values as well as organic contents in water in the soil. In the case of the pneumatic flow mixing method, the sand contents of material soil of 30% or less, and water contents of material soil of 90 to 110% (1.3 to 1.5 times the liquid limits) are considered to be optimal from the viewpoint of the pneumatic transportation of material soil and stabilization materials. The premix method has often been applied to material soil that has a natural water content of 15% or less, and fine-grained particles of 15% or less. For this method, the items to be tested are soil particle density, water contents, grain size distribution of the material soil, and the minimum and maximum density of sand. When the method is implemented for reducing earth pressure, consolidated and drained (CD) triaxial compression tests shall be conducted.

For the types of soil which have been used in each method, refer to the technical manuals of the respective methods[82), 83), 84)].

② **Laboratory mix test**

The lightweight treated soil method, the pneumatic flow mixing method and the premix method do not improve field soil, but fill construction sites such as reclamation sites with treated soil prepared in a manner that improves the characteristics of the source soil by mixing it with stabilization materials or binders and the necessary additives. The laboratory mix tests for these methods are conducted not only to evaluate the strength characteristics of treated soil but also to obtain information such as the fluidity and mixed states necessary for implementing the construction process. The specimens for the laboratory mix tests shall be prepared in accordance with the technical manuals of the respective methods[85), 86), 87)] .

For the lightweight treated soil (foam mixing) method, laboratory mix tests are conducted to determine mix proportions that satisfy the parameters set in the performance verification such as the density and strength characteristics of the treated soil as well as the appropriate fluidity during construction. The factors affecting the density of treated soil are defoaming performance when foam is injected and mixed with other materials, the reduction in bubble sizes due to water pressure when injected into construction sites and the shrinkage of foam through the process to develop strength. In addition, underwater separation resistance tests[88)] shall be conducted because treated soil needs to have adequate viscosity and fluidity while taking into consideration the risks of material separation or strength reduction during construction.

In the pneumatic flow mixing method, the fluidity of treated soil when pneumatically transported and injected is important for the facilitation of construction. Thus, the laboratory mix tests include tests to achieve mix proportions that satisfy the required strength and flow tests (**NEXCO Test Methods, 313-1999, Air Mortar and Air Milk Test Method**)[89)].

In the premix method, segregation preventive agents are used to cope with the risk that soil is separated from the binders when injected into water areas. Thus, laboratory mix tests are conducted to determine the types and additive amounts of binders and segregation preventive agents. Here, in order to prevent the quality of the treated soil from fluctuating at construction sites, the minimum additive amounts of segregation preventive agents are preferably 4.0% or more, or 50 kg/m^3 or more, in terms of the mass ratio with respect to the dry soil mass[90)] .

③ **Hexavalent chrome elution test**

In the lightweight treated soil method, the pneumatic flow mixing method and the premix-type stabilization method, it is required to confirm that the hexavalent chrome elution amounts under the mix conditions at construction sites are equal to or less than the environmental quality standards for soil (0.05 mg/L). The conditions and methods for the hexavalent chrome elution tests are as shown in **Part III, Chapter 2, 5.2.4 (3) ② Hexavalent Chrome Elution Test**.

(3) Strength and quality of treated soil

① **Construction management**

The lightweight treated soil method, the pneumatic flow mixing method and the premix method are implemented in an order of processes to transport material soil to construction sites; prepare the water contents of the material soil if necessary; mix the material soil with stabilization materials or binders, segregation preventive agents and lightweight materials; and inject the treated soil or fill construction sites with treated soil. Each of the above processes is subjected to the appropriate construction management. Because different methods have different characteristics, the details of construction management, such as management items and frequency, shall be determined with reference to the technical manuals of the respective methods[91), 92), 93)] .

② **Confirmation of quality after mixing**

In the lightweight treated soil method, the pneumatic flow mixing method and the premix method, the quality of the treated soil shall be confirmed in a manner that prepares specimens by sampling unstabilized soil before injection in molds and curing the samples for certain periods, then conducts unconfined tests with the specimens. It is necessary to pay close attention to keep the specimens from being damaged by impacts, drying or water absorption.

After implementing these methods, unconfined tests shall also be conducted with samples taken from construction sites. In addition, the strength of the treated soil may be confirmed through sounding. For the lightweight treated soil method, tests to confirm the density of the treated soil shall be conducted in addition to unconfined compression tests.

③ **Hexavalent chrome elution tests and tank leaching tests after installation**[78)]

When improving volcanic cohesive soil through the lightweight treated soil method or the pneumatic flow mixing method, it is necessary to conduct hexavalent chrome elution tests for the actual stabilized soil regardless of the results of these tests following the mix proportion tests. Furthermore, construction with a stabilization volume of 5,000 m^3 or more is subjected to tank leaching tests in addition to the hexavalent chrome elution tests. The conditions and methods of the hexavalent chrome elution and tank leaching tests shall be the same as those shown in **Part III, Chapter 2, 5.2.4(4) ② Hexavalent chrome elution tests and tank leaching tests after installation**.

5.3 Replacement methods

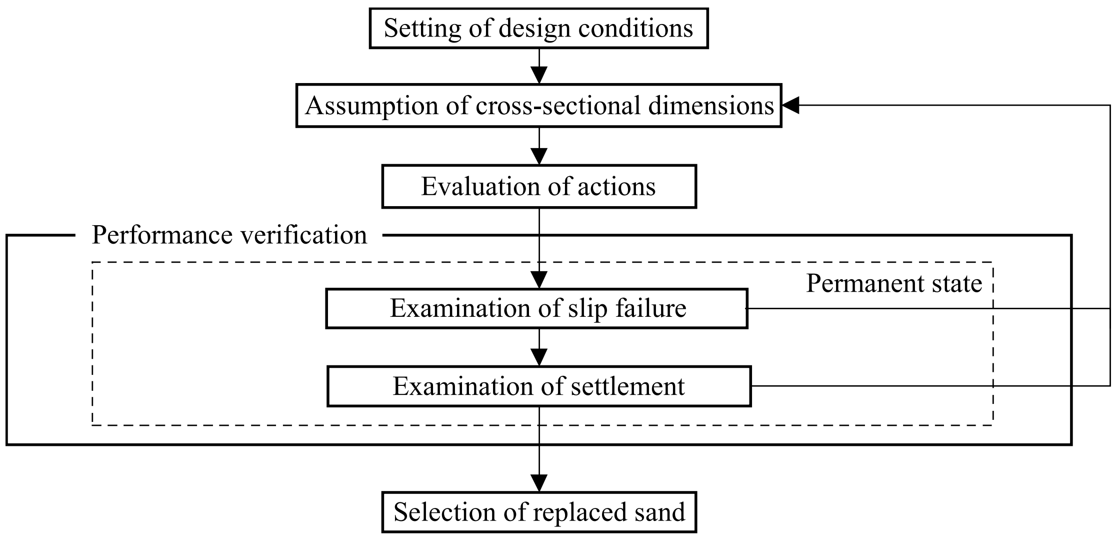
(1) The replacement methods can be separated into the replacement of ground by excavation (foundation replacement by excavation) and forced replacement. The replacement of ground by excavation is widely used in offshore work and implemented in a manner that removes soft soil by excavating it with pump or grab dredgers and fills the excavated areas with quality soil. The forced replacement is for replacing soft soil with quality soil by forcibly pushing the soft soil in lateral directions with the weight of fill or by using explosions[6)].

(2) The performance verification of the replacement methods shall give due consideration to ensuring stability through slip circle analyses and confirming the appropriateness of the settlement amounts as well as workability.

(3) The following explain the performance verification method of the replacement of ground by excavation (foundation replacement by excavation), which is widely used in offshore work.

① **Procedure of performance verification**

As shown in **Fig. 5.3.1**, performance verification for the replacement methods is preferably carried out in the order of setting the verification conditions, estimating the cross sections (the depth and width of replacement as well as the slope of excavation) subjected to performance verification, conducting examinations with respect to slip failures, examination with respect to settlement, and selecting the replacement sand. Although not shown in **Fig. 5.3.1**, there may be cases which require examinations of the possibility that the replacement sand is subjected to liquefaction and evaluations of the adverse effects of liquefaction on superstructures[59)].



*Note: Although not shown in the figure, additional examinations may be required for the evaluation of the effects of liquefaction.

Fig. 5.3.1 Example of the Performance Verification Procedure for Replacement Methods

② **Assumption of cross-sectional dimensions**

The performance verification of the replacement method is mainly carried out by incrementally changing the cross sections subjected to replacement until the predetermined stability and settlement amounts are satisfied. The assumption of cross-sectional dimensions can be made with reference to the following.

(a) Replacement depth

The target replacement depths can be set at those which allow all the soft layers to be replaced in the case of thin soft layers, or which allow the vertical stress at the depths to be smaller than the bearing capacity of the ground. (Refer to **Part III, Chapter 2, 3.3.4 Vertical Bearing Capacity of Deep Foundations** and **Part III, Chapter 2, 3.5.1 Underground Stresses**.) The replacement depths shall also be determined with due consideration to the capacity of the construction machines.

(b) Replacement width

According to examples of previous construction works, the relationship between the replacement widths and depths is as shown in **Fig. 5.3.2**.

(c) Slope of excavation

The slopes of excavation are determined in relation to the strength of the original ground and excavation depths (refer to **Part III, Chapter 2, 4 Stability of Slopes**), but are generally set at 1:1.5 to 1:3[94)].

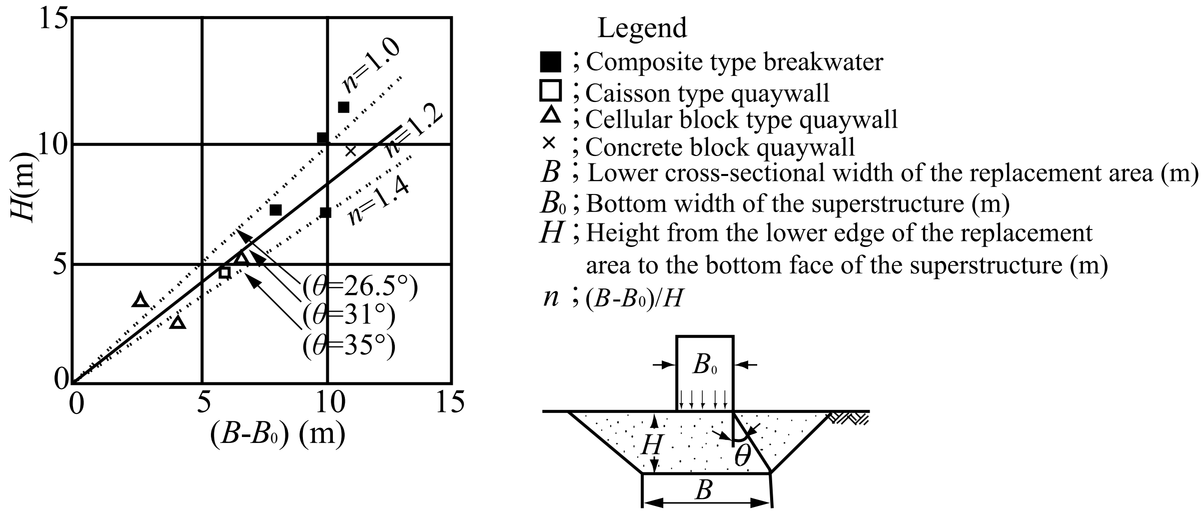


Fig. 5.3.2 Relationship between Replacement Widths and Depths

③ **Examination of slip circle failure**

The examination of slip failures through slip circle analyses can be carried out with reference to **Part III, Chapter 4 Stability of Slopes**. Partial factors can be set with reference to the relevant provisions in **Part III** as needed.

The shapes of the cross sections of replacement areas are generally inverted trapezoids. When calculating earth pressure on sheet piles or anchorage work to be constructed in the replacement areas, it is preferable to examine the stability of the sheet piles or anchorage work with respect to composite slip failures[95), 96)] . Furthermore, in the case of all layer replacements with inclined excavated bottom surfaces, it is preferable to examine the stability of the replacement areas with respect to composite slip circle failures including sliding failures on the excavated bottom surfaces.

④ **Examination of settlement**

When cohesive soil remains beneath the cross sections of replacement areas (at the bottom of partial excavations or the slopes of foundation excavations), the replacement areas are expected to be subjected to consolidation settlement of the remaining cohesive soil. Thus, it is preferable to examine the effects of settlement on superstructures while taking into consideration the consolidation yield stresses of cohesive layers and vertical loads acting on them.

⑤ **Selection of replacement sand**

Although there have been no clear selection criteria for replacement sand, it is preferable to select replacement sand that has good grain size distribution and a lower content of fine particles than silt. According to examples of previous construction, lower contents of fine particles than silt are generally limited to 15%, although there are cases of using pit sand with contents of fine particles smaller than silt by 20% or more. The internal friction angle of replacement sand are generally about 30 degrees, but it shall be noted that there may be cases of significantly lower internal friction angle depending on the grain sizes and grain size distribution of replacement sand, the methods and procedures of placing sand, retention time and surcharge.

⑥ ***N*-value of replacement sand**

The *N*-values of replacement sand are also susceptible to the grain sizes and grain size distribution of replacement sand, the methods and procedures of placing sand, retention time and surcharge. There are investigation reports showing that the *N*-values of replacement sand are around 10 in the case of instantaneous placement of a large quantity of sand with barges, around 5 in the case of placement with self-propelling grab hopper barges, and less than 5 in the case of placement with pump dredgers. In addition, there are cases of loose replacement sand which show an increase in *N*-values depending on surcharges and retention time (after the placement of replacement sand, the placement of rubble stones or the installation of caissons).

⑦ **Examination of liquefaction**

The basic method for determining whether or not replacement sand is subjected to liquefaction uses grain size distribution and *N*-values as determination criteria. In cases where the basic method is unworkable, the possibility of liquefaction shall be determined through cyclic triaxial compression tests[97)] (Refer to **Part II, Chapter 7 Liquefaction of Ground**.) When the cross sections of replacement areas and the property of replacement sand are specified in the examinations of liquefaction, the types of replacement sand are preferably selected accordingly. It is also preferable to compact placed sand when the placed sand does not have sufficient *N*-values.

5.4 Vertical Drain Method

5.4.1 Fundamentals of Performance Verification

(1) The vertical drain method shall ensure the following performance depending on the purpose of improvement.

① Increases in strength that satisfy the targeted amounts
② Residual settlement equal to or less than the amount allowed
③ Stability required for facilities

(2) Because the performance verification of the vertical drain method has a close relationship with the performance verification of facilities, as is the case with other soil improvement methods, actual performance verification cannot only be carried out for the vertical drain method. Generally, the performance verification of the vertical drain method is carried out by assuming the following factors. These factors need to be determined with due consideration to the stability of facilities, the earth pressure acting on facilities and the bearing capacity of the ground.

① Target increases in strength
② Allowable settlement of facilities
③ Area of work of the vertical drain method

(3) Ground conditions

The ground conditions related to the performance verification of the vertical drain method include the undrained strength of original ground, the increasing rates of strength, unit weight, coefficient of consolidation, coefficient of volume compressibility, preconsolidation pressure and the thicknesses of consolidation layers. When fill is used as consolidation loads, the shear strength and unit weight of the fill are also included in the ground conditions.

(4) Performance verification procedure

The vertical drain method is generally implemented for the purpose of accelerateing consolidation time of the preloading, surcharge or vacuum consolidation methods. Because there is no change in the strength of original ground immediately after the installation of vertical drains, all the consolidation loads necessary for achieving predetermined improvement effects cannot generally be applied to the improvement areas at once. Thus, the consolidation loads need to be applied in stages while confirming the increases in the strength of the ground. Furthermore, the heights of fill allowed to be constructed at each stage depend on the intensity of the consolidation loads and the degrees of consolidation in the respective stages, the arrangement of vertical drains and retention time. Thus, the performance verification shall be carried out in a manner that first approximately calculates the consolidation loads (the heights, widths and shapes of fill) necessary to achieve predetermined improvement effects (refer to **Part III, Chapter 2, 5.4.2(1) ① Heights and widths of fill necessary for ground improvement** and

② **Examination of ground stability with respect to fill**), then confirms the stability of the fill by assuming the degrees of consolidation at each stage (refer to **Part III, Chapter 2, 5.4.2(1) ③ Heights and widths of fill at the respective stages**). Finally, the arrangement of vertical drains which enables all the required fill to be constructed and consolidation of cohesive soil to be completed within the construction periods are examined (refer to **Part III, Chapter 2, 5.4.2(2) Performance verification of drains**).

An example of the procedure of the performance verification of vertical drains is shown in **Fig. 5.4.1**.

(5) Construction management

In the vertical drain method, it is of importance to manage the drain materials, installation depths, and arrangement and continuity of drains. In addition, construction management with a particular focus on the continuity between drains and sand mats or existing sand layers immediately below the improvement areas is important in the case of enhancing the drainage function by laying sand mats or utilizing existing sand layers. While constructing fill, it is necessary to confirm the increases in strength and progress of settlement as planned as well as the stability of fill by investigating the changes in pore water pressure in cohesive soil layers, increases in strength, settlement of the ground, cross-sectional shapes of fill and unit weight as needed.

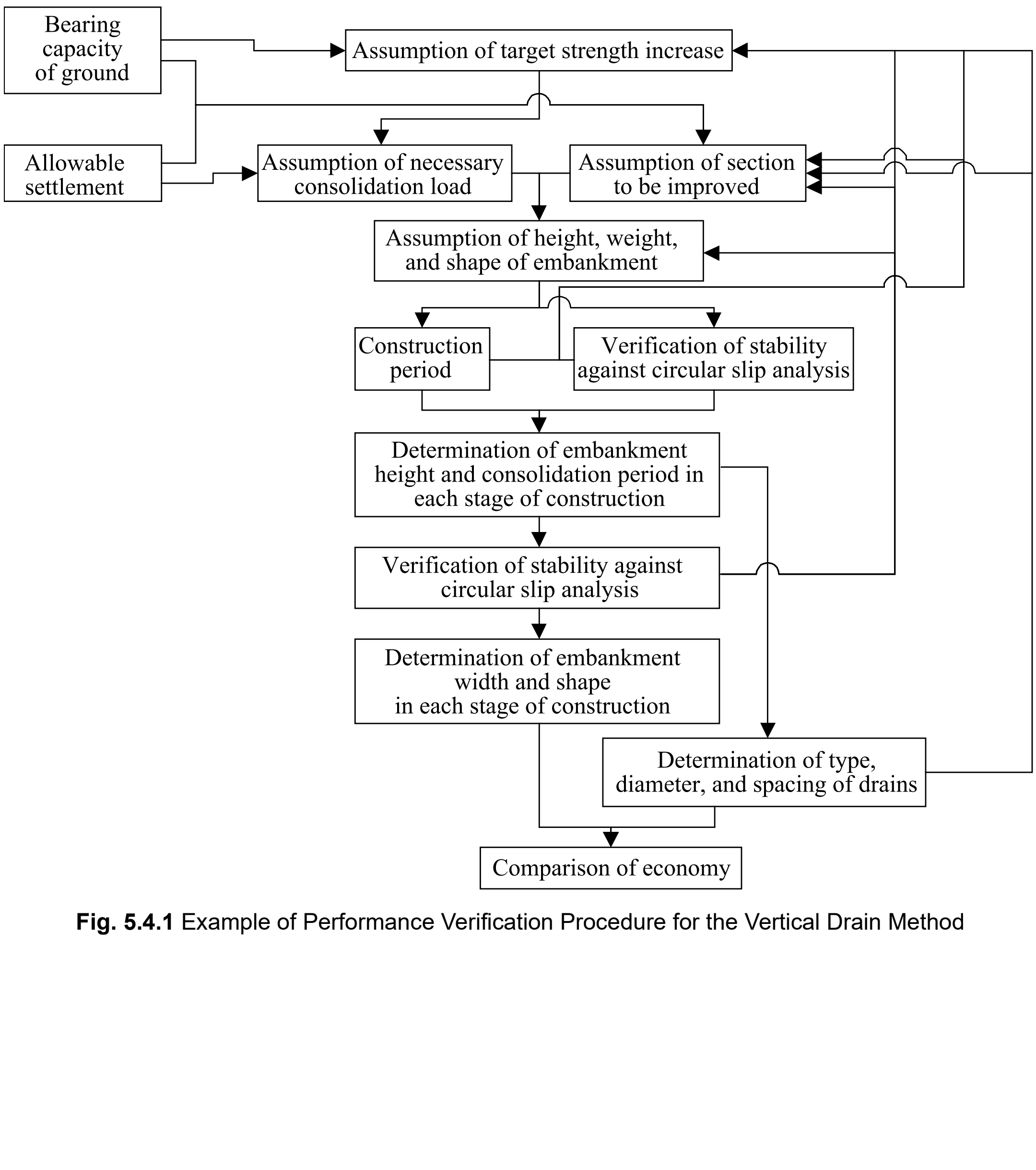


Fig. 5.4.1 Example of Performance Verification Procedure for the Vertical Drain Method

5.4.2 Performance Verification

(1) Determination of heights and widths of fill

① Heights and widths of fill necessary for ground improvement

(a) When fill is used as consolidation loading in the preloading and surcharge methods, the heights and widths of the fill shall be determined in consideration of the increases in strength necessary for stabilizing the fill during and after the staged construction of fill, the stability and allowable settlement of the facilities to be constructed, and the effects on the surroundings.

(b) The crown widths of fill shall be equal to or greater than the necessary ground improvement widths (refer to **Fig. 5.4.2**).

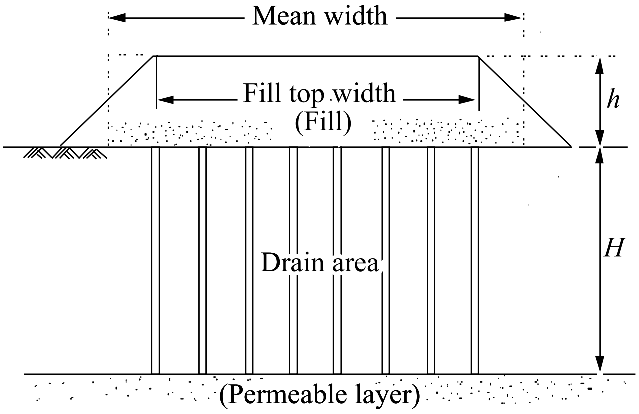


Fig. 5.4.2 Width of Fill for the Vertical Drain Method

(c) In the examination of the increases in strength (Δc) of original ground and residual settlement (Δs), the **equations (5.4.1)** and (**5.4.2**) can be used.

$$C_a \leq \Delta c$$
$$\Delta c = \Delta c/\Delta p\left(p_0'+\alpha\gamma' h - p_c'\right)U \qquad \textbf{(5.4.1)}$$

where

C_a : target increase in strength (kN/m^2)

h : height of the fill (m)

p_0' : initial pressure (vertical pressure before the commencement of construction) (kN/m^2)

p_c' : preconsolidation pressure (kN/m^2)

U : degree of consolidation

α : coefficient of stress distribution (ratio of distributed stress in the ground to a consolidation load (fill load))

γ' : unit weight of fill (wet unit weight for the portion above the sea surface and submerged unit weight for the portion below the sea surface) (kN/m^3)

Δc : increases in strength (kN/m^2)

$\Delta c/\Delta p$: increase rate of strength.

$$S_a \geq \Delta S$$
$$\Delta S = m_v\left(p_0' + \alpha\gamma' h - p_c'\right)H\left(1-U\right)$$
$$\Delta S = \frac{\Delta e}{1+e_0}H\left(1-U\right) \qquad \textbf{(5.4.2)}$$
$$\Delta S = \frac{Cc}{1+e_0}H\log_{10}\frac{p_0'+p'}{p_0'}\left(1-U\right)$$

where

C_c : compression coefficient

h : height of fill (m)

H : thickness of the consolidation layer (m)

m_v : coefficient of volume compressibility (m^2/kN)

p' : an increment of consolidation pressure (kN/m^2)

p_0' : initial pressure (vertical pressure before the commencement of construction) (kN/m^2)

p_c' : preconsolidation pressure (kN/m^2)

S_a : allowable residual settlement (m)

U : degree of consolidation

e_0 : initial void ratio of original ground

α : coefficient of stress distribution (ratio of distributed stress in the ground to a consolidation load (fill load))

γ' : unit weight of fill (wet unit weight for the portion above the sea surface and submerged unit weight for the portion below the sea surface) (kN/m^3)

Δe : a decrement of the void ratio of original ground

ΔS : residual settlement (m).

The coefficient of stress distribution can be estimated by using Boussinesq's solution (refer to **Part III, Chapter 2, 3.5.1 Ground Stress**). However, the Boston Code method may be used for estimating the coefficient of stress distribution in cases of wide improvement widths with crown widths of fill equal to or wider than the improvement widths. In such cases, the estimation is generally based on the average fill widths (as shown in **Fig. 5.4.2**) and the stresses at the intermediate depths of consolidation layers assuming uniform stress distribution in the depth direction. For the estimation of the coefficient of stress distribution with the Boston Code method, refer to **Part III, Chapter 2, 3.5.1 Ground Stress**. In cases where the unit weight of fill is not uniform, or where the fill widths or the degrees of consolidation largely fluctuate between staged loading, or where the consolidation object layers are not uniform, the **equations** (**5.4.1**) and (**5.4.2**) shall be applied to each loading stage or each layer.

The symbol U in the **equation** (**5.4.1**) is the degree of consolidation with respect to stress, and in the **equation** (**5.4.2**) is the degree of consolidation with respect to strain. Because the degrees with respect to stress are smaller than those with respect to strain, care shall be taken when predicting the increases in strength of cohesive soil from settlement.

② **Examination of ground stability with respect to fill**

(a) The ground stability with respect to the heights and widths of fill determined in **Part III, Chapter 2, 5.4.2(1) ① Heights and widths of fill necessary for ground improvement** shall be verified through slip circle analyses or other means. In cases where the stability cannot be verified, the fill in the final stage needs to be further divided into multiple stages and additional stability verification shall be conducted for the respective stages.

(b) Examination of the stability of fill with respect to slip failures

The examination of the stability of fill through slip circle analyses can be made with reference to **Part III, Chapter 2, 4 Slope Stability**. For the partial factors to be used in the examination, those for the slip circle analyses of the respective facilities can be used as reference. Here, it is necessary that the examination of stability is based on ground strength while taking into consideration the increases in strength calculated by the **equation (5.4.1)**.

(c) Approximate increases in strength

Because fill used as a consolidation load is generally constructed in several stages, every consolidation loading stage requires different degrees of consolidation (U) to be used in the **equations (5.4.1)** and (**5.4.2**). However, a degree of consolidation of approximately 80% is commonly applied to the calculation of the increases in strength at each stage.

③ **Heights and widths of fill at the respective construction stages**

(a) Fill is actually constructed in stages so as to achieve the final cross sections as determined in **Part III, Chapter 2, 5.4.2(1) ① Heights and widths of fill necessary for ground improvement**. The cross-sectional shapes of fill at each construction stage shall be determined from stage to stage while examining the stability of fill at every stage based on the increases in strength of consolidation object layers in the previous stages.

(b) Degrees of consolidation

Generally, setting large degrees of consolidation for the construction stages causes the drain intervals to be accelerateed or the retention periods of each stage to be extended, thereby reducing the economic efficiency of the ground improvement work. In contrast, setting small degrees of consolidation for the construction stages causes the additional height of the fill in the next stage to be lowered because the increases in strength in the current stage to support the additional height are reduced, thereby increasing the number of construction stages. In the actual construction, the degrees of consolidation range from 50 to 90% for each construction stage and are set at approximately 80% in most cases.

(c) Re-examination of cross sections

After determining the drain intervals, it is preferable to re-examine the cross sections of the fill at each stage based on an accurate calculation of the degrees of consolidation. Achieving a degree of consolidation of 80% at certain stages means that the consolidation object layers have undergone consolidation equivalent to 80% or more of that which was designed for the consolidation loads previously applied to the layers. Furthermore, in the case of high groundwater levels (with fill partially subjected to buoyancy), consolidation loads are gradually reduced along with consolidation settlement. Thus, a re-examination of the final cross sections while taking into consideration the above factors is necessary for accurately carrying out the performance verification.

(d) Points of caution when removing preloading

When utilizing fill used for preloading as part of the facilities, it is not necessary to consider the effect from removing the preloading. However, when the fill used for preloading is partially or fully removed after the completion of consolidation, it is necessary to carry out the performance verification of the stability of the facilities to be constructed with due consideration to the fact that cohesive ground absorbs water and swells over time, thereby reducing strength (refer to **Part II, Chapter 3, 2.3.3 Shear Characteristics**).

(2) Performance verification of drains

The performance verification of drains shall be carried out based on calculations while taking into consideration the drain intervals and diameters, the drainage conditions above and below the consolidation object layers, the permeability characteristics of the drain materials and sand mats, and the thicknesses of the sand mats.

① **Drains and sand mats**

(a) Drains and sand mats shall have predetermined drainage functions.

(b) Consolidation degrees and drain diameters

The rate of progression of consolidation is almost proportional to the drain diameters and inversely proportional to the squares of the drain intervals. Generally, the quantity of drain materials is smaller when arranging drains of small diameters at short intervals than when arranging drains with large diameters at wide intervals. However, when the diameters of the sand piles in the sand drain method are too small, there is a risk of the drains becoming clogged with particles of cohesive soil and the sand piles breaking halfway through due to the inability of the drains to bend along the ground deformation during the preloading and retention periods. According to examples of previous construction using the sand drain method, the diameters of the sand piles range from 40 to 50 cm and are about 40 cm in most cases. In the sand drain method that uses sand filled in small diameter bags, a lightweight installation machine installs four piles at once with sand filled in synthetic bags[11], which each have a diameter of about 12 cm. This method has often been used for improving soft ground on land. Another sand drain method which uses bags with a diameter of about 40 cm has been developed for the purpose of improving soft ground[9), 98)] .

(c) Materials for sand piles

The sand used for sand piles shall have an appropriate permeability and grain size distribution capable of preventing sand piles from being clogged with cohesive soil particles. According to the standard proposed by Terzaghi, it is considered to be necessary that a 15% grain diameter (D_{15}) of sand for the well point method is not less than 4 times of D_{15} of the consolidation soil, and not more than 4 times an 85% grain diameter (D_{85}) of the consolidation soil[99] . In contrast, the consolidation theory which considers drain pressure losses[100] by Aboshi and Yoshikuni requires coarser sand than the standard proposed by Terzaghi. The examples of grain size distribution curves of sand used in previous construction are shown in **Fig. 5.4.3**. Recently, there have been cases which use slightly finer sand than the examples in the figure.

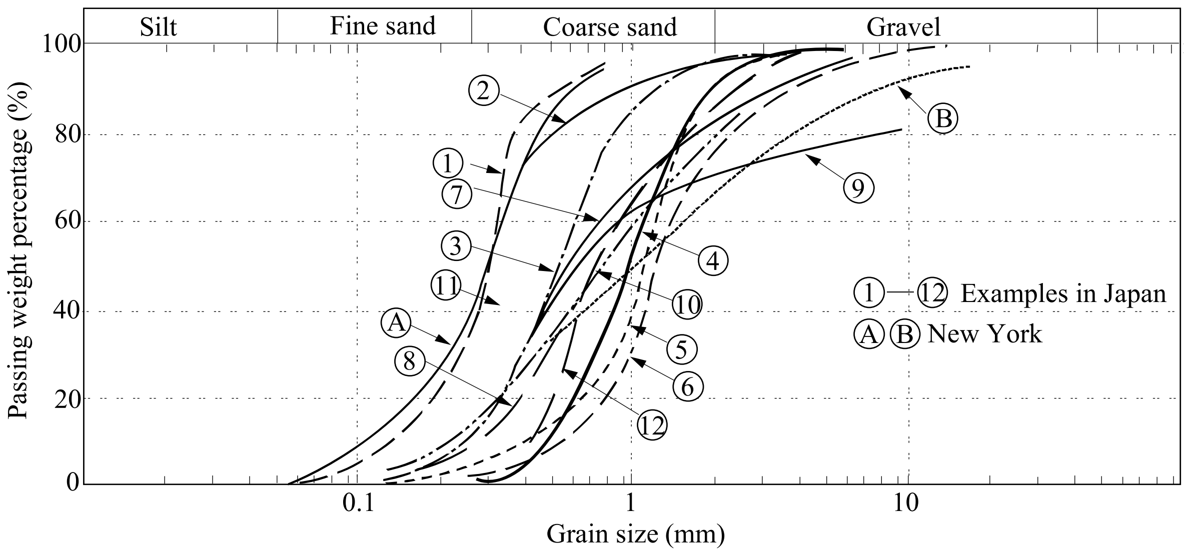


Fig. 5.4.3 Examples of Sand used in Sand Piles

(d) Materials for prefabricated drains

Several variations of vertical drains which use materials other than sand have been developed including band-shape drains having composite structures formed by bag-shaped filters made of nonwoven fabric or nonwoven fabric with synthetic resin cores, and unitary porous structures formed by specially-treated polyvinyl chloride. These variations of vertical drains are generally called prefabricated drains. The performance verification of these prefabricated drains shall be carried out by converting the cross sections of band-shape drains (with widths of about 10 cm and thicknesses of about 5 mm in general) into circles having equivalent outer perimeters. Practically, the performance verification of prefabricated drains has been carried out by assuming them as sand drains having a diameter of 5 cm[101]. It shall be noted that prefabricated drains having low discharge capacity will cause delays in the consolidation of soft layers at the tips of the drains (the lower sections of the consolidation layers)[102].

(e) Sand mats

Sand mats are used for discharging water drained through vertical drains out of the improvement areas. Quality sand with appropriate permeability is used for sand mats. The thickness of the sand mats is generally 1.0 to 1.5 m for offshore work and 0.5 to 1.0 m for on land work. Thick sand mats may cause difficulties in installing vertical drains. In contrast, thin sand mats may allow cohesive soil particles to easily degrade their permeability. Furthermore, the thicknesses of the sand mats may cause delays in consolidation as a result of increased pressure loss with the discharge capacity of sand mats reduced. In such cases, it is preferable to improve the permeability of the sand mats by installing drainage pipes. There is an approximate solution[103] which can be used as a reference for the relationship between permeability and time delays in consolidation. Recently, there has been a development of new methods which maintain horizontal drainage passages with extra portions of vertical drains interconnected in grid patterns, thereby eliminating the need for sand mats[104].

② **Drain intervals**

(a) Drain intervals shall be determined so as to enable prescribed improvement effects (degrees of consolidation) to be achieved within the given construction periods.

(b) General

The vertical drain method is normally used when the rates of consolidation through the preloading, surcharge or vacuum consolidation methods are lower than those determined in relation to the construction periods of the entire improvement work. **Fig. 5.4.4** shows the relationships when implementing the preloading, surcharge and vacuum consolidation methods without using drains, among the number of days required to achieve 80% consolidation of the cohesive soil layer drainage distances H, and the coefficient of consolidation c_v.

Note: In **Fig. 5.4.4**, the drainage distances H and the coefficient of consolidation c_v are expressed in units of (m) and (cm^2/min), respectively.

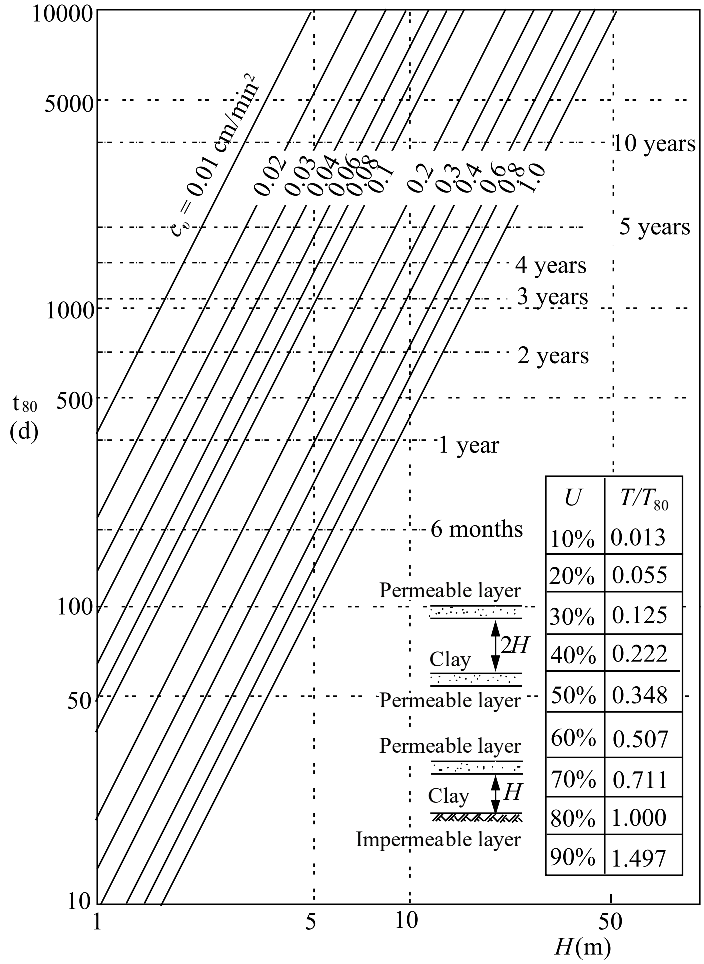


Fig. 5.4.4 The Number of Days Required to Achieve 80% Consolidation of Cohesive Layers

(c) Determination of drain intervals

The drain intervals can be obtained from **Fig. 5.4.5** and the **equation** (**5.4.3**) based on the Baron's or Bio's theories[105]. It has been pointed out that the effects of the smear of cohesive soil ground as a result of installing drains at too narrow intervals may cause delays in consolidation[106), 107), 108)].

$$D = \beta n D_w \tag{5.4.3}$$

where

D : drain interval (cm)

β : coefficient related to the arrangement of drains

β = 0.886 in the case of a square arrangement; β = 0.952 in the case of a regular triangle arrangement

n : $n = \dfrac{D_e}{D_w}$ (n can be obtained from **Fig. 5.4.5**)

D_e : equivalent diameter of a drain (cm)

D_w : diameter of a drain (cm)

T_h' : parameter similar to a time factor $T_h' = \dfrac{c_h t}{D_w{}^2}$

c_h : coefficient of consolidation related to water flow in the horizontal direction (cm^2/min)

t : consolidation time (min).

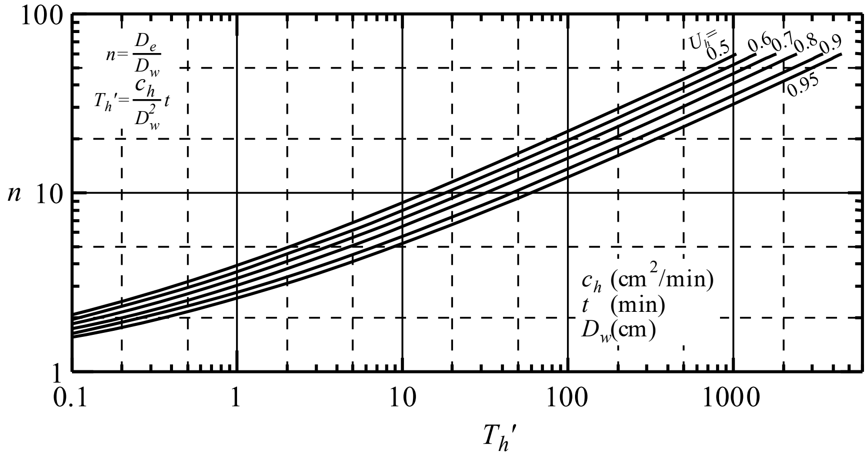


Fig. 5.4.5 Calculation Chart for the *N*-value

(d) Equivalent drain diameters

An equivalent drain diameter (D_e) is a diameter of a circle with an area equivalent to the equivalent area of the drain. Equivalent diameters have the following relationships with drain intervals (D)

In the case of a square arrangement: $D_e = 1.128D$ **(5.4.4)**

In the case of a regular triangle arrangement: $D_e = 1.050D$ **(5.4.5)**

(e) Water flow in a vertical direction

Although the vertical drain method expects the enhancement of consolidation with pore water drained in a horizontal direction, consolidation enhancement by water flowing in a vertical direction cannot be ignored when the thicknesses of the consolidation layers are relatively small with respect to the drain intervals.

For the performance verification of drain intervals, taking into consideration consolidation enhancement by water flowing in a vertical direction, refer to the **Reference 102)**.

(f) Coefficient of consolidation in a horizontal direction

No appropriate test method has been established for the coefficient of consolidation (c_h) for water flowing in a horizontal direction in cohesive soil layers. In general, the coefficient of consolidation in a horizontal direction are considered to be 5 to 10 times greater than those in a vertical direction, but some reports say that the coefficients are almost identical in both directions[109)]. When considering the effects of pressure loss in drains and from smear, it is not always acceptable to use the results of the consolidation tests which reproduce water flows in the horizontal direction. According to examples of previous construction work, it is practically allowed to substitute the coefficient of consolidation (c_v) in a vertical direction for those in a horizontal direction.

(g) Calculation of the degrees of consolidation

After determining the drain intervals, the relationships between the degrees of consolidation and elapsed time can be obtained from the **equations** (**5.4.6**) and (**5.4.7**) as well as **Fig. 5.4.6**.

$$T_h = \frac{c_h t}{D_e^{\ 2}} \tag{5.4.6}$$

$$n = \frac{D_e}{D_w} \tag{5.4.7}$$

where

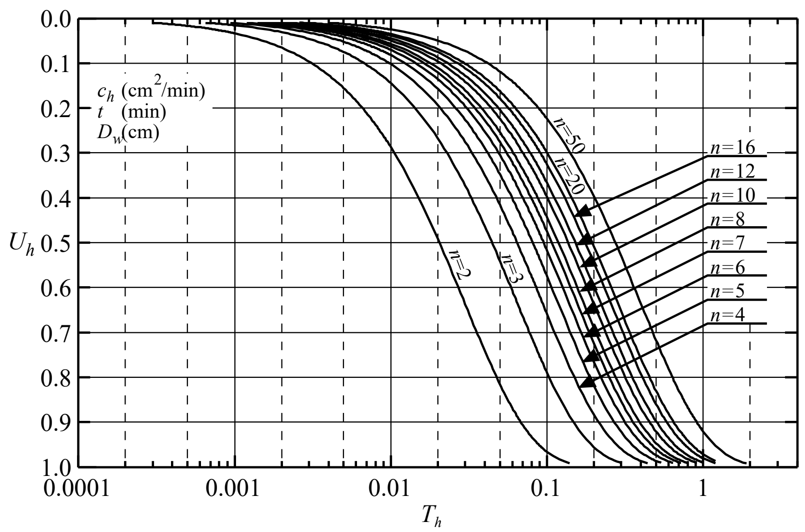
T_h : time factor for consolidation due to water flowing in a horizontal direction

c_h : coefficient of consolidation due to water flowing in a horizontal direction (cm^2/min)

t : elapsed time since the commencement of consolidation (min)

D_e : equivalent diameter of a drain (cm)

D_w : diameter of a drain.



Fig, 5.4.6 Calculation Chart for the Degrees of Consolidation

(h) Settlement behavior of ground surfaces (free and even settlement)

In the consolidation object layers of the vertical drain method, the progress of consolidation is faster in areas close to the drains than in other areas, as is the progress of settlement. However, there is an idea that settlement is averaged with the consolidation pressure in areas close to drains reduced due to the effect of an arch action preventing the consolidation settlement of the areas (even settlement). Conversely, there is another idea that the distribution of consolidation pressure is constant (free settlement)[102). **Figs. 5.4.5** and **5.4.6** are obtained based on the concept of even settlement. In these figures, the discrepancies in the averages of the degrees of consolidation between the two concepts become larger in the range of n < 10 and U_h < 60%.

(i) Consolidation due to incremental loads[102)]

Because fill needs to be constructed in stages, ground improvement work using fill as consolidation loads requires long construction periods. In these cases, consolidation loads are gradually increased over time and finally fixed. For the consolidation processes under incremental loads, there is a simplified calculation method[102)] which can be used as a reference.

(j) Cases of partial penetration drains

Consolidation requires a very long period of time in cases where thick cohesive soil layers or mechanical constraints on the construction machines allow drains to be installed only halfway through and not to the lower ends of the consolidation object layers. For the consolidation processes with partial penetration drains, there is a simplified calculation method[110)] which can be used as a reference.

(k) Cases of inhomogeneous cohesive soil layers

Inhomogeneous cohesive layers shall be analyzed layer by layer. The **References 111)** and **112)** can be used as references.

5.5 Deep Mixing Method

5.5.1 Fundamentals of Performance Verification

(1) Scope of application

① The deep mixing method dealt with in this section is a method which mechanically mixes field soil with cement.

② The majority of port and harbor facilities to which soil improvement using the deep mixing method has been applied are breakwaters, revetments (including partition dikes), and quaywalls having caissons or similar structures as their superstructures. The performance verification methods and partial factors presented in this section can be applied to the improved ground on which gravity-type breakwaters, revetments or quaywalls are constructed as the superstructures.

③ The deep mixing method is applied to port facilities when constructing high rigidity subsurface improved ground with construction machines in a manner that overlaps the stabilized soil in the form of piles. The shapes of the improved ground are determined depending on the properties of the ground subjected to improvement and the types as well as the scale of the superstructures. In general, the block-type and wall-type shown in **Fig. 5.5.1** are the typically used patterns. Accordingly, the types of improved ground discussed in this section are the block-type and wall-type improved groundwhich are typically used in the field of port development.

④ A wall-type improved ground consists of long and short walls as shown in **Fig. 5.5.1(b)**. The basic concept of the wall-type improvement is that the long walls are to transfer the actions of the superstructures to the bearing ground and short walls are to enhance the integrity of the improved ground.

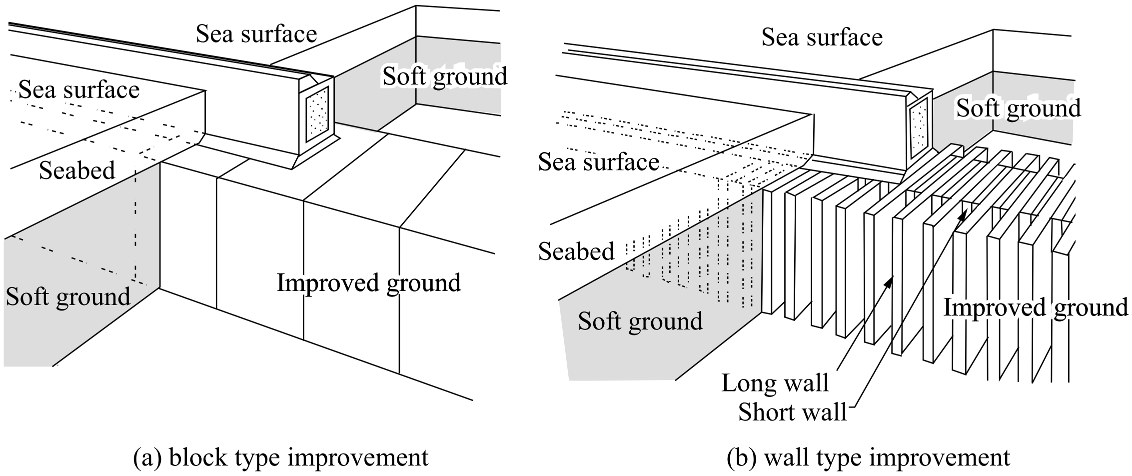


Fig. 5.5.1 Typical Improvement Patterns in the Deep Mixing Method

(2) Basic concept

① Definitions of the terms used in this section are as follows.

(a) Stabilized soil: Improved soil produced by the deep mixing method.

(b) Stabilized body: A kind of structure formed underground with stabilized soil.

(c) Improved ground: A portion of the ground where stabilized bodies and untreated soil are positioned close to each other (including untreated soil between long walls in the case of wall-type improved ground).

(d) Improved ground system: A portion of the ground above the bottom face of the improved ground and between the vertical planes passing through the front toe and hind toe of the improved ground.

(e) External stability: An examination of the stability of the process from the integration of improved ground and the superstructure (main construction) into a rigid body to the behavior of the rigid body until its final failure.

(f) Internal stability: The examination of internal failure of a stabilized body which is stable externally.

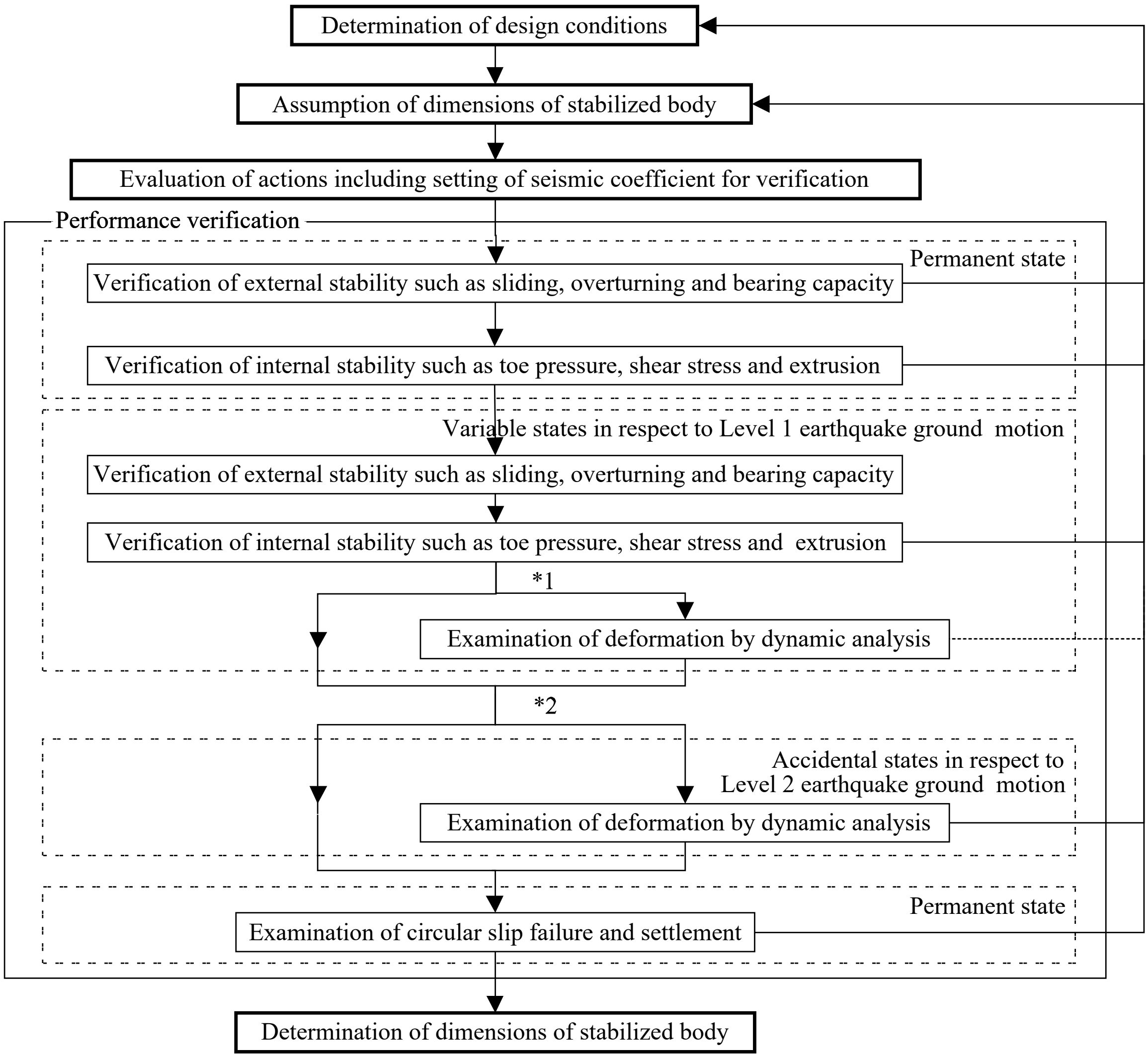
(g) Fixed type: A type of improvemed ground constructed by improving soft ground all the way through the bearing layer so that a stabilized body is seated on the bearing layer and transfers actions of the superstructure to the bearing layer.

(h) Floating type: A type of improved ground constructed by improving soft ground with a portion of soft ground remaining untreated below a stabilized body as if the stabilized body is floating on the soft ground without being seated on a bearing layer.

② Stabilized soil using the deep mixing method generally has extremely high strength and a high elastic modulus and extremely small strain at failure in comparison with the original ground soil[113)]. Accordingly, stabilized bodies made of stabilized soil are preferably regarded as structures subjected to examination of the stability of the structures as a whole (external stability), examination of the resistance of the structures themselves (internal stability), and, if particularly necessary, the examination of settlement, horizontal displacement and rotation of the stabilized bodies as rigid bodies.

③ The performance verification of the deep mixing method can be carried out with reference to the **Technical Manual for the Deep Mixing Method in Ports and Airports**[114)] .

④ An example of the performance verification procedure for the deep mixing method applied to gravity-type structures is shown in **Fig. 5.5.2.**



*1: Dynamic analyses can be used for the examination of deformation in respect to Level 1 seismic ground motions as needed. It is preferable to use dynamic analyses for examining deformation in cases where the widths of the improved ground are smaller than those of foundation mounds.

*2: Depending on the performance requirements of the main construction, the performance verification shall be carried out in respect to Level 2 seismic ground motions.

Fig. 5.5.2 Example of the Performance Verification Procedure for the Deep Mixing Method

⑤ The performance verification of the deep mixing method under a variable situation in respect to Level 1 seismic ground motions can be carried out in accordance with gravity-type quaywalls through the seismic coefficient method based on the equation of static equilibrium or nonlinear seismic response analyses while considering the dynamic interactions between the ground and structures as shown in **Part III, Chapter 5, 2.2.3 Performance Verification**. When performance verification through the seismic coefficient method results in the widths of improved ground becoming smaller than those of foundation mounds, it is necessary to examine the deformation of the improved ground and main construction through nonlinear seismic response analyses which consider the dynamic interactions between the ground and structures. Furthermore, it is necessary to carry out the performance verification of facilities in an accidental situation in respect to Level 2 seismic ground motions in accordance with the performance requirements of the facilities.

⑥ In the performance verification of the deep mixing method, it is necessary to consider the following items.

(a) Because there is no method for determining the dimensions of stabilized bodies in the deep mixing method in a single calculation, repeated calculations are required in the performance verification to obtain the most economical cross sections that satisfy the stability conditions.

(b) In the case of wall-type improved ground, it is necessary to determine the dimensions of both long walls and short walls. Because the long walls and short walls are constructed in a manner that overlaps columns made of stabilized soil, the cross-sectional shapes of the walls cannot be determined arbitrarily but shall be determined in consideration of the dimensions of the mixing machines to be used.

(c) In the case of wall-type improved ground, existing soil remains untreated between the long walls. Therefore, it is necessary that the internal stability be confirmed through examinations of not only the internal stresses in the stabilized bodies, but also examinations of extrusions of the untreated soil between the long walls.

(d) The limit values for deformation under variable and accidental situations can be set in accordance with the performance requirements of facilities with the deformation of the main construction supported by the ground stabilized by the deep mixing method as an index.

(e) In the verification of deformation due to Level 1 and Level 2 seismic ground motions, it is preferable to use the results of numerical analyses or shaking table tests which can appropriately assess the residual deformation of the improved ground system caused by the seismic ground motions.

(f) When applying numerical analyses to ground which has a risk of liquefaction, it is necessary to use a model which can appropriately assess the effects of liquefaction.

⑦ Recently, there has been technological improvement and development of new methods and technologies for the deep mixing method, and performance verification can be carried out not only with the methods introduced in this section, but also with methods based on the newly developed technologies. For example, when improving soft ground using fly ash as a binder, the performance verification can be carried out with reference to the **Technical Manual for FGC Deep Mixing Method**[115) .

5.5.2 Assumption of the Dimensions of Stabilized Bodies

(1) Mix proportion design method for stabilized soil

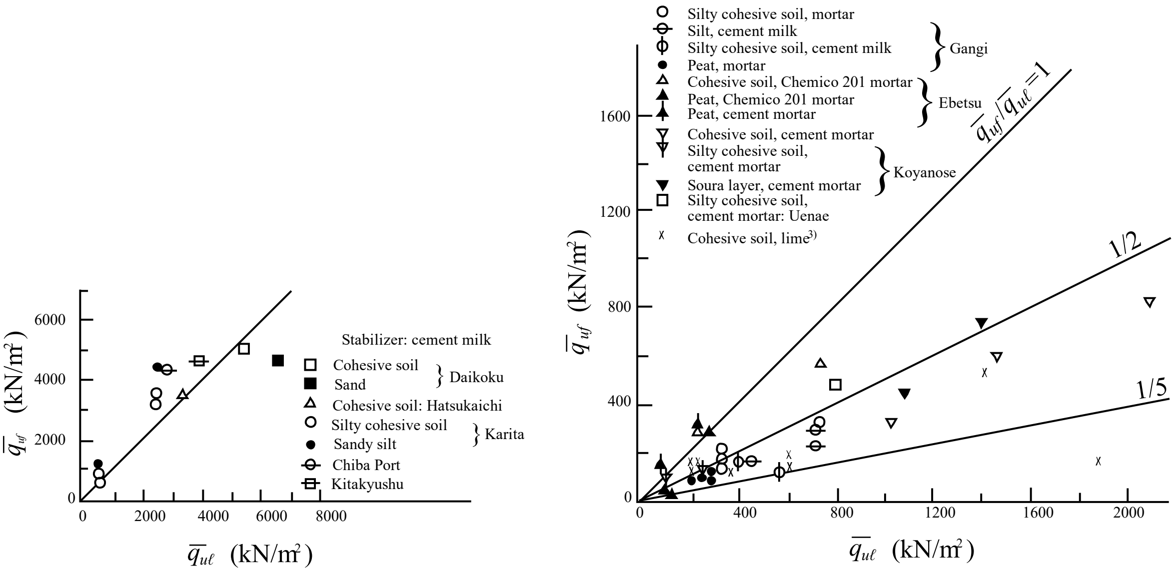
① The strength of stabilized soil depends largely on the physical characteristics and chemical properties of the object soil for improvement, the characteristics of the binders, and the mixing and curing conditions[116) . In addition, the specifications of the construction machines widely vary, and depending on the availability of the construction machines, there may be limitations in the water-cement ratios of the binders. Thus, in the mix proportion design of the stabilized soil, it is necessary to determine the strength through laboratory mix tests or field tests under conditions identical to actual use.

② In the mix proportion design, the strength of stabilized soil can be temporarily set based on examples of previous construction works.

③ Because laboratory mix tests are for obtaining the strength of object soil for improvement under standard test conditions, they cannot be used for directly obtaining the strength of object soil for improvement under actual conditions. When predicting field strength from laboratory mix test results, it is necessary to carefully study existing data on the relationship between the strength obtained through laboratory mix tests and field strength[117) . **Fig. 5.5.3** shows existing data on these relationships when using binders such as normal Portland cement and lime, which have large initial strength as binders.

④ For laboratory mix proportion design methods, refer to the Laboratory Testing Standards for Geomaterials by the Japanese Geotechnical Society[75) .

⑤ When implementing the deep mixing method using small barges susceptible to oscillation due to waves, or when using construction machines without performance records, it is preferable to conduct field tests before actual implementation. Particularly, when confirming the strength of overlapped sections, it is preferable to conduct field tests in a manner that actually constructs overlapped sections with columns made of multiple types of stabilized soil.



(a) Examples of offshore construction[117)]　　　(b) Examples of onshore construction[117)]

Fig. 5.5.3 Relationship between the Average Strength Obtained through Laboratory mix tests and the Average Field Strength

(2) Material strength of stabilized bodies

① When examining the internal stresses of stabilized bodies, it is necessary to set an appropriate material strength.

② The design compressive strength f_c of stabilized bodies can be obtained using the **equation** (**5.5.1**) with the standard design strength q_{uc} as a basis. In the equation, the subscript k represents a characteristic value.

$$f_{c_k} = \alpha\beta q_{uc_k} \tag{5.5.1}$$

where

f_c : design compressive strength of a stabilized body (kN/m²)

α : factor of an effective cross-sectional area

β : reliability index of an overlapped section

q_{uc} : standard design strength (kN/m²).

③ The design shear strength f_{sh} and design tensile strength f_t of stabilized bodies can be obtained from the **equation**s **(5.5.2)** and **(5.5.3)** using the design compressive strength f_c.

$$f_{sh_k} = \frac{1}{2} f_{c_k} \tag{5.5.2}$$

$$f_{t_k} = 0.15 f_{c_k} \leq 200\mathrm{kN/m^2} \tag{5.5.3}$$

where

f_{sh} : design shear strength of a stabilized body (kN/m²)

f_t : design tensile strength of a stabilized body (kN/m²).

④ The performance verification of stabilized bodies is based on the assumption that the stabilized bodies are made of materials with homogeneous strength. However, in actual construction work, because the stabilized bodies are formed by overlapping columns made of stabilized soil, there are cases where inhomogeneous stabilized soil is constructed underground in the form of residual untreated existing soil or overlapped sections with strength different from other sections depending on the mixing machines used and the methods of overlapping. In the **equation** (**5.5.1**), the factors (α and β) are used for treating stabilized soil as materials having homogeneous strength. The concepts behind setting these factors are explained below.

(a) Factor for effective cross-sectional area α

When constructing stabilized bodies using a machine with multiple mixing blades, the cross section of the stabilized bodies consists of multiple piles as shown in **Fig. 5.5.4**. In the block-type and wall-type improved ground, because the stabilized bodies are formed by overlapping columns made of stabilized soil as shown in **Fig. 5.5.4**, existing soil remains unimproved soil around the overlapped sections, thereby making the widths l of the joint areas shorter than the effective widths D of the stabilized bodies. The factor for effective cross-sectional area α is used for correcting the effect of the unimproved soil remaining around the overlapped sections and can be calculated by the **equation** (**5.5.4**).

The values of the factor for an effective cross-sectional area differ depending on the directions and types of the actions (such as compressive, tensile and shear force) which are subjected to the performance verification. For example, when considering the actions of the shear force in the vertical direction of the stabilized bodies or the stress in the direction perpendicular to the overlapped sections, examination of the actions on the joint areas with the narrowest width gives a result on the safe side. In offshore construction, the minimum overlapped width d needs to be set at 25 cm or more while taking into consideration construction accuracy and capabilities.

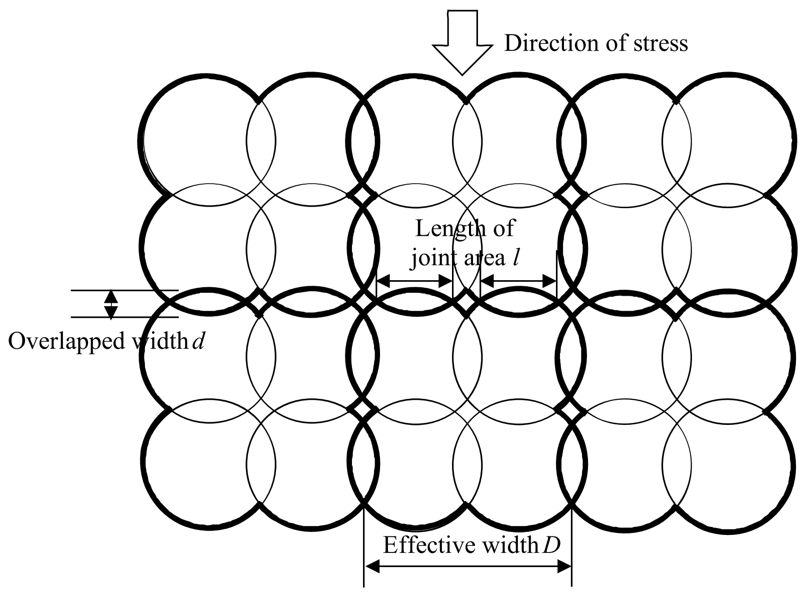


Fig. 5.5.4 Concept of the Factor for Effective Cross-Sectional Area α
(When Using Four-Axis Construction Machines)

$$\alpha = \frac{N \cdot l}{D} \tag{5.5.4}$$

where

N : number of axes on a joint area ($N = 2$ in the case of **Fig. 5.5.4**).

(b) Reliability index of overlap β

Overlapped sections are constructed with columns made of stabilized soil in a manner that partially overlaps new columns with those previously installed, which have already started to harden. Therefore, there may be cases where overlapped sections have lower strength than the other sections. The reliability index of overlap β is defined as a ratio of the strength of an overlapped section to that of other sections. Although the values of β differ depending on the elapsed time until new columns are joined to the existing ones, the mixing capacity of construction machines and the methods for discharging binders, the stabilized bodies can be designed with the value of β set at 1 according to the performance records.

⑤ When designing stabilized bodies using the characteristic values of standard design strength in the range of 1,500 to 2,500 kN/m^2 with an overlapped width of 30 cm or more, and a factor for effective cross-sectional area α of 0.8 or more, the value of $\alpha\beta$ can be set at 0.8 according to the performance records of the deep mixing method.

⑥ Relationship between the standard design strength and field strength and laboratory mix tests

The relationship between the average $\overline{q_{uf}}$ of the unconfined compressive strength q_{uf} of field stabilized soil and the characteristic value q_{uc_k} of the standard design strength can be expressed by the **equation** (**5.5.5**).

$$\overline{q_{uf}} = q_{uc_k} / (1 - KV/100) \tag{5.5.5}$$

where

K : coefficient showing a normal deviation (a magnification ratio with respect to a standard deviation σ) where the value is generally set at 1.0

V : coefficient of variation of unconfined compressive strength q_{uf} of field stabilized soil (although the coefficient varies depending on the construction machines and technologies, and is preferably set for individual cases, it can be set at V = 33 (%) according to examples of previous construction works).

Setting the coefficient K at 1.0 when the variation of the unconfined compressive strength q_{uf} of field stabilized soil corresponds to the normal distribution means that the characteristic value of q_{uc_k} of the standard design strength is set at a level corresponding to a defect occurrence ratio of 15.9%[118] (Refer to **Fig. 5.5.5**).

The relationship between the average $\overline{q_{uf}}$ of the unconfined compressive strength q_{uf} of field stabilized soil and the average of the unconfined compressive strength q_{ul} from laboratory mix tests can be expressed by the **equation** (**5.5.6**).

$$\overline{q_{uf}} = \lambda \overline{q_{ul}} \tag{5.5.6}$$

The value of λ is affected by numerous factors including the construction machines and conditions, types of object soil for improvement and binders, curing conditions and material ages. The target values of λ for offshore construction are 0.8 to 1 when using middle to large scale work barges (refer to **Fig. 5.5.3(a)**), and 0.5 to 1 when using small work barges; provided, however, that the value of λ may also be determined based on tests or performance records.

Fig. 5.5.5 show the schematic diagram of the relationships of standard design strength q_{uc_k} with the average value $\overline{q_{uf}}$ of the unconfined compressive strength of the specimens for laboratory mix tests and the average value $\overline{q_{uf}}$ of the unconfined compressive strength of field stabilized soil.

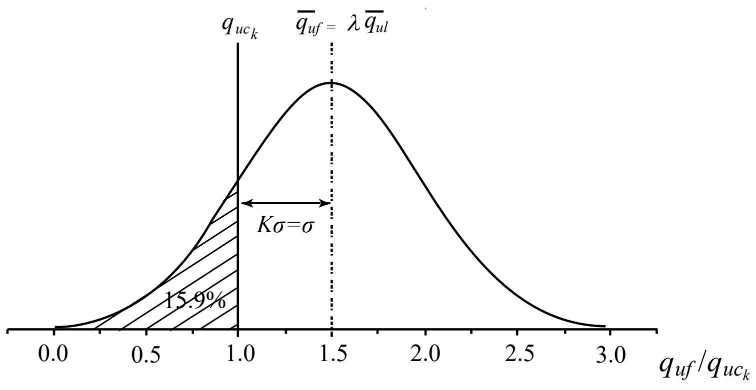


Fig. 5.5.5 Relationships of q_{uc_k} with $\overline{q_{uf}}$ and $\overline{q_{ul}}$ (Schematic Diagram)

5.5.3 Conditions of Actions on Stabilized Bodies[119)]

(1) **Fig. 5.5.6** shows a schematic diagram of loads acting on a stabilized body in the case of gravity-type revetments and quaywalls.

(2) Because untreated existing soil is remaining in wall-type improved ground, the loading conditions shall be set separately for untreated soil sections and stabilized soil sections for certain performance verification items.

(3) For examinations on the external stability of improved ground systems, P_a or P_p can be determined using the active and passive earth pressures specified in **Part II, Chapter 4, 2 Earth Pressure**. When examining internal stability, P_a may be considered as active earth pressure. However, it is preferable that P_p be set appropriately within a range from earth pressure at rest to passive earth pressure while considering the external stability of the improved ground systems.

(4) In cases where a certain amount of displacement of the improved ground is expected, it has been confirmed experimentally that the cohesion of untreated soil acts on the vertical planes of the active and passive sides of stabilized bodies. In the case of embankment and reclamation behind the improved ground, downward negative friction accompanied by consolidation settlement of the untreated soil acts on the vertical plane of the active side of stabilized bodies. Therefore, these types of cohesion shall be considered in the examination of a permanent situation[120)] . However, in the examination of actions associated with seismic ground motions, because the inertial force of stabilized bodies and the earth pressure during seismic ground motions are assumed on the safe side to act simultaneously on stabilized bodies for examinations, C_{ua} and C_{up} can be considered to act in downward and upward directions, respectively, in the examination of both external and internal stability. The values of C_{ua} and C_{up} in this case shall be obtained from the undrained shear strength of in-situ untreated soil.

(5) In the case of wall-type improved ground, it may be assumed that both P_a and P_p act uniformly on long walls and the untreated soil between long walls; provided, however, that when obtaining the subgrade reaction T at the bottom of the stabilized body, it is assumed that the loads acting on the stabilized bodies, such as the weight of the main construction, are concentrated on the long walls, and only the self-weight of the untreated soil acts on the untreated soil between long walls. The shear resistance force R shall be the sum of the shear resistance forces acting on the stabilized bodies and the bottom of the untreated soil.

(6) The deformation of main construction during the actions of seismic ground motions tends to be reduced by soil improvement through the deep mixing method. Therefore, when setting the seismic coefficient for the verification of the main construction and the improved ground systems, it is possible to set rational seismic coefficient for the verification on the basis of the appropriate evaluation of the deformation reduction effect.

When the deep mixing method is applied to ground improvement, the characteristic value of the seismic coefficient for the verification of main construction and the components of improved ground systems (such as superstructures, foundation mounds, backfill, reclamation and surcharge) can be calculated by multiplying the maximum values of corrected acceleration α_c with respect to the untreated ground by a reduction coefficient of 0.64, as shown in the **equation (5.5.7)** [121)] .

$$k_{h1_k} = 1.78 \left(\frac{D_a}{D_r} \right)^{-0.55} \frac{\alpha_c \times 0.64}{g} + 0.04 \qquad \textbf{(5.5.7)}$$

where

k_{h1k} : characteristic value of seismic coefficient for the verification of main construction and components of improved ground systems (superstructures, foundation mounds, backfill, reclamation and surcharge)

D_a : allowable deformation (cm)

D_r : standard deformation (= 10 cm)

α_c : maximum value of corrected acceleration (cm/s^2)

g : gravitational acceleration (= 980 cm/s^2).

This reduction coefficient was obtained based on the results of two-dimensional nonlinear effective stress analyses for untreated and improved ground. For details, refer to the **Reference 121)**. In calculating the maximum value of corrected acceleration α_c for untreated soil, refer to **Reference (Part III), Chapter 1, 1 Detailed Items for the Seismic Coefficient for Verification**.

The characteristic value of the seismic coefficient for verification of improved ground k_{h2k} can be calculated by multiplying the seismic coefficient for verification k_{h1k} obtained by using the **equation** (**5.5.7**) by the reduction coefficient 0.65 (k_{h2k} = 0.65 x k_{h1k}).

However, for the characteristic value of the seismic coefficient for verification k_{h3k} used for calculating earth pressure during earthquakes in improved ground systems, the maximum value of corrected acceleration shall not be multiplied by the reduction coefficient of 0.64 in the **equation (5.5.7).**

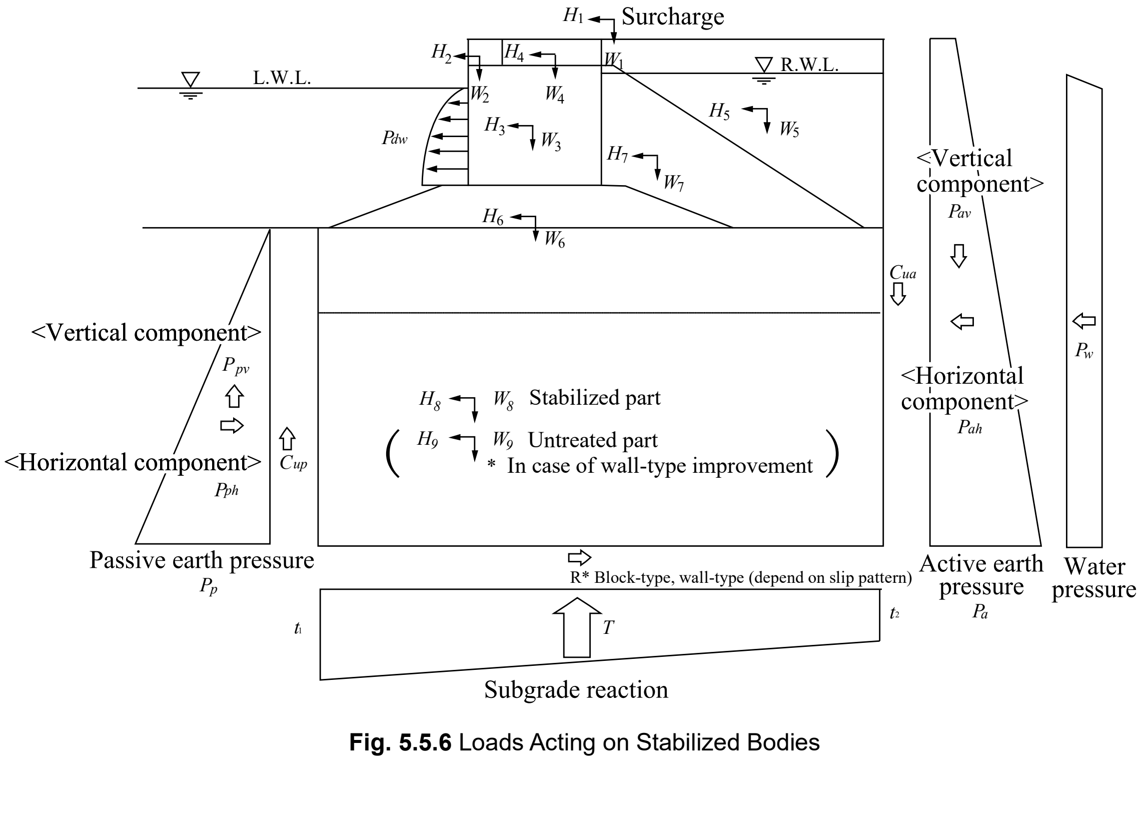


Fig. 5.5.6 Loads Acting on Stabilized Bodies

P_a : earth pressure per unit depth acting on the vertical plane of the active side (kN/m)

P_{ah} : horizontal component of the earth pressure per unit depth acting on the vertical plane of the active side (kN/m)

P_{av} : vertical component of the earth pressure per unit depth acting on the vertical plane of the active side (kN/m)

P_p : earth pressure per unit depth acting on the vertical plane of the passive side (kN/m)

P_{ph} : horizontal component of the earth pressure per unit depth acting on the vertical plane of the passive side (kN/m)

P_{pv} : vertical component of the earth pressure per unit depth acting on the vertical plane of the passive side (kN/m)

P_w : residual water pressure per unit depth (kN/m)

P_{dw} : dynamic water pressure per unit depth (kN/m)

W_1 to W_9 : weight per unit depth of each section (kN/m)

H_1 to H_9 : seismic inertia force per unit depth of each section (kN/m)

C_{ua} : cohesion on the vertical plane per unit depth acting on the vertical plane of the active side (kN/m)

C_{up} : cohesion on the vertical plane per unit depth acting on the vertical plane of the passive side (kN/m)

R : shear resistance per unit depth acting on the bottom of the improved ground (kN/m)

T : subgrade reaction per unit depth acting on the bottom of a stabilized body (kN/m)

t_1 and t_2 : intensity of the subgrade reaction at the toes of a stabilized body (kN/m^2).

In the performance verification of soil layers subjected to liquefaction during the actions of seismic ground motions, it is necessary to consider the dynamic water pressure on stabilized bodies during the actions of seismic ground motions. For the calculation of dynamic water pressure, refer to **Part II, Chapter 4, 3 Water Pressure**.

5.5.4 Performance Verification

(1) External stability of improved ground

In the performance verification of the external stability of improved ground, the following items shall be examined, assuming that the stabilized bodies and main construction behave integrally. It shall be noted that the following items provide descriptions for the cases of gravity-type revetments and quaywalls; however, the same descriptions can also be applied to breakwaters, provided that actions due to waves and other relevant factors are appropriately set.

① **Examination of sliding**

(a) Improved ground shall secure the required stability with respect to sliding failures.

(b) It is necessary to conduct performance verification of wall-type improved ground for two patterns, namely, the sliding failure pattern 1 (refer to **Fig. 5.5.7(a)**), which considers the frictional resistance of the bottom of the improved ground as a whole as resistance to slip failure, and the sliding failure pattern 2 (refer to **Fig. 5.5.7(b)**), which considers t the frictional resistance directly under long walls and the shearing resistance of the unimproved ground between walls, while considering the improved ground to be a structure in which the stabilized ground long walls fully demonstrate shear strength. For an examination of the stability with respect to sliding failures, the **equation** (**5.5.8**) can be used. In the equation, the subscripts k and d denote the characteristic value and design value, respectively.

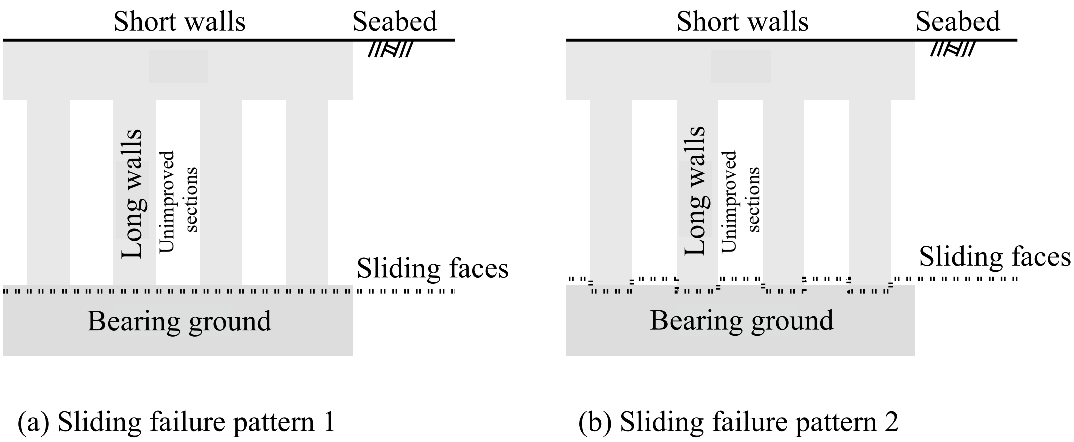


Fig 5.5.7 Sliding Failure Patterns

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k$$

$$R_k = P_{ph_k} + R_{1_k} + R_{2_k} \quad \text{(Sliding failure pattern 1)} \qquad R_k = P_{ph_k} + R_{1_k} + R_{3_k} \quad \text{(Sliding failure pattern 2)} \tag{5.5.8}$$

$$S_k = P_{ah_k} + P_{w_k} + P_{dw_k} + H_{i_k}$$

Where,

$$R_{1_k} = \mu_k \left(\sum W_{i_k} + W_{8_k} + P_{av_k} - P_{pv_k} + C_{ua_k} - C_{up_k} \right)$$

$$R_{2_k} = \mu_k W_{9_k}$$

$$R_{3_k} = C_{u_k} B R_s$$

$$P_{w_k} = \rho_w g (RWL_k - WL_k) \left\{ \frac{1}{2} (RWL_k - WL_k) + h_L + WL_k \right\}$$

$$P_{dw_k} = \frac{7}{12} k_{h3} \rho_w g (h_1 + WL_k)^2$$

$$H_{i_k} = k_{h1} \sum W_{ni_k} + k_{h_2} (W_{n8_k} + W_{n9_k})$$

R_k : characteristic value related to a resistance term (kN/m)

S_k : characteristic value related to a load term (kN/m)

R_1 : frictional resistance of bearing ground per unit depth acting on the bottom of a stabilized body (kN/m)

R_2 : frictional resistance of bearing ground per unit depth acting on the bottom of an untreated soil section (kN/m)

R_3 : shearing resistance per unit depth acting on the bottom of an untreated soil section (kN/m)

P_w : residual water pressure per unit depth (kN/m)

P_{dw} : dynamic water pressure during an earthquake per unit depth (kN/m)

H_i : inertia force per unit depth acting on each section (kN/m)

W_i : weight per unit depth of materials (such as surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) on improved ground constituting an improved ground system (kN/m)

W_8 : weight of a stabilized body per unit depth (kN/m)

W_9 : weight of untreated soil between long walls per unit depth (kN/m)

B : improvement width of a stabilized body (m)

R_s : ratio of short walls to long walls in a stabilized body

μ : static friction coefficient

C_u : shear strength on the bottom of untreated soil (kN/m^2)

P_{ah} : horizontal component of the earth pressure per unit depth acting on the vertical plane of the active side (kN/m)

P_{av} : vertical component of the earth pressure per unit depth acting on the vertical plane of the active side (kN/m)

P_{ph} : horizontal component of the earth pressure per unit depth acting on the vertical plane of the passive side (kN/m)

P_{pv} : vertical component of the earth pressure per unit depth acting on the vertical plane of the passive side (kN/m)

C_{ua} : cohesion on a vertical plane per unit depth acting on the vertical plane of the active side (kN/m)

C_{up} : cohesion on a vertical plane per unit depth acting on the vertical plane of the passive side (kN/m)

$\rho_w g$: unit weight of seawater (kN/m^3)

RWL : residual water level (m)

WL : water level on the offshore side (m)

h_L : water depth at the bottom of a stabilized body (m)

h_1 : water depth on the offshore side of a structure (m)

k_{h1} : seismic coefficient for verification when calculating the inertia force acting on materials (surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) over improved ground constituting an improved ground system

k_{h2} : seismic coefficient for verification when calculating the inertia force acting on improved ground

k_{h3} : seismic coefficient for verification when calculating the earth pressure and dynamic water pressure acting on an improved ground system

W_{ni} : weight per unit depth of materials (surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) over improved ground constituting an improved ground system (saturated unit weight when submerged) (kN/m)

W_{n8} : weight per unit depth of a stabilized body (saturated unit weight when submerged) (kN/m)

W_{n9} : weight per unit depth of untreated soil between long walls (saturated unit weight when submerged) (kN/m)

γ_R : partial factor multiplied by the resistance term

γ_S : partial factor multiplied by the load term

m : adjustment factor.

(c) In a broad sense, the sliding failures of wall-type improved ground may include the shear failures of long walls in cases where the strength of the improved ground is low; however, such shear failures are excluded from examinations in this section because improved ground has rarely been developed with low strength and there have been very few cases of such shear failures.

(d) The partial and adjustment factors in the **equation** (**5.5.8**) can be selected from **Table 5.5.1**. If a corresponding column in the table has the symbol "**—**," the value in parentheses in the column can be used for performance verification for convenience.

(e) The partial factors listed in **Table 5.5.1** are set with reference to the safety levels in the past standards based on the use of the characteristic values of the physical properties of ground specified in **Part II, Chapter 3, 2.1 Methods for Estimating the Physical Properties of Ground**.

Table 5.5.1 Partial Factors to be used in the Examination of Sliding Failures

Mode of failure		Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
External stability of the stabilized body (Sliding failure: permanent state)	Sliding failure pattern 1 (**Fig. 5.5.7(a)**)	0.90	1.09	– (1.00)
	Sliding failure pattern 2 (**Fig. 5.5.7(b)**)	0.91	1.10	– (1.00)
External stability of the stabilized body (Sliding failure: variable state of Level 1 seismic ground motions)		– (1.00)	– (1.00)	1.00

② **Examination of overturning**

(a) Improved ground shall secure the required stability with respect to overturning. The **equation** (**5.5.9**) can be used for the examination of stability with respect to the overturning of wall-type improved ground. In the equation, the subscripts k and d denote the characteristic value and design value, respectively.

$$\left.\begin{aligned}
&m\cdot\frac{S_d}{R_d}\leq 1.0 \qquad R_d=\gamma_R R_k \qquad S_d=\gamma_S S_k\\
&R_k=P_{ph_k}y_p+\sum(W_{i_k}x_i)+W_{8_k}x_8+W_{9_k}x_9+P_{av_k}x_{av}+C_{ua_k}x_{C_{ua}}\\
&S_k=P_{ah_k}y_a+P_{w_k}y_w \qquad \text{(Permanent situation)}\\
&S_k=P_{ah_k}y_a+P_{w_k}y_w+P_{dw_k}y_{dw}+\sum H_{i_k}y_i \qquad \text{(Variable situation in respect to Level 1 earthquake ground motions)}
\end{aligned}\right\} \quad \textbf{(5.5.9)}$$

Where,

$$P_{w_k}=\rho_w g(RWL_k-WL_k)\left\{\frac{1}{2}(RWL_k-WL_k)+h_L+WL_k\right\}$$

$$P_{dw_k}=\frac{7}{12}k_{h3}\rho_w g(h_1+WL_k)^2$$

$$\sum H_{i_k}=k_{h1}\sum W_{ni_k}+k_{h_2}(W_{n8_k}+W_{n9_k})$$

R_k : characteristic value related to a resistance term (kN/m)

S_k : characteristic value related to a load term (kN/m)

W_i : weight per unit depth of materials (such as surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) on improved ground constituting an improved ground system (kN/m)

W_{ni} : weight per unit depth of materials (surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) over improved ground constituting an improved ground system (saturated unit weight when submerged) (kN/m)

W_8 : weight of a stabilized body per unit depth (kN/m)

W_9 : weight of untreated soil between long walls per unit depth (kN/m)

W_{n8} : weight per unit depth of a stabilized body (saturated unit weight when submerged) (kN/m)

W_{n9} : weight per unit depth of untreated soil between long walls (saturated unit weight when submerged) (kN/m)

H_i : inertia force per unit depth acting on each section of an improved ground system (kN/m)

P_{ph} : horizontal component of the earth pressure per unit depth acting on the vertical plane of the passive side (kN/m)

P_{av} : vertical component of the earth pressure per unit depth acting on the vertical plane of the active side (kN/m)

P_{ah} : horizontal component of the earth pressure per unit depth acting on the vertical plane of the active side (kN/m)

C_{ua} : cohesion on a vertical plane per unit depth acting on the vertical plane of the active side (kN/m)

P_w : residual water pressure per unit depth acting on the vertical plane of the active side (kN/m)

P_{dw} : dynamic water pressure during an earthquake per unit depth acting on the vertical plane of the active side (kN/m)

RWL : residual water level (m)

WL : water level on the offshore side (m)

k_{h1} : seismic coefficient for verification when calculating the inertia force acting on the materials (surcharge, superstructures, main construction, foundation mounds, backfill and reclamation) over improved ground constituting an improved ground system

k_{h2} : seismic coefficient for verification when calculating the inertia force acting on improved ground

k_{h3} : seismic coefficient for verification when calculating the earth pressure and dynamic water pressure acting on an improved ground system

x_i, x_{av}, x_{Cua} : distances from the action lines of the vertical force acting on improved ground to the front toe of a stabilized body (m)

y_i, y_p, y_a, y_w, y_{dw} : heights from the action lines of the horizontal force acting on improved ground to bottom of a stabilized body (m)

γ_R : partial factor multiplied by a resistance term

γ_S : partial factor multiplied by a load term

m : adjustment factor.

(b) The partial factors to be used in the examination of the overturning of improved ground can be selected from **Table 5.5.2**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience.

(c) The partial factors listed in **Table 5.5.2** are set with reference to the safety levels in the past standards based on the use of the characteristic values of the physical properties of ground specified in **Part II, Chapter 3, 2.1 Methods for Estimating the Physical Properties of Ground**.

Table 5.5.2 Partial Factors to be used in the Examination of Overturning

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
External stability of the stabilized body (Overturning failure: permanent state)	0.97	1.18	– (1.00)
External stability of the stabilized body (Overturning failure: variable state of Level 1 seismic ground motions)	– (1.00)	– (1.00)	1.10

③ **Examination of bearing capacity**

(a) Improved ground shall secure the required stability with respect to the failure of the bearing capacity of the original ground under the bottom of the improved ground. In the examination of the bearing capacity of block-type improved ground, refer to **Part III, Chapter 2, 3.2 Shallow Spread Foundations**.

(b) In the case of wall-type improved ground with sandy ground as the bearing ground, the bearing capacity can be verified by the **equation** (**5.5.10**) using toe pressure t_1 and t_2 while taking into consideration the effect of the mutual interference of long walls[122)]. In the equation, the subscripts k and d denote the characteristic value and design value, respectively.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k$$

$$R_k = q_{ap_k} + q_{ar1_k} \ \left(\text{In the case of } \frac{1}{\eta} \geq 3\right), \ R_k = q_{ap_k} + q_{ar_k} \ \left(\text{In the case of } 1 \leq \frac{1}{\eta} < 3\right), \ S_k = t_{1_k}, t_{2_k} \qquad \textbf{(5.5.10)}$$

where

$$q_{ap_k} = \frac{1}{m_B} p_{0_k} \left(N_{q_k} - 1\right) + p_0$$

$$q_{ar1_k} = \frac{1}{m_B} w_k \frac{L_\ell}{2} N_{\gamma_k}$$

$$q_{ar2_k} = \frac{1}{m_B} w_k \frac{B}{2} N_{\gamma_k}$$

$$q_{ar_k} = q_{ar1_k} + \frac{1}{2}\left(q_{ar2_k} - q_{ar1_k}\right)\left(3 - \frac{1}{\eta}\right)$$

$$\eta = \frac{L_\ell}{L_\ell + L_s}$$

γ_R : partial factor multiplied by a resistance term

γ_S : partial factor multiplied by a load term

m : adjustment factor

m_B : adjustment factor with respect to the bearing capacity

N_q, N_r : bearing capacity coefficient (refer to **Part III, Chapter 2, 3.2.2 Bearing Capacity of Foundations on Sandy Ground**)

p_0 : effective overburden pressure to a sandy bearing layer (kN/m^2)

w : unit weight of bearing ground (submerged unit weight when submerged) (kN/m^3)

L_l : length of a long wall in the normal direction (m) (refer to **Fig. 5.5.10**)

L_s : length of a short wall in the normal direction (m) (refer to **Fig. 5.5.10**)

B : improvement width (m) (refer to **Fig. 5.5.10**).

(c) The partial factors to be used in the examination of the bearing capacity can be selected from **Table 5.5.3**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience.

(d) The partial factors listed in **Table 5.5.3** are set with reference to the safety levels in the past standards.

Table 5.5.3 Partial Factors to be used in the Examination of Bearing Capacity

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m	Adjustment factor with respect to bearing capacity m_B
External stability of the stabilized body (Failure of bearing capacity: permanent state)	0.49	1.15	– (1.00)	– (1.00)
External stability of the stabilized body (Failure of bearing capacity: variable state of Level 1 seismic ground motions)	– (1.00)	– (1.00)	– (1.00)	1.50

(2) Examination of internal stability

① For the characteristic values of the material strength of stabilized bodies, refer to **Part III, Chapter 2, 5.5.2 Assumption of the Dimensions of Stabilized Bodies**.

② The stresses generated in stabilized bodies can be obtained by assuming that the stabilized bodies are elastic bodies under the conditions specified in **Part III, Chapter 2, 5.5.3 Conditions of Actions on Stabilized Bodies**.

③ The internal stability of the block-type and wall-type improved grounds can be examined by the method presented below; provided, however, that the examination by FEM analysis is preferable in cases where the shapes of the stabilized bodies are complex, or the depths of the stabilized bodies are large in comparison with their widths.

④ **Examination of toe pressure**

(a) The verification of the internal stability with respect to the toe pressure at the bottom of the stabilized bodies can be performed using the **equation** (**5.5.11**) while considering the effect of the confining pressure acting on the improved ground. In the equation, the subscripts k and d denote the characteristic value and design value, respectively.

$$\left.\begin{array}{l} m \cdot \dfrac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\ \\ R_k = f_{ck}、\quad S_k = t_{1,2_k} - K\sum(w_{i_k} h_i) \end{array}\right\} \qquad \textbf{(5.5.11)}$$

where

R_k : characteristic value related to a resistance term (kN/m)

S_k : characteristic value related to a load term (kN/m)

f_c : design compressive strength (kN/m^2)

$t_{1,2}$: toe pressure (kN/m^2)

K : coefficient of earth pressure

w_i : unit weight of untreated soil (submerged unit weight when submerged) (kN/m^3)

h_i : thickness of untreated soil layers (m)

γ_R : partial factor multiplied by a resistance term

γ_S : partial factor multiplied by a load term

m : adjustment factor.

However, it is necessary to determine the value of the confining pressure $K\sum(w_{i_k} h_i)$ of untreated soil acting on the bottom edges of the stabilized bodies while taking into consideration the improvement patterns and the external stability of improved ground.

(b) The partial factors to be used in the examination of toe pressure can be selected from **Table 5.5.4**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience.

Table 5.5.4 Partial Factors to be used in the Examination of Toe Pressure

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Internal stability of the stabilized body (Toe pressure: permanent state)	0.72	1.33	– (1.00)
Internal stability of the stabilized body (Toe pressure: variable state of Level 1 seismic ground motions)	– (1.00)	– (1.00)	1.50

⑤ **Examination of shear stresses along vertical planes immediately beneath the face lines of the superstructure**

The examination of internal stability with respect to shearing stresses along the vertical planes immediately beneath the face lines of the superstructure (**Fig. 5.5.8**) can be performed for the long wall and short wall sections using the **equations** (**5.5.12**) and (**5.5.13**), respectively. In these equations, the subscripts k and d denote the characteristic value and design value, respectively.

(a) Long walls

$$\left.\begin{array}{l} m \cdot \dfrac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\ \\ R_k = \dfrac{1}{2}\alpha\beta q_{uc_k} \text{、} \quad S_k = \left(T_{\ell_k} - W_{\ell_k}\right)/A \end{array}\right\} \qquad \textbf{(5.5.12)}$$

where

R_k : characteristic value related to a resistance term (kN/m)

S_k : characteristic value related to a load term (kN/m)

α : factor for an effective cross-sectional area

β : reliability index of an overlap section between improvement piles

T_ℓ : subgrade reaction acting on an area from the front toe of improved ground to B_ℓ (kN)

q_{uc} : standard design strength (kN/m^2)

W_ℓ : submerged weight of a stabilized body from the front toe of improved ground to B_ℓ (kN)

A : cross-sectional area of a stabilized body; in the case of long walls $A = D_\ell L_\ell + D_s L_s$ (m^2) (see **Fig. 5.5.8**)

D_ℓ, D_s : vertical length (improvement depth) of a long wall and the vertical length of a short wall (m)

L_ℓ, L_s : lengths of long and short walls in a normal direction (m)

γ_R : partial factor multiplied by a resistance term

γ_S : partial factor multiplied by a load term

m : adjustment factor.

When a foundation mound exists between a stabilized body and the superstructure, the examination of shear stress can be performed with respect to a plane considering the dispersion of loads inside the foundation mound from the position of the face line of the superstructure (refer to **Fig. 5.5.8** where θ is a load dispersion angle inside the foundation mound).

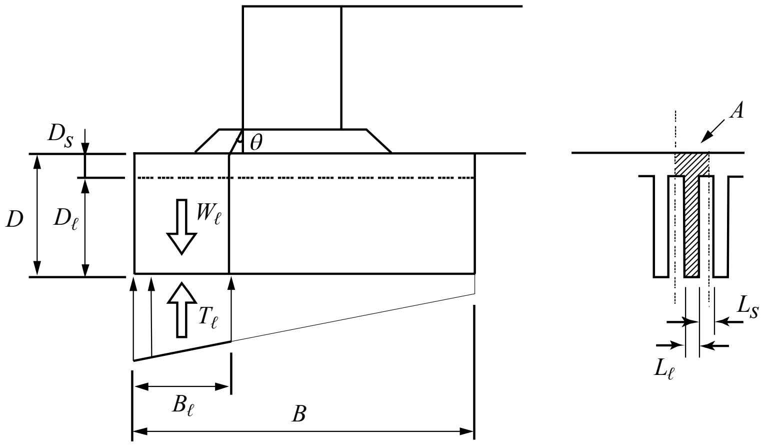


Fig. 5.5.8 Schematic Calculation Diagram of Vertical Shear Stress (Long Wall)

The partial factors to be used in the examination of the vertical shear failures of long wall sections can be selected from **Table 5.5.5**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience.

Table 5.5.5 Partial Factors to be used in the Examination of Vertical Shear Failures of Long Wall Sections

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Internal stability of the stabilized body (Vertical shear failure (long wall sections) under a permanent state)	– (1.00)	– (1.00)	1.80
Internal stability of the stabilized body (Vertical shear failure (long wall sections) under a variable state of Level 1 seismic ground motions)	– (1.00)	– (1.00)	1.50

(b) Short walls

$$\left.\begin{array}{l} m \cdot \dfrac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\ \\ R_k = \dfrac{1}{2}\alpha\beta q_{uc_k} \text{、} \quad S_k = \left(T'_{1_k} + w_{m_k} h_m + w_{i_k} D_s\right) L_s / \left(2D_s\right) \end{array}\right\} \quad \textbf{(5.5.13)}$$

where

R_k : characteristic value related to a resistance term (kN/m)

S_k : characteristic value related to a load term (kN/m)

α : factor for an effective cross-sectional area

β : reliability index of an overlap section between improvement piles

T_1' : toe pressure after dispersion inside a mound (excluding the self-weight of the mound) (kN/m^2) (refer to **Fig. 5.5.9**)

q_{uc} : standard design strength (kN/m^2)

w_m : unit weight of a mound (submerged unit weight when submerged) (kN/m^3)

h_m : thickness of a mound (m)

w_i : unit weight of a stabilized body (submerged unit weight when submerged) (kN/m^3)

D_s : vertical length of a short wall (m)

L_s : length of a short wall in a normal direction (m)

γ_R : partial factor multiplied by a resistance term

γ_S : partial factor multiplied by a load term

m : adjustment factor.

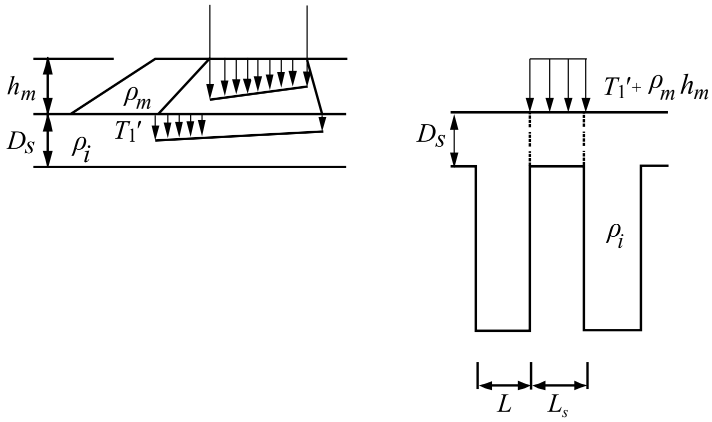


Fig. 5.5.9 Schematic Calculation Diagram of Vertical Shear Stress (Short Wall)

The partial factors to be used in the examination of the vertical shear failures of short wall sections can be selected from **Table 5.5.6**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience.

Table 5.5.6 Partial Factors to be used in the Examination of Vertical Shear Failures of Short Wall Sections

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Internal stability of the stabilized body (Vertical shear failure (short wall sections) under a permanent state)	– (1.00)	– (1.00)	1.80
Internal stability of stabilized body (Vertical shear failure (short wall sections) under a variable state of Level 1 seismic ground motions)	– (1.00)	– (1.00)	1.50

⑥ **Examination of extrusion**

(a) Because wall-type improved ground have a large number of long walls which are connected to each other through short walls with soil remaining untreated between the long walls, there may be a risk of extrusion failures of the untreated soil between the long walls depending on the intervals of the walls, the strength of the untreated soil and the thicknesses of the backfill layers. Thus, it is necessary to verify the possibility of extrusion failure of the untreated soil between long walls[123].

(b) **Fig. 5.5.10** is a schematic diagram of the extrusion of untreated soil from a wall-type improvement body.

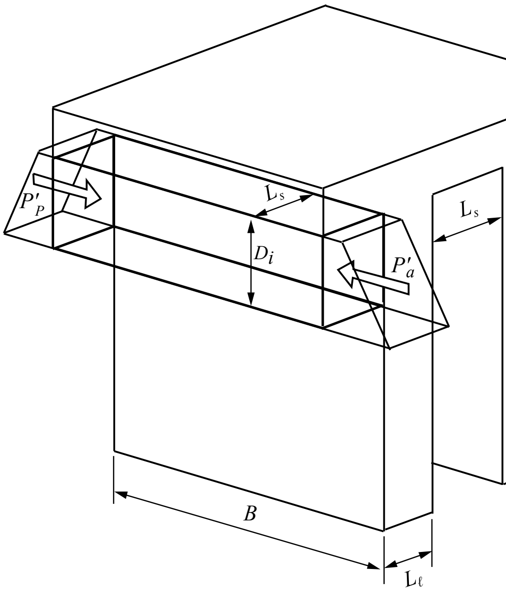


Fig. 5.5.10 Schematic Diagram of the Extrusion of Untreated Soil

(c) The extrusion of untreated soil between long walls can be examined through repeated calculations using the **equation** (**5.5.14**) while changing the values of D_i.

$$\left.\begin{array}{l} m \cdot \dfrac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\ R_k = 2(L_s + D_i) C_{u_k} B + P_{ph}{}'_k \\ S_k = P_{ah}{}'_k + k_{h2_k} w_{i_k} B D_i L_s + h_{wd} \rho_w g D_i L_s \end{array}\right\} \quad \textbf{(5.5.14)}$$

where

R_k : characteristic value related to a resistance term (kN/m)

S_k : characteristic value related to a load term (kN/m)

L_s : length of a short wall in the normal direction (m)

D_i : depth from the lower edge of a short wall to an object cross section (m)

C_u : average shear strength of untreated soil (at the intermediary depth between the lower edge of a short wall and the object cross section) (kN/m^2)

B : improvement width (m)

P_{ah}',P_{ph}' : horizontal components of active and passive earth pressure acting on the untreated soil between long walls (from the lower edge of a short wall to D_i) (kN)

k_{h2} : seismic coefficient for verification when calculating the inertia force acting on improved ground

h_w : difference between the residual water level and the water level on the offshore side (m)

w_i : saturated unit weight of untreated soil (kN/m^3)

$\rho_w g$: unit weight of seawater (kN/m^3)

γ_R : partial factor multiplied by a resistance term

γ_S : partial factor multiplied by a load term

m : adjustment factor.

(d) The partial factors to be used in the examination of the extrusion of untreated soil between long walls can be selected from **Table 5.5.7**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience.

(e) The partial factors listed in **Table 5.5.7** are set with reference to the safety levels in the past standards based on the use of the characteristic values of the physical properties of ground specified in **Part II, Chapter 3, 2.1 Methods for Estimating the Physical Properties of Ground**.

Table 5.5.7 Standard Values of the Partial Factors to be used in the Examination of the Extrusion of Untreated Soil

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Internal stability of the stabilized body (Extrusion under a permanent state)	0.81	1.04	– (1.00)
Internal stability of the stabilized body (Extrusion under a variable state of Level 1 seismic ground motions)	– (1.00)	– (1.00)	1.00

(3) Examination of slip circle failures

① For the examination of slip circle failures, refer to **Part III, Chapter 2, 4 Stability of Slopes**.

② Because stabilized bodies have sufficiently larger strength than ordinary soil, the examination of slip circle failure with failure surfaces passing through the stabilized bodies can be omitted.

(4) Examination of displacement

① When the improved ground is a floating type, the improved ground is subjected to lateral displacement due to the actions of reclaimed soil, waves and seismic ground motions, and vertical displacement due to consolidation. Therefore, it is necessary to preliminarily examine the measures against any estimated displacement so as to enable the facilities to fulfill the required performance.

② Regarding the sliding and slip circle failures of improved ground, because there is a certain degree of relationship between the ratio of the design values of resistance to those of the effects of actions and instantaneous displacement due to lateral displacement of the stabilized bodies, the necessity of examining the lateral displacement can be determined in accordance with the safety margins against such failures. Furthermore, when it is determined that the layer thicknesses of the untreated soil immediately beneath the stabilized bodies are constant, and the estimated displacement in the horizontal direction can satisfy the performance requirements of the facilities, the examination of the displacement of improved ground can be limited to only displacement due to consolidation.

③ Even for bottom fixed type improved ground, it is necessary to examine the amount of consolidation settlement when cohesive soil layers exist below the bearing layers to cope with the possible vertical displacement of stabilized bodies due to consolidation settlement.

④ It is preferable to determine the allowable displacement of improved ground appropriately in accordance with the required performance of facilities.

5.5.5 Deep Mixing Method as a Liquefaction Countermeasure

(1) The deep mixing method has been applied mostly to the stabilization of soft ground to be the foundation of gravity-type superstructures such as caisson breakwaters, revetments and quaywalls. Recently, there has been an increasing number of cases of applying the deep mixing method to liquefaction countermeasures using on land construction machines. In a questionnaire survey on damages in about 850 actual cases using the deep mixing method in 11 prefectures having municipalities with observation records of earthquakes with an intensity of 5 or more in the Tohoku and Kanto regions when the Great East Japan Earthquake occurred in 2011, all the respondents, about 800 cases, reported no damage[124)]. Among the respondents, about 120 cases using the deep mixing method were implemented as liquefaction countermeasures.

(2) There is an economic version of the deep mixing method which constructs grid type improvement with reduced improvement area ratios. Improving soft ground subjected to liquefaction using grid type improvement can prevent liquefaction in a manner that reduces the shear deformation of the ground subjected to liquefaction surrounded by stabilized bodies during earthquakes[125), 126), 127), 128)]. There have been reports that liquefaction has been prevented in the reclamation areas to which grid type improvement were actually applied[129), 130)]. Among the cases in (1) above, where the deep mixing method was implemented as a liquefaction countermeasure, grid type improvement was used in about 50 of the cases.

(3) Takahashi et al. has further modified the method using grid type improvement and proposed an improved version of the method with partially floating grid type improvement[125)]. The **References 131)** and **132),** which introduce a method for examining the internal and external stability of partially floating grid type improvement and numerical analysis models, can be used as references.

5.5.6 Deep Mixing Method to Improve Resistance against Passive Earth Pressure on the Front Faces of Sheet Piles

(1) There are cases where the deep mixing method has been applied to the ground in front of sheet piles as seismic reinforcement of existing sheet pile revetments[133)]. In these cases, the ground in front of sheet piles is improved using the jet grouting method so as to enhance the integrity of the existing structures with stabilized bodies constructed using the deep mixing method.

(2) When applying the deep mixing method to the ground in front of sheet piles for the purpose of improving the resistance of the sheet piles against passive earth pressure, either the block type or elliptic overlap type shall be used in principle. It shall be noted that there have been very few cases of applying other types than the block and elliptic overlap types to actual construction, and, therefore, the methods for calculating earth pressure remain to be fully elucidated[134)] .

(3) The modulus of horizontal subgrade reaction has complex properties; for example, even identical ground shows different values for the modulus of horizontal subgrade reaction depending on the displacement amounts of wall bodies and loading rates (loading time). Thus, when considering the increments in the modulus of horizontal subgrade reaction as the improvement effect of the deep mixing method in the calculation of the cross sections of sheet piles, it is necessary to comprehensively evaluate the increment amounts while taking into consideration the ground conditions, the structures of the sheet piles, the scale of backfill loading and the construction speeds[135)] .

(4) In recent researches, for the case of improving ground in front of cantilevered sheet pile revetments through the deep mixing method, there have been proposals of a design method using simplified beam-spring models based on the FEM analyses and centrifugal model tests[136), 137)] . In addition, there are cases of evaluating the effects of improving the ground in front of sheet piles on reductions in the deformation of quaywalls and fracture moment generated in sheet piles through centrifuge model tests and numerical analyses[138), 139, 140)].

5.6 Lightweight Treated Soil Method

(1) Definition and outline of the lightweight treated soil method

① The provisions in this section can be applied to the performance verification of the lightweight treated soil method.

② The lightweight treated soil method is meant to produce lightweight and stable ground in a manner that prepares soil soil by adding and mixing binder and lightweight materials with dredged soil or construction waste soil in a slurry form with the water content adjusted to be higher than the liquid limit as original material soil, and uses the soil for landfilling or backfilling. The types of treated soil with foam and expanded beads used as lightweight materials are called foam treated soil and expanded bead treated soil, respectively. Lightweight treated soil has the following characteristics:

(a) Because of its light weight, with approximately one half of the sand in the air and one fifth in the seawater, it can prevent ground settlement when used for landfilling and backfilling.

(b) Because of its light weight and high strength, it can reduce earth pressure during normal operation and in the event of an earthquake, thereby enabling highly earthquake-resistant facilities or land to be constructed.

(c) By enabling dredged soil which is constantly generated from ports and disposed of as waste and construction waste soil generated from on land development to be used as original material soil, it can reduce the burden on waste disposal sites.

③ For the details of the performance verification of the lightweight treated soil method, refer to the **Technical Manual for the Lightweight Treated Soil Method in Ports and Airports (Revised Version)**[141] .

(2) Fundamentals of performance verification

① Because the lightweight treated soil is ground material subjected to lightweight and stabilization treatment, its performance verification can be carried out in accordance with the performance verification methods for soil stipulated in **Part III, Chapter 2, 3 Foundations** and **Part III, Chapter 2, 4 Stability of Slopes**.

② Because the lightweight treated soil is ground material subjected to lightweight treatment, the performance verification method of general earth structures can also be basically applied to the lightweight treated soil, except for the mix proportion tests[142), 143)] .

③ An example of the performance verification procedure when using the lightweight treated soil method in backfilling for revetments and quaywalls is shown in **Fig. 5.6.1**.

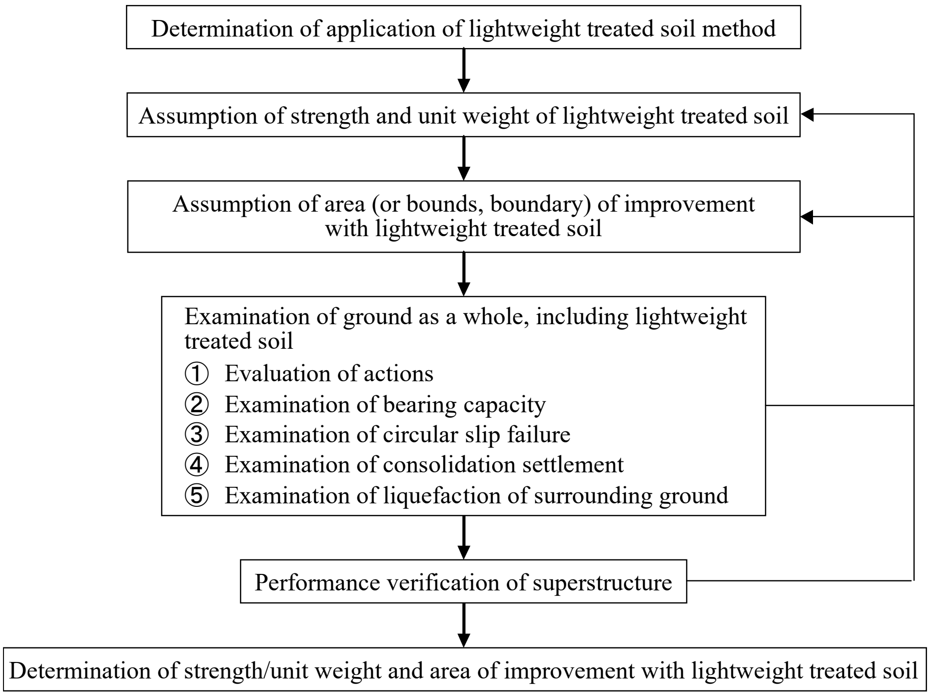


Fig. 5.6.1 Example of the Performance Verification Procedure for the Lightweight Treated Soil Method

④ In the performance verification of the lightweight treated soil method, the following actions are generally considered.

(a) The self-weight of lightweight treated soil, superstructure (caissons, etc.), backfilling materials, filling materials, reclaimed soil and mound materials (that consider buoyancy)

(b) Earth pressure and residual water pressure

(c) Surcharges (including fixed, variable and cyclic loads)

(d) Tractive force by ships and reaction of fenders

(e) Actions in respect to seismic ground motions

The concepts in **Part III, Chapter 2, 5.18 Active Earth Pressure of Geotechnical Materials Treated with Binders** can be applied to the calculations of earth pressure and the earth pressure during an earthquake.

⑤ The soil constant of lightweight treated soil shall be basically evaluated by means of laboratory tests, which take into consideration the environmental and construction conditions at the sites. Generally, they may be evaluated based on the following concepts:

(a) Unit weight

The unit weight of lightweight treated soil γ_t may be set within a range from 8 to 13 kN/m^3 by adjusting the amounts of lightening material and added water. When used for the construction of port facilities, lightweight treated soil with a unit weight less than that of the seawater poses a risk of floating in the case of a rise in sea levels. Therefore, the following values are generally used as the characteristic values of the unit weight of lightweight treated soil.

For use underwater: γ_{tk}＝11.5 to 12.0 kN/m^3

For use in air: γ_{tk}＝10. 0 kN/m^3

The mix proportion of lightweight treated soil shall be designed by taking into consideration the fact that the unit weight of lightweight treated soil varies depending on the environmental conditions, particularly the intensity of the water pressure, during and after placement[144), 145)].

(b) Strength[146)]

The strength of lightweight treated soil is mainly attributable to the stabilized strength of cement-based binders. The standard design strength can be evaluated by unconfined compressive strength q_u and can generally be set with a range of 100 to 500 kN/m^2. Although it cannot be expected that the increase in confining pressure contributes to the increase in the strength of lightweight treated soil because of the inclusion of foam or expanded beads, the residual strength is approximately 70% of the peak strength. The characteristic values of compressive strength shall be the standard design strength and appropriately set so as to satisfy the stability and required performance of the structures of the superstructure and the ground as a whole.

Undrained shear strength c_u can be used as the characteristic value of shear strength. The value of c_u can be calculated by the **equation (5.6.1)**.

$$c_u = q_u/2 \qquad \textbf{(5.6.1)}$$

(c) Consolidation yield stress p_y

The consolidation yield stress p_y can be calculated by the **equation (5.6.2)**.

$$p_y = 1.4q_u \qquad \textbf{(5.6.2)}$$

(d) Elastic modulus E_{50}

Test values can be used as the elastic modulus E_{50}, provided that the tests can be implemented by paying attention to details such as the precise measurement of small deformation amounts and the preparation of specimens with careful end finishing. In cases where such tests are not available, the deformation modulus can be calculated from unconfined compressive strength q_u by the **equation (5.6.3)**.

$$E_{50} = 100\sim200q_u \qquad \textbf{(5.6.3)}$$

where

E_{50} : elastic modulus (kN/m^2)

q_u : unconfined compressive strength (kN/m^2).

The elastic modulus shown above corresponds to a strain level of 0.3 to 1.0%.

(e) Poisson's ratio

Poisson's ratio of lightweight treated soil varies depending on the intensity of stresses and before or after the attainment of peak strength. When the surcharge is less than the consolidation yield stress of lightweight treated soil, the following mean values may be used.

Lightweight treated soil with foam: $v = 0.10$

Lightweight treated soil with expanded beads: $v = 0.15$

(f) Dynamic property

The values necessary for dynamic analyses such as shear modulus G, damping factor h, strain dependency of G and h, and Poisson's ratio v are generally obtained through laboratory tests, or, as a simplified alternative method, these values can be determined through the estimation method which has been used for ordinary soil with reference to the test values of ultrasonic wave propagation velocity tests.

(3) Examination of improvement areas[147)]

① The level of weight-saving to be achieved through the lightweight treated soil method shall be appropriately determined by taking into consideration the types of object structures, action conditions and the stability of the structures and the ground as a whole.

② The areas subjected to soil improvement through the lightweight treated soil method are generally determined in accordance with the purposes of weight-saving; that is, the areas subjected to soil improvement shall be determined based on the allowable amounts of settlement or displacement when the purpose is to prevent settlement or lateral displacement, the results of slope stability analyses when the purpose is to ensure stability, and required earth pressure reduction conditions when the purpose is to reduce earth pressure[148)] .

(4) Concept of mix proportion design

The mix proportion of lightweight treated soil shall be designed by following the instructions below.

① The mix proportion of lightweight treated soil shall be designed so as to achieve the strength and unit weight required at sites.

② The types of binder and lightweight materials shall be determined after confirming their effectivity through tests.

③ The mix proportion of lightweight treated soil shall be determined through laboratory mix tests based on the strength and unit weight required in the performance verification. The mix proportion shall be appropriately corrected at sites in accordance with the differences between the laboratory mix tests and actual construction conditions.

④ The mix proportion design is preferably implemented in the general procedures as shown below.

(a) Implementation of the investigations and tests to understand the basic properties of original material soil and soil to be improved before designing lightweight treated soil. The standard test items are as listed below.

1) Types of tests related to original material soil

i. Soil particle density test

ii. Water content test

iii. Grain size test

iv. Liquid limit and plastic limit tests

v. Wet density test of soil

vi. pH test

vii. Organic carbon test (or ignition loss test)

2) Types of tests related to soil to be treated

(Physical tests immediately after production)

i. Density test

ii. Flow test

iii. Underwater separation resistance test (when used underwater)

(Physical tests after stabilization)

i. Wet density test

ii. Unconfined compression test

iii. Hexavalent chrome elution test

(b) Laboratory mix tests of lightweight treated soil (for water content, density and unconfined compressive strength as standard test items) shall be conducted to set the amounts of water, binder and lightweight materials to be added to original material soil.

⑤ The flow values of treated soil affect material separation during mixing, the difficulty in mixing, transportation (pumping) distances, material separation during placement and the accuracy of surface finishing. Generally, the flow values are determined by the relationships among the properties of the original material soil, the amount of water, the types and amount of stabilization materials, and the types and amount of lightweight materials such as foam and expanded beads, but it is preferable to set the flow value in a range of 130 to 230 mm[149), 150)] .

⑥ The target strength of the laboratory mix tests shall be the value obtained by multiplying the standard design strength by a premium rate α while taking into consideration the differences between the design and field strength and the variation of field strength. The premium rate α is expressed by the ratio of laboratory mix test strength and standard design strength and is generally set at the following value.

$$\alpha = 2.2$$

(5) Workability confirmation test

① When there are no records to be used as reference or there are special construction conditions, it is preferable to conduct tests to confirm workability before the actual implementation of the lightweight treated soil method.

② The methods for confirming workability include water tank placement tests that simulate the actual construction conditions and test mixing using actual mixers[151)] .

③ In workability confirmation tests, it is necessary to confirm the mixing state of lightweight treated soil and density and strength after placement.

5.7 Blast Furnace Granulated Slag Replacement Method

5.7.1 General

(1) It is necessary to give due consideration to the characteristics of blast furnace granulated slag when using it as material for backfilling mooring facilities and revetments, landfilling, covering soft ground and sand compaction piles with high replacement area ratios.

(2) Blast furnace granulated slag is a granular material which has latent hydraulic properties and becomes hardened over time[152)] . When comparing a granular state and a hardened state of blast furnace granulated slag used as a backfill material, the granular state generally imposes more severe conditions in the performance verification than the hardened state. However, depending on the situation, the hardened state may impose more severe conditions for facilities. Thus, it is preferable to fully examine the applicability of blast furnace granulated slag by evaluating the respective conditions.

(3) Blast furnace granulated slag is produced in factories, and when produced in an identical factory, has relatively small variations in material characteristics. However, there may be cases where the material characteristics of the blast furnace granulated slag differ factory by factory. Thus, it is preferable to investigate the material characteristics such as the physical characteristics of the blast furnace granulated slag to be used in actual work.

(4) For the components, properties and standard physical characteristics of blast furnace granulated slag, refer to **Part II, Chapter 11, 7.2.2 Iron and Steel Slag**, the **Recycling Guidelines in Port and Airport Development**[153)] and the **Technical Manual for the Utilization of Granulated Slag in Port and Airport Development**[154)] .

5.7.2 Fundamentals of Performance Verification

(1) Blast furnace granulated slag is considered to become hardened over time. Thus, when used as a backfilling material, blast furnace granulated slag is considered to have no cohesion while it is in the granular state before

becoming hardened, and both the cohesion and the angle of shear resistance as the maximum shear strength once it becomes hardened. However, the examination of residual strength shall be carried out based only on the angle of shear resistance without cohesion.

(2) When blast furnace granulated slag is used for sand compaction piles with high replacement area ratios, the performance verification shall be carried out based only on the angle of shear resistance as is the case with sand.

(3) It shall be noted that there may be cases of a significant reduction in permeability when construction methods to be used cause fractures of the particles of blast furnace granulated slag.

(4) Generally, there is no need to consider the compression overtime of blast furnace granulated slag when used as a material for backfilling, landfilling or sand mats.

(5) When used as a material for backfilling, blast furnace granulated slag in the hardened state is not thought to undergo liquefaction. However, considering that it may undergo liquefaction when in the granular state before becoming hardened, the examination of liquefaction shall be carried out as needed.

(6) The effect of blast furnace granulated slag on the corrosion of steel materials is considered to be equal to that of general soil.

(7) For other detailed items requiring caution when using blast furnace granulated slag, refer to the **Technical Manual for the Utilization of Granulated Slag in Port and Airport Development**[154].

5.8 Premix Method

5.8.1 Fundamentals of Performance Verification

(1) Scope of application

① The performance verification method described in this section can be applied to the performance verification of ground improved through the premix method for the purpose of reducing earth pressure and preventing liquefaction.

② The definitions of the terms related to this method are as follows.

Treated soil: Soil improved with binders
Treated ground: Ground filled and improved with treated soil
Area of improvement: Areas where treated ground is to be developed
Additive ratio of binders: A weight ratio of binders to the dry weight of the base material, expressed in percentages
Earth pressure reduction: A countermeasure to reduce earth pressure on a wall surface (active earth pressure)

③ The premix method is to develop stable ground in a manner that prepares treated soil by adding and mixing binders and segregation preventive agents with soil to be used for filled ground, and fills ground underwater with the treated soil. The principle of this method is to use cement-based binders for adding cohesion to the soil used for filled ground through the chemical stabilization reaction between the soil and the binders[155), 156)] . Here, the term filled ground means the ground at the back of mooring facilities and revetments filled with backfill soil, the ground inside cells filled with infill soil and the ground filled after excavation with replacement soil or original soil.

④ The types of soil (base materials) applicable to this method are sand and sandy soil. Cohesive soil is not the case because when it is used for the premix method, it can cause significant fluctuations in the mechanical characteristics of the treated soil depending on the properties of the cohesive soil. Nevertheless, when it is necessary to use cohesive soil as a base material, the applicability of the cohesive soil shall be examined in accordance with its properties.

⑤ The premix method can be applied not only to earth pressure reduction and liquefaction prevention but also to the reinforcement of filled ground to meet the requirements of the facilities to be constructed. In such cases, the strength of the filled ground shall be evaluated appropriately.

⑥ For other items related to the performance verification and implementation of the premix method, refer to the **Reference 157).**

(2) Basic concept

① For the performance verification, it is necessary to appropriately determine the required strength of the treated soil, the additive ratios of the binders and the areas of improvement.

② When evaluating the earth pressure reduction effect or examining the stability of the ground with respect to slip circle failures, treated soil shall be regarded as a c-ϕ material.

③ The areas of improvement shall be determined based on the stability of not only the treated ground but also the facilities (overall stability) with respect to sliding failures because there is a possibility that the treated ground has significantly larger rigidity than the surrounding untreated ground and will behave as a rigid body during the actions of seismic ground motions.

④ It is preferable to determine the standard design strength of the treated ground and the areas of improvement through the example procedure shown in **Fig. 5.8.1.**

⑤ When implementing the premix method for the purpose of liquefaction prevention, the additive ratios of binders shall be determined accordingly.

⑥ Generally, treated soil using sandy soil as a base material can be regarded as a c-ϕ material; therefore, the shear strength of the treated soil can be calculated by the **equation** (**5.8.1**).

$$\tau_f = c + \sigma' \tan\phi \tag{5.8.1}$$

where

τ_f : shear strength of the treated soil (kN/m^2)

σ' : effective confining pressure (kN/m^2)

c : cohesion (kN/m^2)

ϕ : angle of shear resistance (°)

c and ϕ correspond, respectively, to the cohesion c_d and the angle of shear resistance ϕ_d obtained through consolidated and drained triaxial compression tests.

⑦ The earth pressure of treated ground acting on wall surfaces can be calculated through the method specified in **Part III, Chapter 2, 5.18 Active Earth Pressure When Using Soils Treated with Binders**.

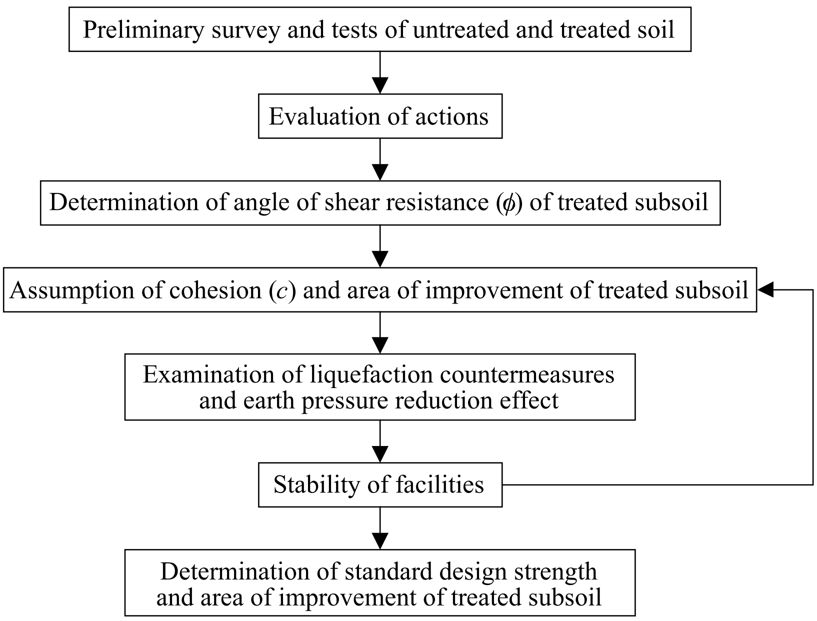


Fig. 5.8.1 Example of the Performance Verification Procedure for the Premix Method

5.8.2 Preliminary Surveys

(1) It is necessary to appropriately evaluate the properties of soil to be used in the premix method through preliminary surveys and tests.

(2) The items for preliminary surveys and tests include tests on particle density, water content, grain size, and the maximum and minimum densities of filling soil, as well as records of surveys and field tests on the soil properties of existing filled ground nearby.

(3) Due consideration shall be given to the water contents and fine particle content rates of filling soil because these factors affect the selection of the methods for mixing treated soil with binders and the strength development of treated soil after mixing.

(4) The density of treated ground after filling shall be appropriately estimated in advance. Furthermore, due consideration shall be given to the density of treated ground after filling because it is the basic data required when setting the density of specimens for laboratory mix tests, and thereby has a large influence on the test results.

(5) The density of treated ground shall be appropriately estimated with reference to the soil data, such as the *N*-values of existing filled ground, or the data on the existing filled ground treated by the premix method. When referring to existing soil property data, it is necessary to confirm the similarities between the properties of the filling soil to be used in the premix method and the properties in the reference data through grain size distribution curves or other means, and the similarity of the filling methods between the premix method and those in the reference data. In the event of difficulties in obtaining the appropriate reference data for estimating the density of treated soil after filling, it is preferable to conduct field tests. If field tests are not feasible, the density of treated soil after filling shall be set with the assumption that the ground has been loosely filled.

(6) According to previous surveys, the *N*-values of existing filled ground without treatment can be around 10, although they widely vary.

5.8.3 Actions

The main actions to be considered in the performance verification of the premix method are surcharge, the self-weight of treated ground, buoyancy, earth pressure, residual water pressure, fender reaction force, seismic ground motions and waves.

5.8.4 Determination of Strength of Treated Soil

(1) The strength of treated soil needs to be determined in such a way as to yield the required improvement effects by taking account of the purposes and conditions of the application of this method.

(2) When implementing the method for the purpose of reducing earth pressure, the cohesion c of treated soil needs to be determined so that the earth pressure can be reduced to the required levels.

(3) When implementing the method for the purpose of preventing liquefaction, the strength of the treated soil needs to be determined so that the treated soil does not undergo liquefaction.

(4) There is a significant relationship between the liquefaction strength and the unconfined compressive strength of treated soil. It is reported that treated soil with an unconfined compressive strength of 100 kN/m^2 or more does not undergo liquefaction. Therefore, when implementing the method for the purpose of preventing liquefaction, the unconfined compressive strength of 100 kN/m^2 can be used as the index value of the strength of the treated soil. When setting the unconfined compressive strength of treated soil at less than 100 kN/m^2, it is preferable to confirm that the treated soil does not undergo liquefaction through cyclic triaxial compression tests.

(5) Generally, the cohesion of treated ground can be calculated in a manner that first estimates the internal friction angle of the treated ground, then inversely calculates the cohesion of treated soil by substituting the estimated internal friction angle and the target earth pressure after reduction by the earth pressure calculation formula while taking into consideration cohesion and the angle of shear resistance.

(6) According to the results of consolidated and drained triaxial compression tests of treated soil with a binder additive ratio of 10% or less, the internal friction angle of the treated soil are equal to or slightly larger than those of the base material soil. Accordingly, in order to be on the safe side in the performance verification, the internal friction angle of the treated soil can be assumed to be the same as those of the untreated soil.

(7) When obtaining the internal friction angle through triaxial compression tests, the angle of shear resistance is obtained from consolidated and drained triaxial compression tests based on the estimated density and effective overburden pressure of the treated ground after filling. The internal friction angle used in the performance verification shall be generally smaller than those obtained from the tests by 5 to 10º. When a triaxial compression test is not performed, ϕ can be obtained from the estimated *N*-values of the treated ground after filling (with attention paid to the use of the *N*-values of the untreated ground).

5.8.5 Mix Proportion Design

(1) The mix proportion of treated soil shall be determined by conducting the appropriate laboratory mix tests. It is preferable to consider a possible decline in strength at the sites because there are cases where the field strength of the treated soil is lower than the laboratory test results.

(2) The purpose of laboratory mix tests is to obtain the relationship between the strength of the treated soil and additive ratios of binders, and to determine the appropriate additive ratios of binders so as to obtain the required strength for the treated soil. The relationship between the strength of the treated soil and the additive ratios of binders is greatly affected by the test conditions such as the types and density of soil. Therefore, laboratory mix tests are preferably conducted under conditions similar to the actual site conditions.

(3) When implementing the method for reducing earth pressure, the relationships among cohesion c, the internal friction angle ϕ and the additive ratios of binders shall be obtained through consolidated and drained triaxial compression tests. When implementing the method for preventing liquefaction, the relationship between the unconfined compression strength and the additive ratios of binders shall be obtained through unconfined compression tests.

(4) It is important to understand the difference between field and laboratory strength when setting the overdesign factor applied to the field mix proportion design. According to past performance records, laboratory strength is generally larger than field strength, and overdesign factors α are around 1.1 to 2.2. Here, the overdesign factor α is defined as the ratio of laboratory strength to field strength in terms of the unconfined compressive strength.

(5) Field tests shall be conducted when it is necessary to figure out the density and the variance of strength of treated ground after the filling, and the difference between field and laboratory strength.

5.8.6 Examination of Areas of Improvement

(1) The areas of improvement through the premix method shall be appropriately determined by examining the stability of the facilities and ground as a whole while taking into consideration the structural types of the object facilities and action conditions.

(2) The areas of improvement in the case of implementing the method for reducing earth pressure shall be set so as to ensure the stability of the object facilities with respect to the earth pressure of treated ground acting on the facilities.

(3) The areas of improvement in the case of implementing the method for preventing liquefaction shall be set so as to ensure that the liquefaction of untreated ground does not affect the stability of the object facilities.

(4) The actions and resistance considered to be applied to facilities and treated ground in cases with or without the liquefaction of untreated ground behind treated ground are shown in **Figs. 5.8.2** and **5.8.3**, respectively.

(5) When implementing the method for either earth pressure reduction or liquefaction prevention, it is necessary to examine the entire stability of the treated ground, including the object facilities, with respect to sliding failures during the actions of seismic ground motions and slip circle failures under a permanent situation.

① **Examination of sliding failures during the actions of seismic ground motions**

The sliding failures of treated ground during the actions of seismic ground motions shall be examined because of a possibility that the treated ground may slide as a rigid body. The appropriate values of the partial factors γ_a used in the examination shall be 1.0 or higher in general and the characteristic values of the friction coefficient on the bottom of the treated ground can be 0.6. In the calculation of sliding resistance on the bottom faces of treated ground against the cohesive original ground, the cohesion of the original ground can be used for the calculation. The **equation** (**5.8.2**) below for examining the stability of untreated ground with no risk of liquefaction and with respect to sliding failures deals with actions such as the earth pressure in simple cases where the residual water level is on the ground surface. When the residual water level is in the ground and the original untreated ground has a risk of liquefaction, the ground above the residual water level can be considered to undergo liquefaction all the way to the ground surface with excess pore water pressure propagated from the lower ground.

Diagrams of the actions to be considered in the cases of implementing the method for earth pressure reduction and liquefaction countermeasures are shown in **Figs. 5.8.2** and **5.8.3,** respectively. It shall be noted that, in cases where the treated ground has shapes which cause the values of ϕ in both figures to be negative, the treated ground has a risk of sliding failure due to the lateral deformation of structures or liquefaction. In addition, when implementing the method for liquefaction countermeasures, the shapes of the treated ground causing the value of ϕ to be negative are disadvantageous to sliding failure prevention in that the treated ground is subjected to upward excess pore water pressure generated in the untreated ground, thereby reducing its effective weight.

(a) When implementing the method for the purpose of earth pressure reduction

With the directions of the respective actions and resistance shown in **Fig. 5.8.2** assumed to be positive, the stability of the treated ground with respect to sliding failures can be verified using the **equation** (**5.8.2**). In the equation, the symbol γ is the partial factor for the respective subscripts, and the subscripts k and d denote the characteristic value and design value, respectively. Furthermore, in the following performance verification, all the partial factors, including the modification coefficient, can be set at 1.0.

$$\left.\begin{aligned}
&m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\
&R_k = R_{1_k} + R_{2_k} + P_{w1_k} \\
&S_k = H_{1_k} + H_{2_k} + P_{h_k} + P_{w2_k} + P_{w3_k}
\end{aligned}\right\} \qquad \textbf{(5.8.2)}$$

The characteristic values in the above equation can be calculated as follows.

$$\left.\begin{aligned}
&R_{1_k} = f_{1_k} W_1{'}_k \\
&R_{2_k} = f_{2_k} (W_2{'}_k - P_{v_k}) \text{ (when the original ground below the treated ground is sandy soil)} \\
&R_{2_k} = c_k l_{bc} \quad \text{(when the original ground below the treated ground is cohesive soil)} \\
&P_{w1_k} = \frac{1}{2} \rho w g {h_1}^2 \\
&P_{w2_k} = \frac{7}{12} k_{h_k} \rho_w g {h_1}^2 \\
&P_{w3_k} = \frac{1}{2} \rho_w g {h_2}^2 \\
&H_{1_k} = k_{h_k} W_{1_k} \\
&H_{2_k} = k_{h_k} W_{2_k} \\
&P_{h_k} = \frac{1}{2} K_a w'_k {h_{2_k}}^2 \frac{\cos(\delta_k + \phi)}{\cos\phi} \\
&P_{v_k} = P_{h_k} \tan(\delta_k + \phi)
\end{aligned}\right\} \qquad \textbf{(5.8.3)}$$

where

R_1 : friction resistance on the bottom face of the structure (ab) (kN/m)

R_2 : friction resistance on the bottom face of the treated ground (bc) (kN/m)

P_{w1} : hydrostatic water pressure acting on the front face of the structure (af) (kN/m)

P_{w2} : dynamic water pressure acting on the front face of the structure (af) (kN/m)

P_{w3} : hydrostatic water pressure acting on the rear face of the treated ground (cd) (kN/m)

H_1 : inertia force acting on the structure (abef) (kN/m)

H_2 : inertia force acting on the bottom face of the treated ground (bcde) (kN/m)

P_h : horizontal component of the active earth pressure of untreated ground during an earthquake acting on the rear face of the treated ground (cd) (kN/m)

P_v : vertical component of the active earth pressure of untreated ground during an earthquake acting on the rear face of the treated ground (cd) (kN/m)

$\rho_w g$: unit weight of seawater (kN/m^3)

w' : submerged unit weight of untreated ground (kN/m^3)

k_h : seismic coefficient for verification

K_a : coefficient active earth pressure of untreated ground during an earthquake

h_1 : water level in front of the structure (m)

h_2 : residual water level (m) (assumed to be at the ground surface in **Fig. 5.8.2** for simplicity)

δ : angle of wall friction between the treated ground and untreated ground (cd) (^{o})

ϕ : angle of the rear face of the treated ground (cd) with respect to the vertical direction (with the counterclockwise direction assumed as positive) (^{o})

f_1 : coefficient of friction on the bottom face of the structure

f_2 : coefficient of friction on the bottom face of the treated ground (= 0.6)

c : cohesion of the original ground (kN/m^2)

l_{bc} : length of the bottom face of the treated ground (bc) (m)

(b) When implementing the method for the purpose of liquefaction countermeasures

With the directions of the respective actions and resistance shown in **Fig. 5.8.3** assumed to be positive, the stability of the treated ground with respect to sliding failures can be verified using the **equation** (**5.8.4**). In the equation, the symbol γ is the partial factor for the respective subscripts, and the subscripts k and d denote the characteristic value and design value, respectively. Furthermore, in the following performance verification, all the partial factors, including the modification coefficient, can be set at 1.0.

When untreated ground at the back of the treated ground undergoes liquefaction, the static and dynamic pressures of the untreated ground are generally considered to act on the back of the treated ground as shown in **Fig. 5.8.3**. The static pressure can be calculated by adding hydrostatic pressure to earth pressure with the coefficient of earth pressure set at 1.0. The dynamic pressure can be calculated using the **equations** (**2.2.1**) and (**2.2.2**) shown in **Part II, Chapter 4, 3.2 Dynamic Water Pressure**; provided, however, that the unit weight of the water in the **equations** (**2.2.1**) and (**2.2.2**) is replaced with the unit weight of the saturated soil.

$$\left.\begin{aligned} & m\cdot\frac{S_d}{R_d}\le 1.0 \qquad R_d=\gamma_R R_k \qquad S_d=\gamma_S S_k \\ & R_k=R_{1_k}+R_{2_k}+P_{w1_k} \\ & S_k=H_{1_k}+H_{2_k}+P_{h_k}+P_{w2_k} \end{aligned}\right\} \qquad \textbf{(5.8.4)}$$

The characteristic values in the above equation can be calculated as follows.

$$
\left.
\begin{aligned}
&R_{1_k} = f_{1_k} W_1'_k \\
&R_{2_k} = f_{2_k}\left[W_2'_k + \left\{ P_{v_k} - \frac{1}{2}\rho_w g h_2{}^2 \tan\phi \right\} \right] && \text{(when the original ground below the treated ground is sandy soil)} \\
&R_{2_k} = c_k l_{bc} && \text{(when the original ground below the treated ground is cohesive soil)} \\
&P_{w1_k} = \frac{1}{2}\rho w g h_1{}^2 \\
&P_{w2_k} = \frac{7}{12} k_{h_k} \rho_w g h_1{}^2 \\
&H_{1_k} = k_{h_k} W_{1_k}
\end{aligned}
\right\} \quad \textbf{(5.8.5)}
$$

$$H_{2_k} = k_{h_k} W_{2_k}$$

$$P_{h_k} = \frac{1}{2} K_a w'_k h_{2_k}{}^2 + \frac{7}{12} k_{h_k} \rho_w h_{2_k}{}^2$$

where

R_1 : friction resistance on the bottom face of the structure (ab) (kN/m)

R_2 : friction resistance on the bottom face of the treated ground (bc) (kN/m)

P_{w1} : hydrostatic water pressure acting on the front face of the structure (af) (kN/m)

P_{w2} : dynamic water pressure acting on the front face of the structure (af) (kN/m)

H_1 : inertia force acting on the structure (abef) (kN/m)

H_2 : inertia force acting on the bottom face of the treated ground (bcde) (kN/m)

P_h : horizontal component of the active earth pressure of untreated ground during an earthquake acting on the rear face of the treated ground (cd) (kN/m)

$\rho_w g$: unit weight of seawater (kN/m^3)

w' : submerged unit weight of untreated ground (kN/m^3)

k_h : seismic coefficient for verification

K_a : coefficient active earth pressure of untreated ground during an earthquake

h_1 : water level in front of the structure (m)

h_2 : water level to calculate the pressure P_h due to liquefaction (m) (assumed to be at the ground surface)

ϕ : angle of the rear face of the treated ground (cd) with respect to the vertical direction (with the counterclockwise direction assumed as positive) (^{o})

f_1 : coefficient of friction on the bottom face of the structure

f_2 : coefficient of friction on the bottom face of the treated ground (= 0.6)

c : cohesion of the original ground (kN/m^2)

l_{bc} : length of the bottom face of the treated ground (bc) (m).

② **Examination of stability with respect to slip circle failures under a permanent situation**

For the examination of stability with respect to slip circle failures under a permanent situation, refer to **Part III, Chapter 2, 4 Stability of Slopes**.

(6) When the stability of the facilities and ground as a whole cannot be secured, it is necessary to take measures such as the revision of the areas of improvement and an increase in the standard design strength of the treated soil.

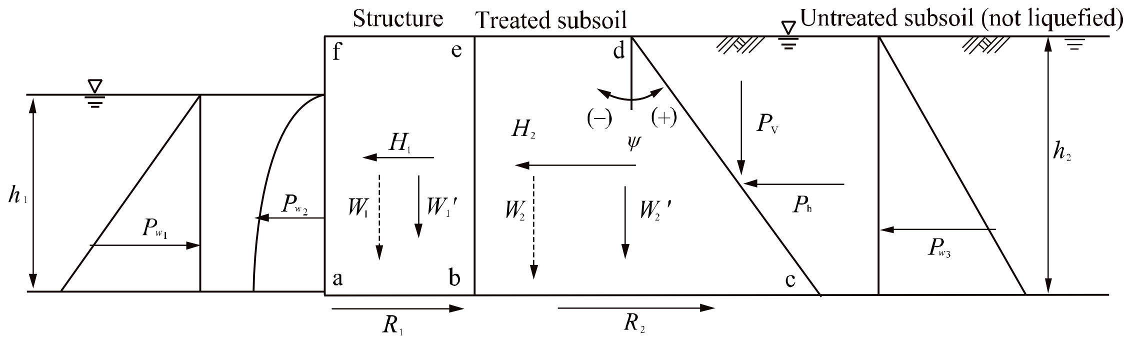


Fig. 5.8.2 Diagram of Actions to be Considered When Implementing the Premix Method for Reducing Earth Pressure

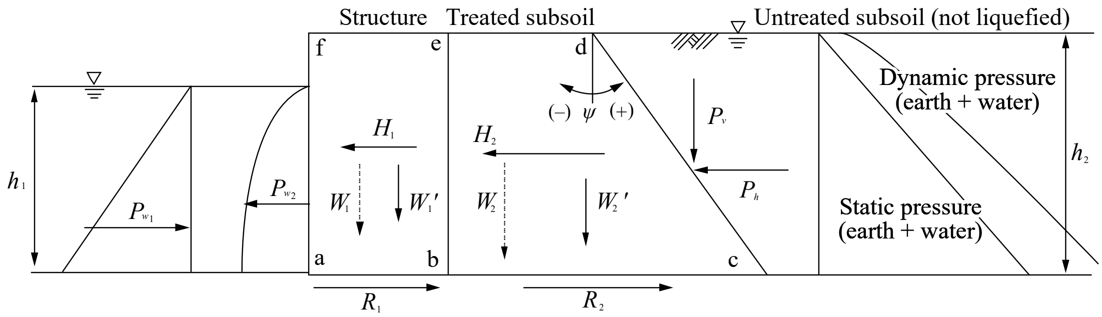


Fig. 5.8.3 Diagram of Actions to be Considered When Implementing the Premix Method for Liquefaction Countermeasures

5.9 Sand Compaction Pile Method (for the Improvement of Sandy Ground)

5.9.1 Fundamentals of Performance Verification

(1) The performance verification of the sand compaction pile method for the compaction of sandy soil shall be appropriately carried out with due consideration to the properties of the improvement object ground and the characteristics of the construction methods with reference to the performance records or the results of the field test.

(2) Purposes of improvement

The purposes of improving loose sandy ground can be largely classified into: (a) improving liquefaction strength; (b) reducing settlement; and (c) improving slope stability or bearing capacity. The risk of liquefaction can be predicted or determined through simple analyses of the *N*-values, grain size distribution and unit weight of sand, or, in cases where simple analyses are not effective, through analyses using the results of cyclic triaxial compression tests. When implementing the sand compaction pile method for liquefaction countermeasures, the appropriate areas shall be compacted so that the *N*-values of sandy ground are improved to a level where the sandy ground is clearly determined not to undergo liquefaction based on the criteria specified in **Part II, Chapter 7 Liquefaction of Ground**. When implementing the sand compaction pile method for reducing settlement, sandy ground shall be compacted as needed in accordance with the settlement calculated based on the theory of elasticity (refer to **Part III, Chapter 2, 3.5 Settlement of Foundation**).

(3) Factors affecting compaction effects

In many cases, vibrations or impacts applied to the superficial layers cannot sufficiently compact the deep end of loose sandy ground. Thus, the methods generally used for compacting loose sandy ground either install sand or gravel piles into the object ground using hollow steel pipes or apply vibrations to the surrounding ground by inserting special vibrating rods. The former methods are collectively categorized as the sand compaction pile method and are described in this section. The latter methods are categorized as the rod compaction method and vibro-flotation method, and are described in **Part III, Chapter 2, 5.11 Rod Compaction Method** and **Part III, Chapter 2, 5.12 Vibro-Flotation Method**, respectively.

Regardless of the methods to be used, the level of compaction is affected by many factors, as listed below. Thus, the prediction of compaction effects cannot be easily made only through theoretical calculations and requires data based on actual performance records. It shall also be noted that field test can improve the accuracy in predicting the construction conditions.

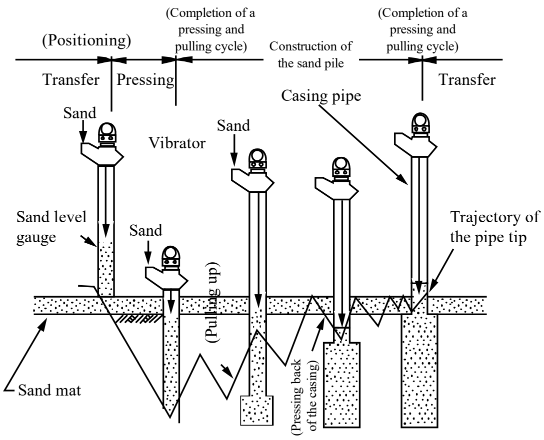
① The properties of the object soil for improvement (grain size distribution and fine particle contents (grain diameters less than 75 μm))
② The degrees of saturation and the positions of groundwater levels
③ The relative density of the object soil before improvement
④ The initial stresses in the object soil layers (overburden pressure) before improvement
⑤ The particle structures and the degrees of compaction of the object soil for improvement before improvement
⑥ The distances from the points to which vibrations are applied
⑦ The properties of the sand supply
⑧ The characteristics of the improvement methods (types and vibration application capacities of construction machines, construction methods and the skills of the engineers)

(4) Types and characteristics of construction methods

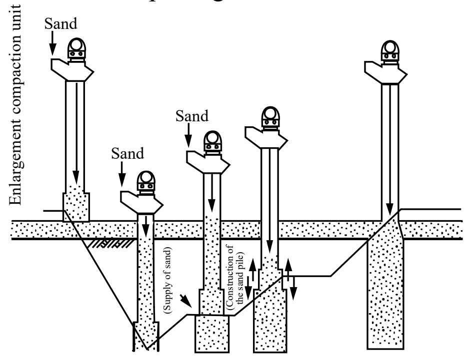
The variations of the sand compaction pile method are largely classified into: (a) sand pile formation by vibro-driving and vibro-removal; (b) expanding bottom diameter type; and (c) bottom vibration type. The characteristics of the respective variations are shown in **Table 5.9.1** and **Fig. 5.9.1**. Generally, a sand compaction pile is constructed in a manner that presses a casing pipe to a predetermined depth while vibrating it using vibration exciters installed at the head section of a construction machine, then fills the casing pipe with sand, pushes out a portion of the sand pile having a certain length from underneath the casing pipe while pulling it up, compacts and expands the diameter of the portion of the sand pile by pressing back the casing pipe while vibrating it with a vibroflot at the lower tip of the casing pipe, and repeats the above procedures until the sand pile is extended to the ground surface or to the predetermined depth. Thus, the sand compaction pile method improves loose sandy ground through the compaction of sand around the piles with vibrations and the pressing of compacted sand piles into the ground. Although the sand compaction pile method can produce a large compaction effect, it also has a large influence on the surrounding environments. There has been an accumulation of performance records of the respective compaction methods which can be used as references for examining the influences on existing facilities around the areas of improvement. Furthermore, in recent years, there have been cases of developing low vibration pile installation machines and compaction grouting machines.

Table 5.9.1 Types and Characteristics of the Sand Compaction Pile Method

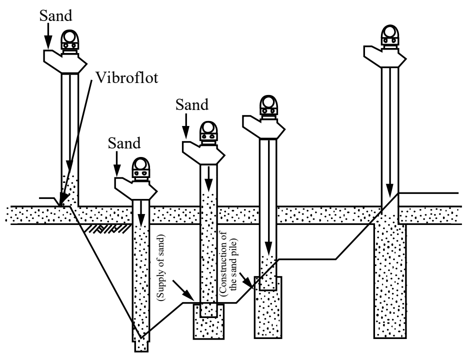
Type	Characteristics
(a) Sand pile formation by vibro-driving and vibro-removal	Typical and most frequently used sand compaction pile method for constructing sand piles by repeatedly pressing and pulling vibrated casing pipes in the ground.
(b) Expanding bottom diameter type	A method that constructs sand piles using an enlargement compaction unit attached to the tip of a casing pipe.
(c) Bottom vibration type	A method that constructs sand piles using a vibroflot attached to the lower tip of a casing pipe.



(a) Sand pile formation by vibro-driving and vibro-pulling-out



(b) Expanding bottom diameter type



(c) Bottom vibration type

Fig. 5.9.1 Examples of Construction Procedures for the Sand Compaction Pile Method

5.9.2 Verification of Sand Supply Ratios

(1) Verification of the sand supply ratios (improvement and replacement area ratios) shall be carried out based on sufficient examinations of the properties, the necessary relative densities and the *N*-values of the improvement object ground. There have been reports that ground improved through the sand compaction pile method has not undergone liquefaction even when being subjected to the actions of seismic ground motions of unexpectedly large severity. These reports suggest that the method for verifying the sand supply ratios for liquefaction countermeasures with target *N*-values set at critical *N*-values still has an unidentified safety margin. Thus, it is necessary to carefully determine whether or not to implement additional installation of sand piles even when the *N*-values measured in post construction surveys are lower than the target *N*-values.

(2) *N*-values of original ground and fine particle contents F_c

It is necessary to obtain the *N*-values and fine particle contents F_c of original ground through preliminary ground investigations. These values are of absolute necessity in that the increases of the *N*-values have a close relationship with the N-values and fine particle contents of the original ground. The increases in the N-values get smaller with an increase in the fine particle contents.

(3) Setting of target N-values

It is necessary to set target N-values for the improvement. When implementing the sand compaction pile method for liquefaction countermeasures, the target N-values shall be set at the levels (critical N-values) which can ensure that the improved ground does not undergo liquefaction due to the design actions of seismic ground motions.

(4) Sand supply ratios

The sand supply ratio is the ratio of the area covered by sand piles after improvement which occupy the original ground, as shown in the **equation (5.9.1)**.

$$F_V = \frac{A_p}{A_0} \tag{5.9.1}$$

where

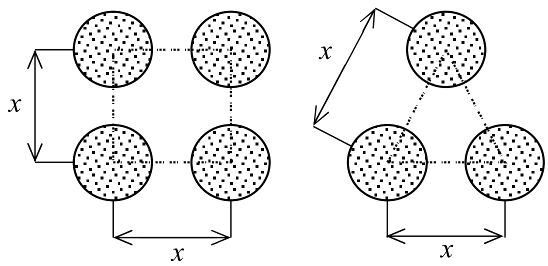
F_V : sand supply ratio

A_p : cross-sectional area of the sand pile

A_0 : area of original ground improved for each sand pile.

When sand piles are installed at an interval of x and arranged in regular triangle and square configurations, the sand supply ratios F_V can be calculated by the **equation (5.9.2)**.

$$F_V = \frac{A_p}{x^2} \quad \text{: Regular triangle configuration}$$
$$F_V = \frac{2}{\sqrt{3}}\frac{A_p}{x^2} \quad \text{: Square configuration} \tag{5.9.2}$$



(a) Regular triangle configuration (b) Square configuration

Fig. 5.9.2 Configurations of Sand Piles

(5) Setting of sand supply ratios

The methods for setting the sand supply ratios when existing data are available and when such data are not available are described separately below.

① **Setting of sand supply ratios when existing data are not available**[158)]

The sand supply ratios can be calculated using the relationship between the sand supply ratios and the N-values after improvement expressed by the **equation (5.9.3)**.

$$N_1 = C_M\left(\frac{\kappa F_V + \gamma_i^*}{c + \kappa F_V + \gamma_i^*}\right)^2 A \tag{5.9.3}$$

where

N_1 : N-value after improvement

C_M : coefficient which can be calculated by $C_M = (1/0.16)^2$

κ : coefficient which can be calculated by $\kappa = 5\times10^{-0.01Fc}$

c : coefficient which can be calculated by $c = \dfrac{0.02F_c + 0.4}{0.02F_c + 2.0}$

F_c : coefficient as fine particle content (%)

$\gamma_i{}^*$: coefficient which can be calculated by the **equation** (**5.9.4**).

$$\gamma_i^* = \frac{c\sqrt{N_0/(AC_M)}}{1-\sqrt{N_0/(AC_M)}} \qquad \textbf{(5.9.4)}$$

where

N_0 : N-value of the original ground

A : coefficient which can be calculated by the **equation** (**5.9.5**).

$$A = \frac{69+\sigma_v'}{167} \qquad \textbf{(5.9.5)}$$

where

σ_v' : effective overburden pressure at the point where the N-value is measured (kN/m^2).

By solving the **equation** (**5.9.3**) in terms of the sand supply ratio F_V, the equation to calculate the sand supply ratio to satisfy the target N-value is obtained as follows.

$$F_V = \frac{(c+\gamma_i^*)\sqrt{N_1/(AC_M)} - \gamma_i^*}{\kappa\left\{1-\sqrt{N_1/(AC_M)}\right\}} \qquad \textbf{(5.9.6)}$$

Because the **equations** (**5.9.3**) and (**5.9.4**) do not consider the effect of the increase in lateral pressure (the effect of the coefficient of earth pressure at rest K_0) due to pressing the sand piles into the ground, these equations have a tendency to underestimate the N-values after pressing sand piles into the ground when the sand supply ratios become large. In cases where sand supply ratios F_V exceed 0.2, the N-values may be calculated through an alternative method[159)] using the **equation** (**5.9.7**), which considers the effect of K_0. However, it shall be noted that the **equation** (**5.9.7**) is less accurate for making predictions because the equation is derived by using the relationship between the K_0 values and sand supply ratios, which varies greatly. Therefore, in order to be on the safe side when using the **equation** (**5.9.7**), it is preferable to set the sand supply ratios F_V at 0.2 even though the required sand supply ratios F_V to achieve the target N-values are calculated to be 0.2 or less.

$$N_1 = C_M\left(\frac{\kappa F_V + \gamma_i^*}{c+\kappa F_V + \gamma_i^*}\right)^2 A_{K_1} \qquad \textbf{(5.9.7)}$$

where

C_M : coefficient which can be calculated by $C_M = (1/0.16)^2$

κ : coefficient which can be calculated by $\kappa = 4\times10^{-0.01Fc}$

c : coefficient which can be calculated by $c = \dfrac{0.02F_c + 0.4}{0.02F_c + 2.0}$

$\gamma_i{}^*$: coefficient which can be calculated by the **equation** (**5.9.8**).

$$\gamma_i^* = \frac{c\sqrt{N_0/\left(A_{K_0}C_M\right)}}{1-\sqrt{N_0/\left(A_{K_0}C_M\right)}} \tag{5.9.8}$$

where

A_{K1} : coefficient which is calculated by the **equation** (**5.9.9**).

$$A_{K1} = \frac{69+(1+\alpha F_V)\sigma_v'}{167} \tag{5.9.9}$$

where

α : coefficient which expresses the increase rate of K_0 with respect to the sand supply ratio and can be set at $\alpha = 4$

A_{K0} : coefficient which is calculated by the **equation** (**5.9.10**).

$$A_{K0} = \frac{69+\sigma_v'}{167} \tag{5.9.10}$$

where

σ_v' : effective overburden pressure at the point where the N-value is measured (kN/m^2).

The above **equations**, (**5.9.3**) to (**5.9.10**), are derived based on existing data which show the sand supply ratios F_V of 0.07 to 0.20 and the fine particle contents Fc of 60% or less. Thus, caution is required when using these equations with sand supply ratios and fine particle contents outside of the above ranges. Furthermore, it shall be noted that these equations may overestimate κ when the fine particle contents Fc are 40% or more[158)].

② **Setting of sand supply ratios when existing data are available**

The increases in N-values after improvement through the sand compaction pile method are largely affected by the ground properties and construction methods. Thus, when abundant construction data are available or field test can be executed at sites, it is preferable to determine the increase in N-values based on the data available at the sites regardless of the method specified in (5) ①. When using the method specified in (5) ①, it is preferable to modify the parameter κ in the **equation** (**5.9.6**) as shown below using the existing data. In addition, when implementing new compaction methods, it is preferable to modify the parameter κ in the **equation** (**5.9.6**) using data specifically suitable for these new methods.

The **equation** (**5.9.11**) below is derived from the **equation** (**5.9.6**) for calculating parameter κ. With this equation, the parameter κ can be calculated from the N-values before and after pressing the sand piles into the ground, the fine particle contents and the sand supply ratios.

$$\kappa = \frac{\left(c+\gamma_i^*\right)\ \sqrt{N_1/\left(AC_M\right)}-\gamma_i^*}{F_V\left\{\ 1-\sqrt{N_1/\left(AC_M\right)}\right\}} \tag{5.9.11}$$

where

γ_i^* : coefficient which can be calculated by the **equation** (**5.9.12**).

$$\gamma_i^* = \frac{c\sqrt{N_0/\left(AC_M\right)}}{1-\sqrt{N_0/\left(AC_M\right)}} \tag{5.9.12}$$

where

C_M : coefficient which can be calculated by $C_M = (1/0.16)^2$

c : coefficient which can be calculated by $c = \dfrac{0.02F_c+0.4}{0.02F_c+2.0}$ $c = \dfrac{0.02F_c+0.4}{0.02F_c+2.0}$

A : coefficient which can be calculated by the **equation** (**5.9.13**).

$$A = \frac{69 + \sigma_v'}{167} \tag{5.9.13}$$

A relational expression between parameter κ and the fine particle contents becomes available by calculating κ from the sand supply ratios and the N-values before and after improvement, and analyzing the relationship between κ and the fine particle contents as shown in **Fig. 5.9.3**. Here, the relational expression between κ and the fine particle contents shall be basically an exponent function as shown in (5) ①.

In setting parameter κ, it is advisable not to use the data obtained when there are large differences in the fine particle contents before and after improvement, and when the N-values before improvement are larger than those after improvement. Furthermore, when the relationship between K_0 values and the sand supply ratios is measured, the parameters in the **equations** (**5.9.7**) and (**5.9.8**) considering the effect of K_0 values can be modified. For the modification of the parameters, refer to the **Reference 159)**.

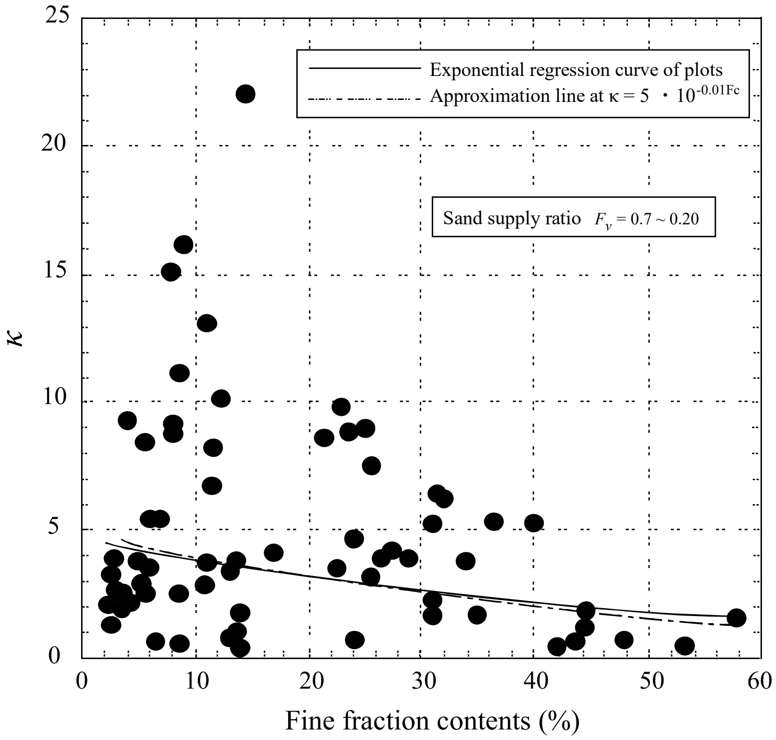


Fig. 5.9.3 Relationship between κ and Fine Particle Contents

(6) Other methods for setting sand supply ratios

The methods for setting the sand supply ratios in (5) are established based on analyses of the data on previous performances, assuming that the original ground is compacted by being subjected to repetitive shear with the sand piles pressed into the original ground. In addition to these methods, Methods A, B and C were proposed and have been used conventionally[160)]. In Method A, the relationship between the N-values before and after improvement is mapped with the sand supply ratios as a parameter so as to enable the sand supply ratios to be easily calculated. However, because Method A does not consider the effects of surcharge and fine particle contents, it has not been widely used compared to the other methods. Method B is used for obtaining the required sand supply ratios for the target N-values using an empirical equation with respect to the N-values, the effective overburden pressure and the grain sizes on the assumption that the entire volume of the sand piles pressed into the ground contributes to the compaction of the ground. However, this method does not consider the effect of the fine particle contents. Method C is basically founded on the same principle as Method B, but the major difference is that Method C considers the effect of the fine particle contents in the calculation of the sand supply ratios. Furthermore, there is another method, Method D, proposed for the calculation of sand supply ratios in consideration of the heaving of ground surfaces[160)] .

The following section describes Method C, which has the largest performance record among the four above methods for use in previous designs[161)].

① Calculations of e_{max} and e_{min} from the fine particle content.

$$e_{\max} = 0.02F_c + 1.0 \tag{5.9.14}$$

$$e_{\min} = 0.008F_c + 0.6 \tag{5.9.15}$$

② Calculation of relative density D_{r0} and e_0 from the N-value of the original ground N_0 and the effective overburden pressure σ_v'.

$$D_{r_0} = 21\sqrt{\frac{100N_0}{\sigma_v' + 70}} \quad (\%) \tag{5.9.16}$$

$$e_0 = e_{\max} + \frac{D_{r_0}}{100}\left(e_{\max} - e_{\min}\right) \tag{5.9.17}$$

③ Calculation of the rate of reduction β for the increase of the N-value due to fine particle content.

$$\beta = 1.0 - 0.5\log F_c \qquad \left(F_c > 1.0\right) \tag{5.9.18}$$

④ Calculation of the corrected N-value (N_1') by applying the rate of reduction β to the calculated N-value (N_1), assuming no fine particle content.

$$N_1' = N_0 + \frac{\left(N_1 - N_0\right)}{\beta} \tag{5.9.19}$$

⑤ Calculation of e_1 using the **equation** (**5.9.17**) shown in ② with N_0 replaced by N_1'.

⑥ Calculation of the sand supply ratio F_V from e_0 and e_1.

$$F_V = \frac{\left(e_0 - e_1\right)}{1 + e_0} \tag{5.9.20}$$

5.9.3 Performance Verification of Sand Supply Volumes

(1) The sand supply volume per unit volume is set based on the sand supply ratios F_V obtained in **Part III, Chapter 2, 5.9.2 Verification of Sand Supply Ratios**.

(2) The sand supply is subjected to volume compression with the sand piles pressed into the ground, and, therefore, the sand supply volume needs to be increased accordingly.

5.10 Sand Compaction Pile Method (for the Improvement of Cohesive Ground)

5.10.1 Fundamentals of Performance Verification

(1) Scope of application

① The scope of application of the performance verification of the sand compaction pile (SCP) method to be described in this section shall be the improvement of ground under gravity-type breakwaters, revetments and quaywalls.

② Regarding references to the lateral resistance of piles in improved ground, there are examples of field loading test results[162)] and centrifuge model test results[163), 164)] as well as a proposal of performance verification methods using the coefficient of lateral subgrade reaction[165)] . However, there have not been sufficient results from detailed examinations, and it has not been fully elucidated how improved ground behaves when the SCP method is implemented for increasing the resistance of piles and sheet pile walls against passive earth pressure or for reducing active earth pressure. Whether or not a practical evaluation formula for the shear strength of composite ground (refer to **Part III, Chapter 2, 5.10.4 Calculation Formula for the Shear Strength of**

Improved Ground) can be applied to passive regions is a subject which requires future research. In cases where the SCP method needs to be implemented for the improvement of cohesive ground under these situations, trial examinations shall be conducted in a manner that identifies slip surfaces which provide the least resistance against passive earth pressure, and examinations of composite slip failures shall be carried out when the sand piles do not reach the bearing layers. When examining slip failures with the expectation that the sand piles will produce large shear resistance, it is effective to the increase vertical loads on the sand piles by combining the counterweight fill with the SCP method (refer to **Part III, Chapter 2, 5.10.4 Calculation Formula for the Shear Strength of Improved Ground**).

③ Centrifuge model tests and seismic response analyses have been conducted for examining the vibration characteristics and seismic resistance of the cohesive ground improved through the SCP method[166), 167), 168), 169)] . In the **References 166)** and **167)**, the appropriateness of the input parameters has been verified through response analyses of gravity-type revetments, which actually underwent deformation during earthquakes, using the FLIP. There are also reports on cases of soft ground which was improved by the SCP method and successfully resisted the 2016 Kumamoto Earthquake[170)].

(2) Basic concepts

① The SCP method for the improvement of cohesive ground is to construct sand piles in a manner that drives casing pipes to predetermined depths at constant intervals in cohesive ground and discharges sand into the ground through the casing pipes while compacting the sand. The properties of the improved ground are intricately affected by (a) the strength of the sand piles, (b) the replacement area ratios of the sand piles, (c) the positional relationships of the areas of improvement in respect to the structures, (d) the action conditions (magnitude, directions, loading routes and loading rates), (e) the strength of the original ground between the sand piles, (f) the confining pressure that the sand piles receive from the surrounding ground, (g) the effect of disturbances on the original ground inside and outside the areas of improvement due to the construction of the sand piles, and (h) the characteristics of the heaved soil on ground surfaces generated through the construction of the sand piles, with or without the reuse of heaved soil.

② **Effect of the implementation of the SCP method**

The SCP method, which presses a large number of sand piles into the ground, causes a disturbance to the ground inside and around the areas of improvement with the existing soil forcibly displaced in lateral and upward directions, thereby reducing the strength of the ground. The SCP method also causes heaving of ground surfaces because of the displacement of the ground and the overflow of surplus soil in the casing pipes on the ground surface. Thus, when implementing the SCP method, it is necessary to examine the effects of ground displacement on neighboring structures.

③ **Performance verification methods**

The performance verification of composite ground comprising the sand piles and the ground between the sand piles can be carried out through either of the following two methods: one method which uses slip circle analyses based on the modified evaluation formula of average shear strength so as to reflect the characteristics of the composite ground, and another method which separates the composite ground into a portion which behaves as sandy ground and another portion which behaves as cohesive ground for the convenience of analyses, and distributes actions to the respective portions so that both portions have an equal level of safety against slip circle failures[171)] . Currently, the former method has been generally used for the performance verification of composite ground.

In the existing reports on the fracture behavior of ground improved through the SCP method with low replacement area ratios for developing foundation ground of caisson type quaywalls, it has been pointed out that fixed type improved ground and floating type improved ground with large improvement depths have not produced slip surfaces inside the areas of improvement, but have undergone deformation associated with the bending of sand piles in areas of improvement which are wider. This suggests the possibility that the stabilization mechanism on which slip circle analyses have been based does not work in composite ground improved using the SCP method with low replacement area ratios[172)]. Thus, when examining the reduction in the replacement area ratios, it is necessary to pay careful attention to measures that ensure the stability of the entire structures such as numerical analyses and model tests.

5.10.2 Sand Piles

(1) The materials for sand piles preferably have high permeability, low fine particle contents (with particle diameters less than 75 μm), favorable grain size distribution, and the property of being easily compacted, ensuring the required strength as well as being easily discharged through casing pipes. When implementing the SCP method with low proportions of sand piles in the areas of improvement (with low replacement area ratios) in expectation that the sand piles will function as drainage paths to enhance the consolidation of cohesive soil, it is of importance to give due consideration to the permeability of the materials and measures against clogging. In contrast, when implementing the SCP method with high replacement area ratios close to the ratios of the forced displacement method, the level of importance with respect to the permeability of the sand piles is low. Therefore, materials shall be selected with due consideration to the purposes of the improvement and the levels of the replacement area ratios.

Recently, there have been cases of implementing the SCP method using steel slag[173), 174)], copper slag[175), 176)], ferronickel slag[177), 178)] and oyster shells[179)] (refer to **Part II, Chapter 11, 7 Recycled Materials**). When applying the SCP method to port facilities using these types of slag, it is necessary that the slag satisfy the Environmental Safety and Quality Standards[180), 181)]. Because steel slag can be considered to have a characteristic value of 40° for the angle of shear resistance, the SCP method using steel slag can be an economical solution for cohesive ground improvement. However, it shall be noted that steel slag has hydraulic-setting properties[182)], and hardened sand piles are not always effective in the areas of improvement with eccentric loads. In addition, the permeability of steel slag is reduced over time. It has been confirmed through laboratory tests that sand piles with steel slag maintain the permeability necessary for enhancing consolidation for about 300 days after installation, but it is necessary to pay attention to the periods necessary for consolidation when relying on the permeability of the sand piles. When using oyster shells, it is preferable to confirm whether or not they satisfy the required performance through laboratory tests or field test with reference to the performance records[183)].

(2) Because there have been no particular regulations established for the materials of sand piles, it is necessary to select the appropriate materials which satisfy the requirements described above from those economically and locally available. **Fig. 5.10.1** shows examples of sand materials used in previous construction works. There have also been cases of the SCP method using sand with fine particle contents slightly larger than in these examples.

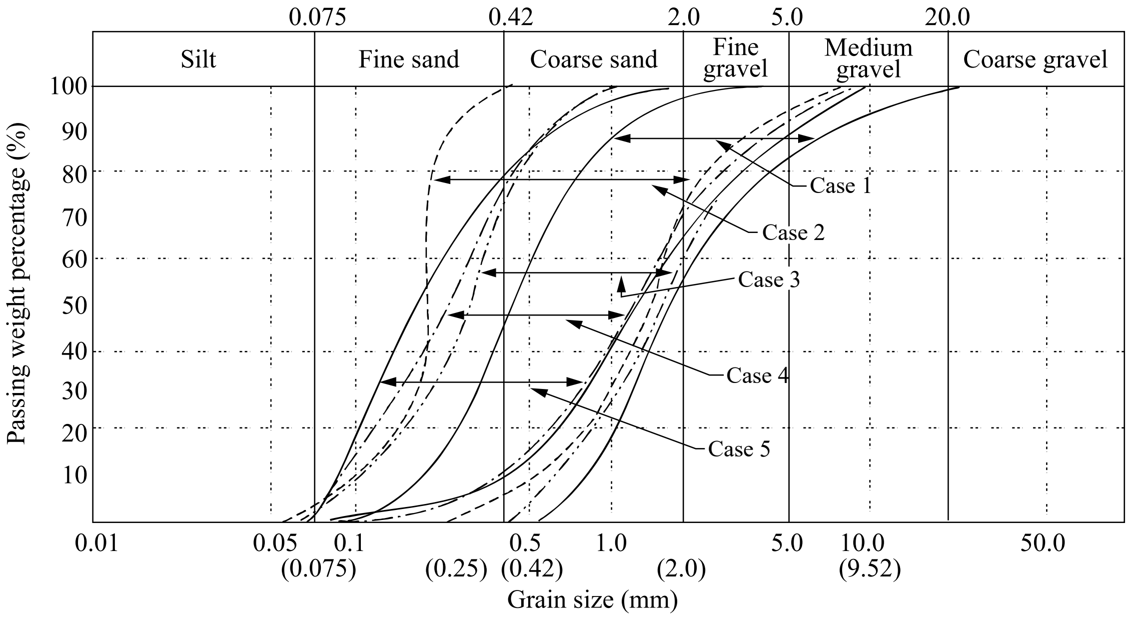


Fig. 5.10.1 Examples of the Grain Size Distribution Ranges of Sand used in the Actual Implementation of the SCP method

(3) When using sand pile materials which do not satisfy the sand grain distribution ranges of previous construction as shown in (2) above or the regulations established by other authorities for economic reasons, such materials are preferably selected based on evaluations of (a) the maximum and minimum density, compaction properties, permeability and the internal friction angle from the viewpoint of ensuring the characteristics of the sand pile materials; (b) the property of being easily discharged through casing pipes from the viewpoint of workability; and

(c) the correlation between the N-values and the relative density from the viewpoint of confirming the completion of the SCP method.

(4) Relationships of *N*-values with replacement area ratios and improvement depths

The target N-values shall be set with reference to cases of previous construction because the N-values of the sand piles vary significantly depending on the materials used and the construction conditions. Furthermore, the construction management of sand piles has been generally carried out not by finished density but by replacement area ratios in a manner that confirms whether or not the required volume of sand is forcibly pressed into the ground. Thus, the density (or N-values) of the sand piles varies depending on the material characteristics of the sand piles, the strength of the original ground, the confining pressure and the construction conditions (the compression ratios in the vertical direction during the installation of the sand piles, replacement area ratios, compaction energy, etc.).

5.10.3 Cohesive Ground

(1) Estimation of heaved soil volume

① The volume of heaved soil associated with the installation of sand piles is affected by many factors including the original ground conditions, replacement area ratios and construction conditions. Although there are some methods proposed for the estimation of heaved soil volume based on statistical analyses of the actual measurement data in previous construction works[184), 185), 186)], due consideration shall be given to prediction accuracy when using these methods.

② Shiomi and Kawamoto[184)] proposed the **equation** (**5.10.1**), where the heaving ratio μ is defined as a ratio of the volume of heaved soil to the design sand supply of the sand pile. The relationship between the estimated values and the actual measurements is shown in **Fig. 5.10.2**[184)].

The **equation** (**5.10.1**) is obtained through a multiple linear regression analysis of the data from 28 examples with pile lengths in the range of 6 m ≤ L ≤ 20 m, and additional data from 6 construction sites including 2 examples with a pile length of 21 m and 1 example with a pile length of 25.5 m. As a result of the analysis, it was found that the contribution ratios to μ decrease in the order of $1/L$, a_s and q_u. Because of its low contribution ratio, q_u (the unconfined compressive strength of the original ground) has been neglected in the **equation** (**5.10.1**).

$$\begin{aligned}\mu &= \frac{v}{v_s} \\ &= 0.356a_s + 2.803L^{-1} + 0.112\end{aligned} \qquad \textbf{(5.10.1)}$$

where

a_s : replacement area ratio

L : average length of the sand piles (m)

v : volume of heaved sand (m^3)

v_s : design sand supply (m^3)

μ : heaving ratio

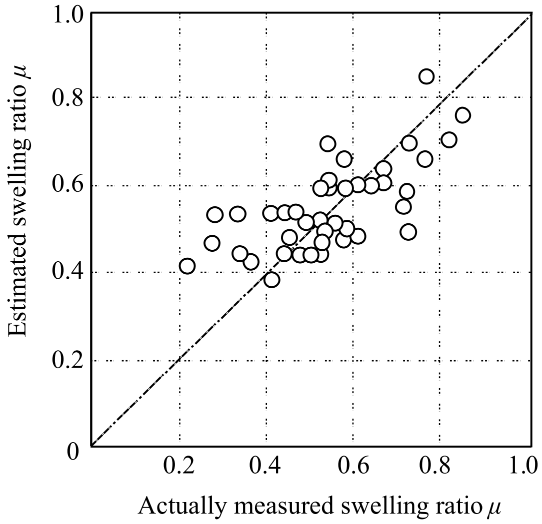


Fig. 5.10.2 Comparison of Estimated Heaving Ratios by the **equation (5.10.1)** and Actually Measured Heaving Ratios[184)]

③ Hirao et al. proposed the **equation** (**5.10.2**) for estimating the heaving ratios of large-sized sand piles with a diameter of ϕ2.0 m, which have started being used recently[185)]. It has been reported that the estimation results with the **equation** (**5.10.2**) show a relatively strong correlation with the actual measurements[185), 187)]. Thus, it is preferable to use the **equation** (**5.10.2**) for the estimation of heaving ratios for sand piles with diameters of ϕ 2.0 m.

$$\mu = 0.718 a_s + 2.117 L^{-1} + 0.056 \qquad \textbf{(5.10.2)}$$

where

a_s : replacement area ratio

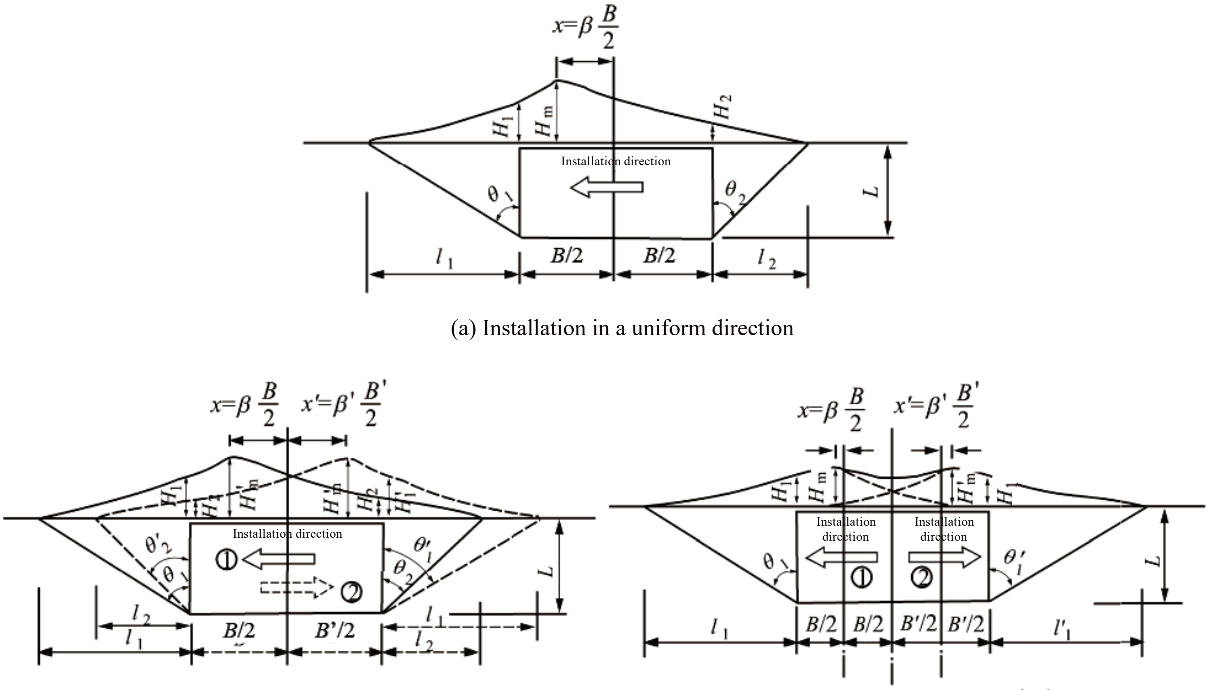
L : average length of the sand piles (m)

μ : heaving ratio

(2) Rough estimate of shapes of heaving

① The shapes of heaving are largely influenced by the construction method of the sand piles (the installation directions and presence or absence of neighboring areas of improvement). The common influence of the installation directions on the shapes of heaving is expressed as displacement of the peak positions in the installation directions from the centers of the areas of improvement. The shape of heaving when sand piles are installed in uniform directions can be expressed as shown in **Fig. 5.10.3** (a) using the maximum heaving height H_{max}, the heaving height at the front edge of the area of improvement H_1, the heaving height at the rear edge H_2, the distance between the point of the maximum heaving height and the center of the area of improvement x, the area of front heaving l_1, and the area of rear heaving l_2. Here, the shapes of heaving can also be expressed by sets of nondimensional values combining α_1 or α_2 which is a ratio of H_1 or H_2 to H_{max}, β which is a ratio of x to $B/2$ and θ_1 and θ_2 which are the angles of upward dispersion of the heaving areas. Judging from the results of the field test and field tests, the target set of nondimensional values is $\theta_1 = 60°$, $\theta_2 = 45°$, $\alpha_1 = 0.85$, $\alpha_2 = 0.4$ and $\beta = 0.7$.

② The sand piles are generally installed in uniform directions from one side to the other of the cross sections (as shown in **Fig. 5.10.3(a)**), in two alternating directions (as shown in **Fig. 5.10.3(b)**), and in two directions from the center to both sides (as shown in **Fig. 5.10. 3(c)**). Assuming that both the installation in two alternating directions and two directions from the center to both sides can be considered a superimposition of the installation in uniform directions, the shapes of heaving can be expressed by combining the coefficient related to the shapes of heaving described above and the prediction results of the maximum heaving height H_{max}, which is described later. For the influence of the presence or absence of neighboring existing areas of improvement on the shapes of heaving, refer to the survey results[186)].



(a) Installation in a uniform direction

(b) Two alternating directions

(c) Two directions from the center to both sides

Fig. 5.10.3 Shapes of Heaving by Installation Direction

(3) Heaving heights

There are two methods for predicting heaving heights: one is to estimate the heaving heights for the respective points or the average heaving height in a manner that combines the estimated heaving ratios by the estimation equation described in (1) and the shapes of heaving described in (2), and the other is to estimate by deriving a statistical estimation equation directly from the performance records as with the heaving ratio.

(4) Evaluation of the strength of cohesive soil

In the shapes of heaving described in (1) to (3) above, the cohesive soil inside the area surrounded by planes extended from the lower edges of the sand piles to the ground surfaces at angles of upward dispersion θ_1 and θ_2 is considered to be disturbed through the sand pile installation process, and, therefore, undergoes a reduction in strength. The degree of disturbance of the cohesive soil between the sand piles in the area of improvement differs from that of the cohesive soil in other areas. The cohesive soil between the sand piles in the area of improvement deals with a large amount of disturbance but restores its strength quickly because the installed sand piles function as drainage layers. There have been reports on cases of cohesive ground which restored its original strength in 1 to 3 months after the installation of sand piles[188), 189)]. When it is difficult to have sufficient time from the installation of the sand piles to the construction of the superstructures, the performance verification shall be carried out while taking into consideration the reduction in strength of the original ground. **Fig. 5.10.4**[188)] shows an example of a trend of the strength of cohesive soil between the sand piles immediately after the installation of the sand piles. In the figure, q_{u0} is the unconfined compressive strength of the original ground, q_u is the unconfined compressive strength of the cohesive soil between the sand piles after certain periods have lapsed, and x and σ are the average and standard deviation of q_u/q_{u0}, respectively.

As can be seen in **Fig. 5.10.4**, strength reduction rates with an elapsed time of less than one month after sand pile installation are at about a maximum of 50% and about 20% on average of the strength of the original ground. In addition, there have been reports that the cohesive soil outside the area of improvement also had a strength reduction of up to about 50%. Because the surroundings of the areas of improvement are not provided with sand piles which facilitate a drainage function, the surroundings of the areas of improvement have slower rates of strength restoration than the areas of improvement. For the restoration of the strength of the areas of improvement after sand pile installation, refer to the performance records[190), 191)] and reports on laboratory tests[190)].

When superstructures are constructed in phases, the performance verification can consider the increases in strength due to the consolidation of cohesive soil between the sand piles. However, effective consolidation loads applied to the cohesive soil between sand piles shall be determined using the stress reduction coefficient to be described later.

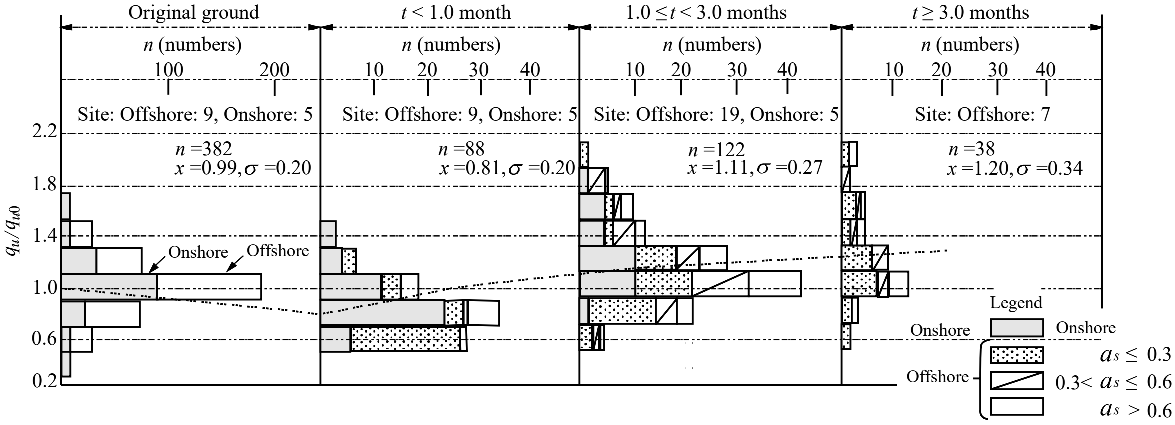


Fig. 5.10.4 Disturbance and Restoration of Cohesive Soil in Areas of Improvement (between Sand Piles)[188)]

(5) Evaluation of the properties and strength of heaved soil

Although heaved soil has been removed in many cases, there have been an increasing number of examples of utilizing heaved soil as part of the foundation ground. In such cases, because there may be a possibility that the utilization of heaved soil enables the implementation of the SCP method to be economical with the excavation volume reduced, the properties and strength of heaved soil need to be evaluated.

In one example of the utilization of heaved soil after sand pile installation, heaved soil generated as a result of installing sand piles with a replacement area ratio of 70% was improved using the same construction machines without compaction (as large diameter sand drains) to a level with the improvement area ratio of 40% (1.7 m square arrangement of ϕ 1.2 m piles)[192)] . As a result, large diameter loose sand piles were constructed in heaved soil with an average *N*-value of 3.6 and heaving heights in the area of improvement in the range of 3 to 4 m. According to the test results of heaved soil immediately after the installation of sand piles, it was confirmed that there was almost no difference in the physical properties (unit weights, water contents and grain size distribution) between the improved heaved soil and the original soil at the depth corresponding to the heaved soil.

Table 5.10.1[193)] shows the results of a comparison between the unconfined compressive strength q_u of heaved soil and that of unconfined compressive strength q_{u0} of the original soil before improvement. In the table, the heaved soil outside the areas of improvement is classified into two types depending on whether the soil exists outside the lines extended from the lower ends of the sand compaction piles at angles of upward dispersion of 45 or 60°. It was reported that the heaved soil in the areas of improvement showed a 50% reduction in strength when the sand piles were installed and restored its strength in 1.5 to 3.5 months. In contrast, the heaved soil outside the area of improvement showed a 30 to 40% reduction in strength and restored its original strength slowly in as long as 8 months.

For the final shapes and properties of heaved soil subjected to compaction, refer to the report by Fukute et al[186)].

Table 5.10.1 Reduction and Restoration of the Strength of Heaved Soil[193)]

		Before construction	Immediately after construction	1.5-3.5 months after construction
q_u/q_{u0}	In improved area	1. 00	0. 46	0. 93
	Outside improved area (45º)	1. 00	0. 62	0. 65
	Outside improved area (60º)	1. 00	0. 72	0. 72

5.10.4 Calculation Formula for the Shear Strength of Improved Ground

(1) Although several formulae have been proposed for the calculation of the shear strength of improved ground (composite ground comprising sand piles and soft cohesive ground)[171)], the **equation** (**5.10.3**) has been used in many cases regardless of the replacement area ratios (refer to **Fig. 5.10.5**). In the equation, the first term has often been ignored in the case of $a_s \geq 0.7$. In addition, there have been cases of evaluating the improved areas as uniform sandy soil having an angle of shear resistance of $\phi = 30°$ without using the **equation** (**5.10.3**).

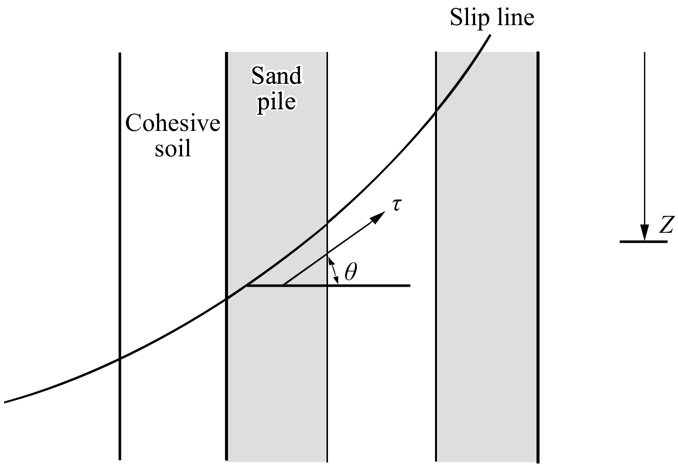


Fig. 5.10.5 Shear Strength of Composite Ground

$$\tau = (1 - a_s)(c_0 + kz + \Delta\sigma_z \mu_c (\Delta c/\Delta p)U) + (w_s z + \mu_s \Delta\sigma_z) a_s \tan\phi_s \cos^2\theta \tag{5.10.3}$$

where

a_s : replacement area ratio of the sand pile (cross-sectional area of a single sand pile/the effective cross-sectional area affected by a single sand pile)

c_0 : undrained shear strength of the original ground when z = 0 (kN/m^2)

c_0+kz : undrained shear strength of the original soil (kN/m^2)

k : strength increase rate of the original ground in the depth direction (kN/m^3)

n : stress sharing ratio = (n=$\Delta\sigma_s/\Delta\sigma_c$)

U : average consolidation degree

z : vertical coordinate (m)

τ : average shear strength on a slip surface (kN/m^2)

μ_s : coefficient of stress concentration on the sand pile ($\mu_s = \Delta\sigma_s/\Delta\sigma_z = n/\{1 + (n-1)\, a_s\}$)

μ_c : coefficient of stress reduction in the cohesive soil ($\mu_s = \Delta\sigma_c/\Delta\sigma_z = 1/\{1 + (n-1)\, a_s\}$)

w_s : unit weight of the sand pile (submerged unit weight when submerged) (kN/m^3)

ϕ_s : angle of shear strength of the sand pile (°)

θ : angle between the slip surface and the horizontal plane (°)

$\Delta\sigma_z$: average of the increases in vertical stress due to an action at a point on the object slip surface (kN/m^2)

$\Delta\sigma_s$: average of the increases in vertical stress due to an action at a sand pile on the object slip surface (kN/m^2)

$\Delta\sigma_c$: increase in vertical stress due to an action on the cohesive soil between the sand piles on the object slip surface (kN/m^2)

$\Delta c/\Delta p$: strength increase rate of the original ground

(2) Constants used in performance verification

In past performance verifications, the **equation** (**5.10.3**) has been used with a range of constants. Thus, the values of the constants to be used in the performance verification shall be determined by taking into consideration the strength of the original ground, the margin of safety to be applied, the performance verification methods to be applied (refer to **C Part III, Chapter 2, 5.10.6 Performance Verification**), and the construction speeds. The standard stress sharing ratio and angle of shear resistance obtained through an inverse analysis of the **equation** (**5.10.3**) using data from the performance records are shown below[194].

$a_s \leq 0.4$	$n = 3$	$\phi_s = 30°$
$0.4 \leq a_s \leq 0.7$	$n = 2$	$\phi_s = 30°$ to $35°$
$a_s \geq 0.7$	$n = 1$	$\phi_s = 35°$

In recent years, there has been an increasing number of cases of using slag as a material for sand piles. There are types of slag which show relatively large internal friction angle, and when using such slag, the performance verification can be carried out with the internal friction angle close to the actual values by paying particular attention to the setting of the stress share ratios.

(3) Types of equations for calculating the shear strength of composite ground

In past performance verifications, the following three equations have also been used in addition to the **equation** (**5.10.3**).[194] The **equations** (**5.10.5**) and (**5.10.6**) are proposed for calculating shear strength with high replacement area ratios. According to past survey results, almost every past performance verification has used the **equation** (**5.10.3**) and very few cases have used the **equation** (**5.10.4**) for calculating shear strength with low replacement area ratios ($a_s \leq 0.4$). The majority of past performance verifications for replacement area ratios in the range of $0.4 \leq a_s \leq 0.6$ have used the **equation** (**5.10.3**), and about 1/5 of the cases have used the **equation** (**5.10.5**). For $0.6 < a_s$, the **equations** (**5.10.5**) and (**5.10.6**) have been used in many cases.

$$\tau = (1 - a_s)(c_0 + kz) + (w_m z + \Delta\sigma_z)\mu_s a_s \tan\phi_s \cos^2\theta \quad \textbf{(5.10.4)}$$

$$\tau = (w_m z + \Delta\sigma_z)\tan\phi_m \cos^2\theta \quad \textbf{(5.10.5)}$$

$$\tau = (w_m z + \Delta\sigma_z)\mu_s a_s \tan\phi_s \cos^2\theta \quad \textbf{(5.10.6)}$$

Here, definitions of the symbols used in the above three equations, but not in the **equation** (**5.10.3**), are as follows:

w_m : average unit weight ($w_m = w_s a_s + w_c(1-a_s)$)

w_c : unit weight of cohesive soil (submerged unit weight in water when submerged) (kN/m^3)

ϕ_m : average angle of shear resistance assuming uniformly improved ground with high replacement area ratios

$\phi_m = \tan^{-1}(\mu_s a_s \tan\phi_s)$

5.10.5 Characteristic Values of the Seismic Coefficient for Verification in the Case of Gravity-Type Quaywalls on Improved Ground

(1) The main body over the ground improved through the SCP method has shown a tendency to have reduced displacement due to the actions of seismic ground motions. Thus, the seismic coefficient for main body over the ground improved through the SCP method can be reasonably set with appropriate evaluation of the reduction effect of the SCP method. For the basic procedures and points of caution when calculating the seismic coefficient for verification, refer to **Reference (Part III), Chapter 1, 1 Details of Seismic Coefficient for Verification.**

The characteristic values of the seismic coefficient for verification in the case of gravity-type quaywalls on ground improved through the SCP method with replacement area ratios of 70% or more can be calculated by applying reduction ratios to the maximum corrected acceleration obtained for unimproved ground as shown in the **equation** (**5.10.7**). The maximum corrected acceleration of unimproved ground can be calculated with reference to **Reference (Part III), Chapter 1, 1 Details of Seismic Coefficient for Verification.** The reduction ratios can be obtained for gravity-type quaywalls based on two-dimensional, nonlinear effective stress analysis results with respect to improved ground having a replacement ratio of 70%.

$$k_{h_k}' = 1.78\left(\frac{D_a}{D_r}\right)^{-0.55}\frac{\alpha_c c}{g} + 0.04 \qquad \textbf{(5.10.7)}$$

where

k_{hk}' : characteristic value of the seismic coefficient for verification

α_c : maximum corrected acceleration (cm/s^2)

g : gravitational acceleration (= 980 cm/s^2)

D_a : allowable displacement (cm) (= 10 cm)

D_r : standard displacement (cm) (= 10 cm)

c : reduction ratio of vibration characteristics due to ground improvement (c = 0.75)

5.10.6 Performance Verification

(1) Examination of slip circle failures

① In the performance verification of ground improved through the SCP method, the modified Fellenius method has been frequently used for the slip circle analyses. For slip circle analyses based on the modified Fellenius method, the ground and superstructures are divided into sliced pieces and the vertical stresses on the slip surfaces are calculated while ignoring the non-static stability force between the sliced pieces. That is, those actions acting on the original ground included in the sliced pieces are assumed to contribute to the vertical stresses on the slip surfaces passing through the sliced pieces (hereinafter, this calculation method is referred to as the “slice method”). However, the loads are distributed in the actual ground to some extent.

There is another method capable of incorporating the effect of this stress distribution in the ground on slip circle failures in a manner that obtains the increases in vertical stresses $\Delta\sigma_z$ at arbitrary positions on slip surfaces using Boussinesq’s solution and applies the increases to the modified Fellenius method (hereinafter, this method is referred to as the “stress distribution method”).

② In the performance verification of ground improved through the SCP method, either the slice method or the stress distribution method can be used. The **equation** (**5.10.8**) can be used for the examination of slip circle failures under a permanent situation. In the equation, the subscript k denotes the characteristic value.

$$m\cdot\frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R \sum_i M_{R_i} \qquad S_d = \gamma_S \sum_i M_{D_i} \qquad \textbf{(5.10.8)}$$

where

γ_R : partial factor multiplied by a resistance term

γ_S : partial factor multiplied by a load term

m : adjustment factor.

$\sum_i M_{R_i}$: sum of the resistant moment (kN•m/m)

$$\sum_i M_{R_i} = \sum rs\overline{\tau}_k \sec\theta$$

r : radius of a slip circle (m)

s : width of a sliced piece (m)

θ : angle between a slip surface and a horizontal plane (°)

$\bar{\tau}$: shear strength of the ground (kN/m^2)

$\sum_i M_{D_i}$: sum of the driving moment (kN•m/m).

In the case of quaywalls: $\sum_i M_{D_i} = \sum\left\{\left(w'_k + q_k + q_{RWL_k}\right)x\right\}$

w' : effective weight of a sliced piece (kN/m)

q : surcharge acting on a sliced piece (kN/m)

q_{RWL_k} : buoyancy acting on a sliced piece due to a higher residual water level behind the facility (RWL) than the water level in front of the facility (LWL) $\rho_w g\ (RWL\text{-}LWL)\ s$ (kN/m)

x : horizontal distance between the gravity center of a sliced piece and the center of a slip circle (m).

In the case of breakwaters: $\sum_i M_{D_i} = \sum\left\{\left(w'_k + q_k\right)x\right\}$

w' : effective weight of a sliced piece (kN/m)

q : distributed load acting on the area of a sliced piece obtained by dividing the effective weight of a breakwater body by its width (kN/m)

θ : angle between the bottom face of a sliced piece and a horizontal plane (°)

For the calculation of the characteristic values in the equation, reference can be made to Part III, Chapter 5, 2.2.3(2) Performance verification for the overall stability of structures under a permanent action situation in respect to self-weight in the case of quaywalls and Part III, Chapter 4, 3.1.4(2) Performance verification for the overall stability of breakwater bodies under a permanent action situation in the case of breakwaters.

The shear strength of the improved ground can be calculated by the **equations** (**5.10.3**) to (**5.10.6**) depending on the design conditions. For example, when using the **equation** (**5.10.3**), the characteristic value of the shear strength of the improved ground can be calculated by the following equation, with $\Delta\sigma_z$ obtained by using Boussinesq's solution. In the equation, the subscript k denotes the characteristic value and the definitions of the symbols are the same as those in the case of the **equation** (**5.10.3**).

$$\bar{\tau}_k = (1 - a_s)\{c'_k + kz + \Delta\sigma_Z \mu_c (\Delta c / \Delta p) U\} + (w_{s_k} z + \mu_s \Delta\sigma_z) a_s \tan\phi_{s_k} \cos^2\theta \qquad \textbf{(5.10.9)}$$

③ **Fig. 5.10.6** shows a schematic diagram of a slip circle failure.

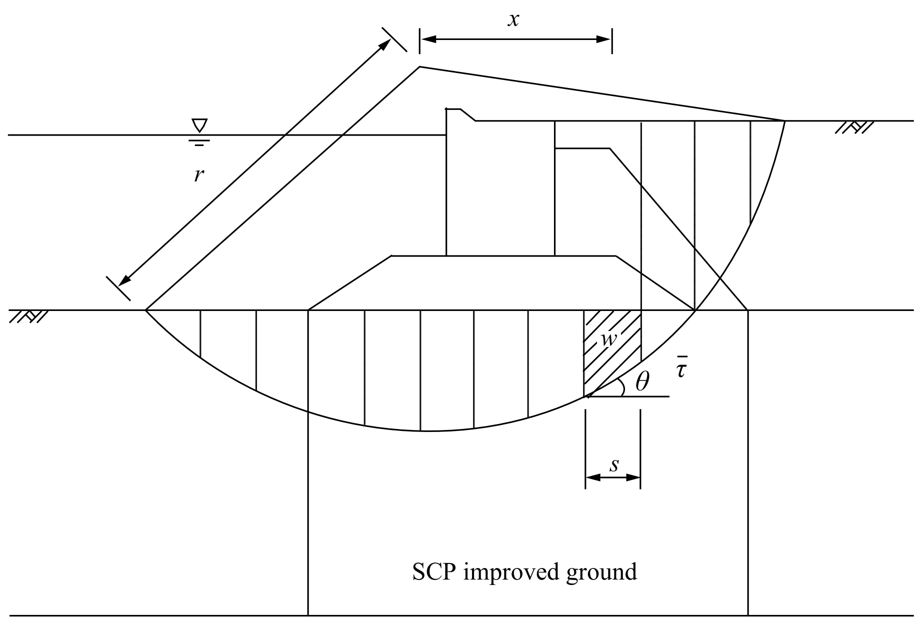


Fig. 5.10.6 Schematic Diagram of a Slip circle Failure

④ When examining the slip circle failures of ground improved through the SCP method with replacement area ratios of 50 to 80% using the **equation** (**5.10.9**), the partial factors can be used in **Table 5.10.2**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience. The partial factors listed in **Table 5.10.2** are set with reference to the safety levels in the past standards[4)] based on the use of the characteristic values of the physical properties of the ground specified in **Part II, Chapter 3, 2.1 Methods for Estimating the Physical Properties of Ground.**

Without using the **equation** (**5.10.9**), the slip circle failures of ground improved through the SCP method can be examined with reference to the partial factors related to the slip failures shown in **Part III, Chapter 2, 4.2.1 Stability Analyses by Slip circle Surfaces.** In addition, the partial factors in **Table 5.10.2** have not been set for use in the examination of slip circle failures with slip surfaces passing through sandy ground below improved ground. In such cases, the examination of slip circle failures shall be additionally examined through other appropriate methods.

Table 5.10.2 Standard Partial Factors

Mode of failure	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Slip circle failure of foundation ground (Revetments and quaywalls)	0.82	1.01	– (1.00)
Slip circle failure of foundation ground (Breakwaters)	0.87	1.02	– (1.00)

⑤ **Points of caution for performance verification**

Because the **equation** (**5.10.3**) is generally used in combination with the stress distribution method, there may be cases where performance verification results greatly vary depending on the selection of factors to be used in the equations for obtaining the shear strength or the selection of such equations. Thus, when selecting the equation to be used in the performance verification and the factors to be used in the equations, it is necessary to give due consideration to examples of combinations of the equations and factors actually used in past designs and construction. In this regard, reference can be made to the **Reference 194)** for the sensitivity of the types of equations for obtaining shear strength, the selection of factors to be used in the performance verification and the combination of stability calculation methods to the safety margin, as well as the evaluation of safety margins through inverse analyses of the performance records of offshore construction.

(2) Examination of consolidation

① **Calculation of consolidation**

The **equation** (**5.10.10**) can be used for the performance verification of settlement amounts.

$$S_a \geq S_f$$
$$S_f = \beta S_{f_0}$$
$$S_{f_0} = m_v(p_0' + \alpha\gamma' h - p_c')H(1-U)$$
$$S_{f_0} = \frac{\Delta e}{1+e_0}H(1-U) \qquad \textbf{(5.10.10)}$$
$$S_{f_0} = \frac{C_c}{1+e_0}H\left(\log_{10}\frac{p_0' + p'}{p_0'}\right)(1-U)$$

where

C_c : compression index

h : height of fill (m)

H : thickness of a consolidation layer (m)

m_v : coefficient of volume compressibility (m^2/kN)

p' : consolidation pressure (kN/m^2)

p_0' : initial pressure (vertical pressure before construction) (kN/m^2)

p_c' : preconsolidation pressure (kN/m^2)

S_a : allowable settlement (m)

U : consolidation degree

e_0 : initial void ratio of the original ground

α : coefficient of stress distribution (a ratio of distributed stress in ground and a consolidation load (fill load))

β : settlement reduction ratio (ratio of the settlement of composite ground to the settlement of unimproved ground)

γ' : submerged unit weight of fill (kN/m^3)

Δe : reduction of the void ratio of the original ground

S_f : settlement of composite ground (m)

S_{f0} : settlement of unimproved ground (m)

② **Comparison between calculated and measured settlement**

The design residual settlement of improved ground can be calculated by multiplying the predicted settlement of unimproved ground by the settlement reduction ratio β as shown in the **equation** (**5.10.10**). The settlement reduction ratios β are generally expressed in a form similar to the stress reduction coefficient μ_c. **Fig. 5.10.7** shows an example of a comparison between the calculated and measured values of the settlement reduction ratios[189)]. The values of β on the vertical axis represent the ratios of the final settlement of improved ground estimated through the hyperbolic approximation using measured settlement to the calculated final settlement of original ground. **Fig. 5.10.7** also shows the settlement reduction ratios when the stress sharing ratios n are 3, 4 and 5, and the settlement reduction ratios ($\beta = 1-a_s$) which have been empirically used in the case of the SCP with high replacement area ratios. As can be seen in the figure, ground improvement has a large effect on the reduction in settlement, the settlement reduction effect is largely affected by the replacement area ratios, and the calculated values based on the stress sharing ratio n of approximately 4 are close to the measured values, although the measured values vary widely.

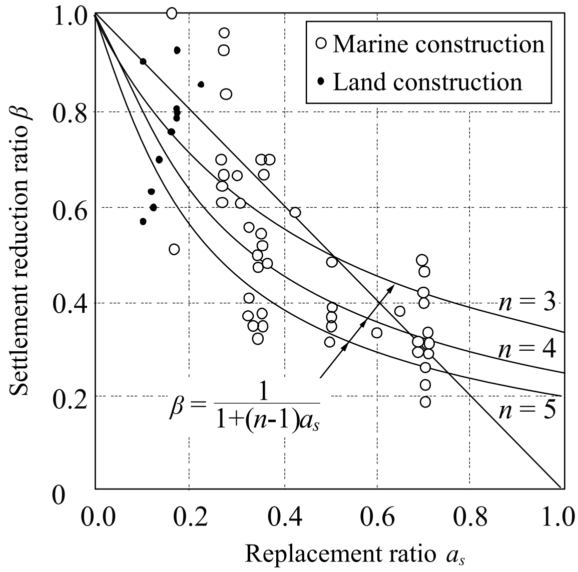


Fig. 5.10.7 Relationship between Settlement Reduction Ratios and Replacement area ratios[189)]

③ **Comparison between calculated and measured consolidation time**

The consolidation degrees of ground improved through the SCP method tend to be lower than the consolidation degrees predicted with Barron's solution. **Fig. 5.10.8** shows the comparison results of settlement rates at different replacement area ratios with the coefficient of consolidation as a major parameter to evaluate the differences in settlement rates based on the performance records[195)]. Here, C_{vp} is the coefficient of consolidation inversely calculated from the measured relationship between the time and settlement, with C_{v0} as the coefficient of consolidation obtained through soil tests. As can be seen in the figure, the tendency of ground improved through the SCP method to be slow in consolidation in comparison with the predicted consolidation becomes more conspicuous with an increase in the replacement area ratios.

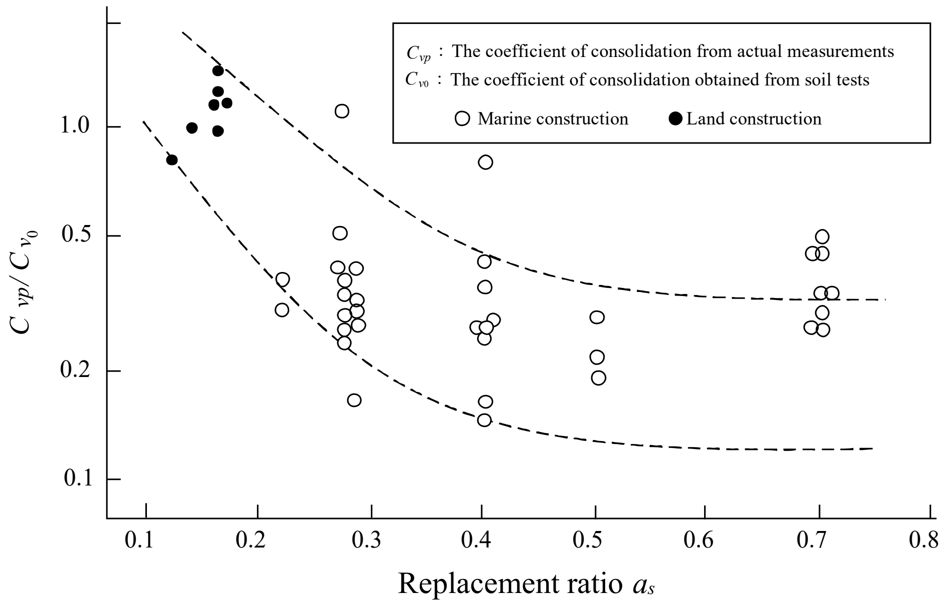


Fig. 5.10.8 Low Consolidation Degrees of Ground Improved through the SCP Method[195)]

④ **Comparison between calculated and measured increases in strength**

Increases in the strength of cohesive soil between sand piles Δc can be calculated by the **equation (5.10.11)**. **Fig. 5.10.9** shows the values of μ_c inversely calculated from the measured increases in the strength of cohesive soil between sand piles[189]. In the figure, the vertical axis represents the ratios (μ_c (= $\Delta c_a/\Delta c_c$)) of the measured increases in the strength of the ground improved with sand compaction piles Δc_a to the predicted values of the increases in the strength of unimproved ground Δc_c (= $\Delta\sigma_z (\Delta c/\Delta p)$ U). The measured increases in strength vary based around the stress sharing ratios n of 3 to 4.

$$\Delta c = \mu_c \, \Delta\sigma_z \, (\Delta c / \Delta p) U \tag{5.10.11}$$

where

μ_c : stress reduction coefficient of cohesive soil ($\mu_c = \Delta\sigma_c / \Delta\sigma_z = 1/\{1+(n-1)a_s\}$)

$\Delta\sigma_z$: average value of the increases in vertical stress due to actions at the object depth (kN/m^2)

$\Delta c/\Delta p$: increase rate of strength of the cohesive soil in the original ground

U : average degree of consolidation

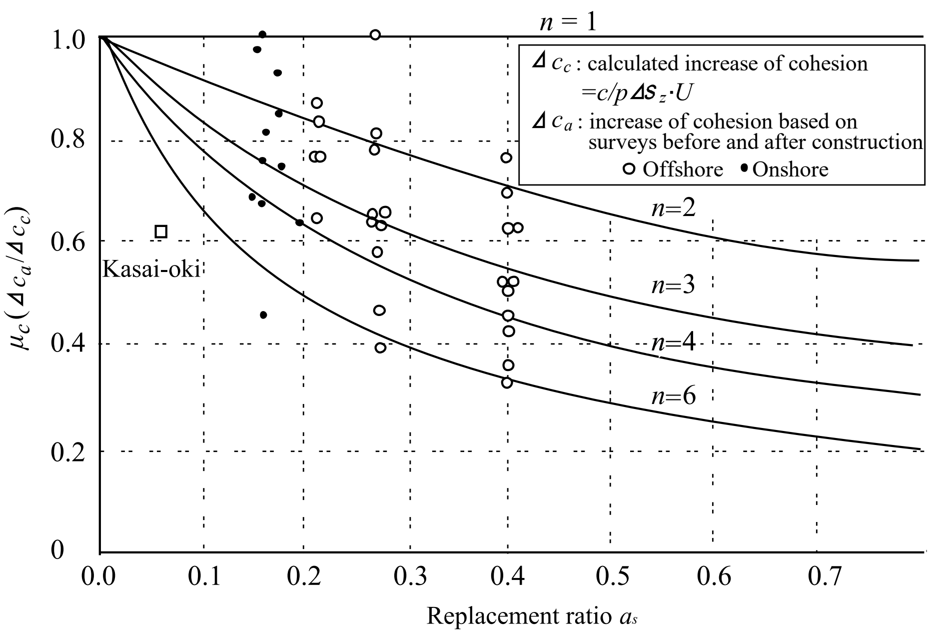


Fig. 5.10.9 Increases in the Strength of Cohesive Soil between Piles in Improved Ground[189]

(3) Performance verification for the T-shaped SCP method

In the SCP method, the cross-sectional shapes of the areas of improvement are generally rectangular. Recently, there have been cases of the modified SCP method, called the T-shaped SCP method, which improves ground so that the areas of improvement have T-shaped cross sections with sections below the flanges left unimproved, as shown in **Fig. 5.10.10**, on the condition that the stability with respect to slip circle failure can be secured[196), 197)] . Because of its capability to reduce the areas of improvement and accelerate construction periods compared to the conventional SCP method, the T-shaped SCP method can be seen as a method that considers economic efficiency. The performance verification of the T-shaped SCP method can be carried out in accordance with that of the conventional SCP method with attention paid to the following items:

① The settlement behavior of both the improved regions, such as the crown sections of structures, and the unimproved regions below the flanges;

② The differences in the examination results of slip circle failures between the T-shaped SCP method with the reduction of the areas of improvement and the conventional SCP method; and

③ The stability with respect to slip circle failures after settlement and deformation.

When implementing the T-shaped SCP method from shallow sections, the succeeding improvement of deep sections may cause the heaving of ground, thereby preventing the design improvement depths from being secured. Thus, it is preferable to implement the T-shaped SCP method in two directions from the centers to both edges.

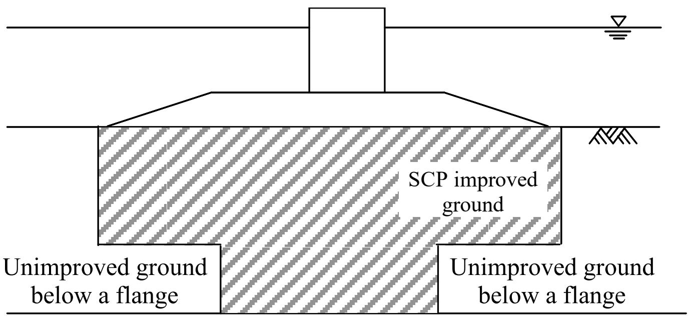


Fig. 5.10.10 T-shaped SCP Method

5.11 Rod Compaction Method

5.11.1 Fundamentals of Performance Verification

(1) For the principles and characteristics of the rod compaction method, refer to the Handbook of Countermeasure against Liquefaction of Reclaimed Land (Revised Edition)[198].

(2) Examples of the variations of the rod compaction method include those using steel pipes, H-section steel and rods with branching protrusions. New variations which have been developed recently include those which can curb noise, vibration and ground deformation, and those which can enhance ground compaction effects[199) . In addition, another variation of the rod compaction method[200] which has already been implemented combines the drainage work around existing structures.

(3) The performance verification of the rod compaction method shall be appropriately carried out with due consideration to the properties of the object ground and the characteristics of the construction procedures based on the performance records or results of field test.

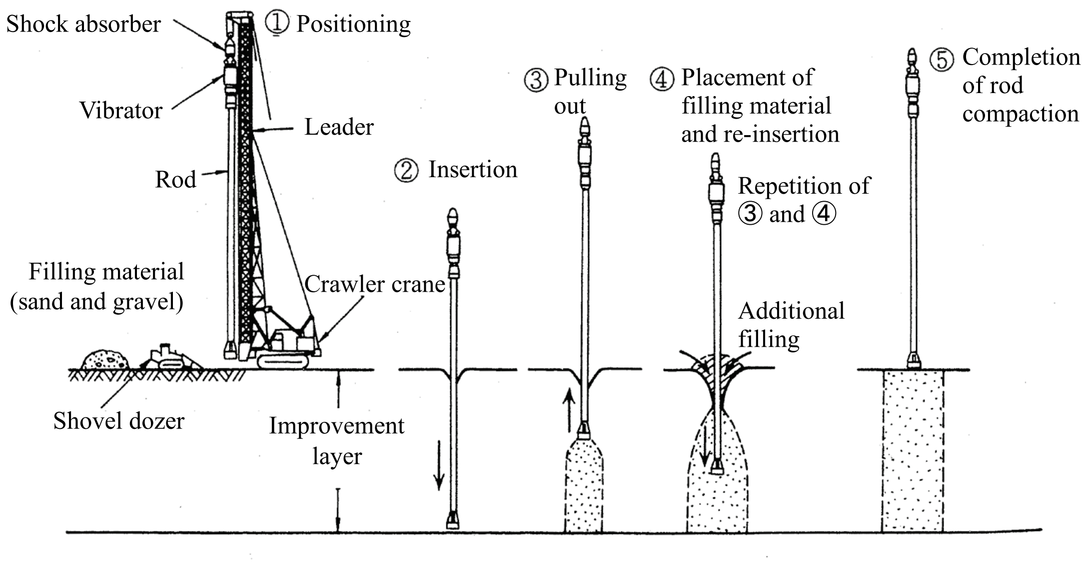


Fig. 5.11.1 Schematic Drawing of the Implementation Procedure of the Rod Compaction Method[201]

(4) The rod compaction method is implemented in a manner that repeats a cycle of the insertion and pulling out of a rod starting from the object improvement depth to the ground surface. The number of cycles, which varies depending on the target degrees of compaction, the constitutions of the soil layers to be improved and the grain size compositions, shall be determined based on the preliminary field test.

(5) When implementing the rod compaction method in close vicinity to the existing structures, particularly sheet pile quaywalls, it is necessary to execute construction management that gives due consideration to the states of the stresses and deformations on the sheet piles, as well as the stresses on the tie rods, during the implementation. In the cases of possible adverse effects on the existing structures, it is preferable to confirm such effects through field test. There are examples of field test implemented to confirm the ground improvement effects of the rod compaction method in the restoration work at the ports damaged by earthquakes. The scope of investigation of the field test includes the effects of vibrations on the sheet pile walls, the drainage effect of gravel piles installed between the areas of improvement through the rod compaction method and sheet piles, and the extent of these drainage effects[202), 203)] .

(6) Points of caution for strength tests of improved ground and implementation of the rod compaction method

Regarding the points of caution for the confirmation of post-construction ground improvement effects and the implementation of the rod compaction method, refer to the Handbook of Countermeasure against Liquefaction of Reclaimed Land (Revised Edition)[198)].

5.11.2 Performance Verification

(1) For the performance verification of the rod compaction method, refer to **Part III, Chapter 2, 5.9 Sand Compaction Pile Method (for the Improvement of Sandy Ground).**

(2) Arrangement and intervals of vibration rods

Because the rod compaction method achieves ground compaction effects only through vibrations, such compaction effects decrease exponentially with distance. Thus, it is preferable to determine the arrangement and intervals of the vibration rods based on the relationship between the intervals of the vibration rods and the *N*-values after ground improvement obtained through performance records and field test. When applying the rod compaction method to existing sheet pile quaywalls, it is necessary to determine the intervals of the vibration rods in the face line directions by giving due consideration to the intervals of the tie rods of existing sheet pile quaywalls.

5.12 Vibro-Flotation Method

5.12.1 Fundamentals of Performance Verification

(1) Characteristics of the implementation method

The vibro-flotation method is to achieve the deep compaction of loose sandy ground in a manner that inserts a rod-like or pile-like vibration body to a predetermined depth in the ground, and pulls up the vibration body while filling the space created around the vibration body with sand or gravel supplied from the ground surface. **Fig. 5.12.1** shows a schematic diagram of the implementation procedure of the vibro-flotation method[204)].

(2) The performance verification of the vibro-flotation method shall be appropriately carried out with due consideration to the properties of the object ground and the characteristics of the construction procedures based on the performance records or results of the field test.

(3) Compaction effect

Because the compaction effect of the vibro-flotation method on loose sand is affected by many factors, it is preferable to execute field test to confirm the compaction effect.

(4) Points of caution when implementing the vibro-flotation method

When improvement object layers include cohesive layers, the vibro-flotation method may not achieve the desired compaction effect in the layers below the cohesive layers because the vibration created using the method cannot produce voids in the cohesive layers that are large enough to allow the filling sand to pass down to the lower layers. In this type of situation, it is necessary to expand the voids such as by providing vibroflots using protrusions[204)] .

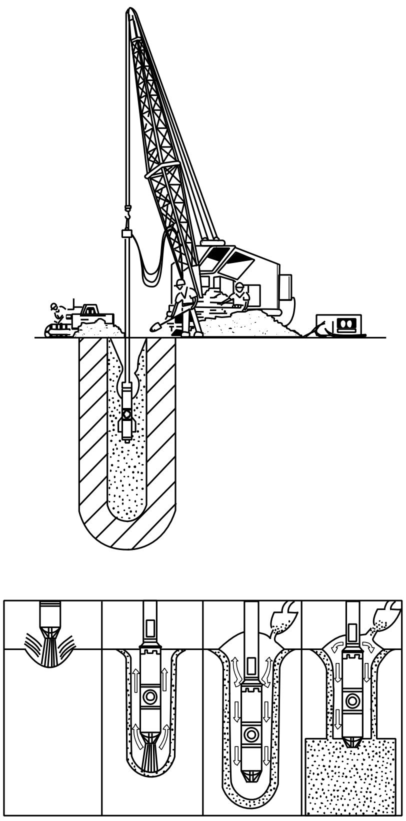


Fig. 5.12.1 Schematic Drawing of the Implementation Procedure of the Vibro-Flotation Method Alteration of 204)

(5) Generally, the basic concept of the performance verification of the vibro-flotation method is almost the same as that of the sand compaction pile method for the improvement of sandy ground, except that the performance verification of the vibro-flotation shall be carried out in consideration of the fine particle contents. Among the several differences in the compaction mechanisms between the vibro-flotation and sand compaction pile methods, the largest difference is that the former fills voids in the ground created by vibration with filling sand, and the latter forcibly presses a predetermined amount of supply sand into the ground while vibrating the casings. Thus, the vibro-flotation method shall be implemented with a particular focus on the variations in the quantities of filling sand per compaction point and the extent of the compaction effect depending on the specifications and capacity of the vibroflots as well as the construction procedures.

5.12.2 Performance Verification

(1) Examinations using performance records

① In cases where there is an availability of sufficiently reliable performance records on the properties of the improvement object ground, the installation density of the vibro-flotation method, the capacity of vibroflots and the correlation between the *N*-values before and after the implementation of the vibro-flotation method, the performance verification of the vibro-flotation method can be carried out based on these performance records.

② According to the performance records of the vibro-flotation method, the applicable limit of the method is estimated as shown in **Fig. 5.12.2**[204)] . This figure has been established based on the actual measurements from 11 cases using the vibro-flotation method implemented with a regular triangle arrangement with pile intervals of 1.2 to 1.5 m, in addition to other similar cases, and can explain the applicable limit of the vibro-flotation method.

③ **Grain size limit of original ground**

The vibro-flotation method is not suitable for silty ground. **Fig. 5.12.2** suggests that although the improvement effect is reduced, the method is still applicable to soil that has content ratios of fine particles smaller than silt of up to 40%. However, there is a report[205)] that soil with fine particle content ratios of 30 to 40% or more cannot expect improvement effects through the vibro-flotation method. In cases of ground improvement in Europe and the United States, there is another report that even powerful vibroflots cannot achieve sufficient improvement effects for soil which has content ratios of fine particles smaller than silt of 25% or more[36)].

④ **Grain size limit of filling sand**

Materials frequently used as filling sand include gravel with grain sizes ranging from 5 to 40 mm, coarse sand, slag and local sand[205)]. Materials with small grain sizes may be suspended in upward mud water flows or have slow falling velocities, thereby preventing the smooth compaction of filling sand. The minimum sizes of grain acceptable as filling sand are indicated by the dashed lines in **Fig. 5.12.2**.

⑤ **Target *N*-values after improvement**

The *N*-values shown in **Fig. 5.12.2** were measured at positions furthest from the vibro-piles after the implementation of the method and indicate the approximate compaction limits under the construction conditions described in ③ and ④ above.

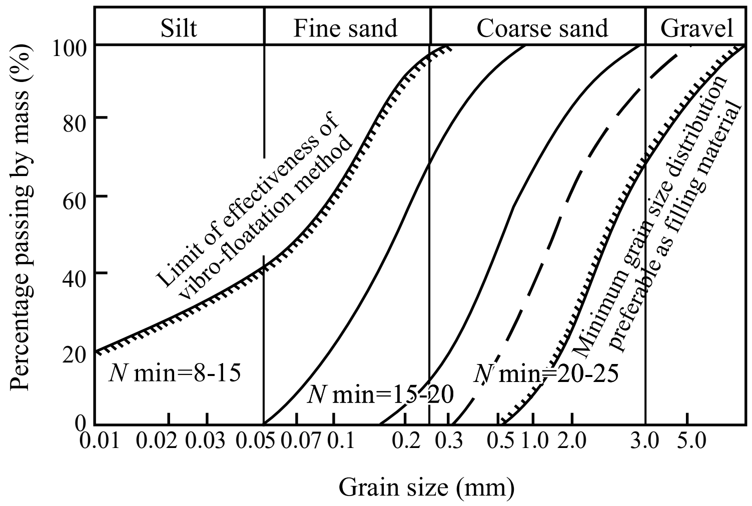


Fig. 5.12.2 Relationship between the Grain Sizes of Original Ground and the Minimum *N*-Values[204)] (in the Case of Sandy Ground Improvement)

(2) Examination through field test

① In cases of the absence of sufficiently reliable performance records, soil containing fine particles smaller than silt, or the presence of alternate layers of sandy and cohesive soil, it is preferable to carry out the performance verification based on field test. In these cases, field test shall be planned through comprehensive evaluation of the results and performance records of the examination of the degrees of compaction in terms of the void ratios.

② The preliminary examination of field test can be carried out in accordance with the **Reference 206)**. When determining the quantity of filling sand and the appropriate installation intervals of vibroflots, it is necessary to refer to previous examples of the vibro-flotation method.

5.13 Drain Method as a Liquefaction Countermeasure

(1) The performance verification of the drain method as a liquefaction countermeasure shall be appropriately carried out with due consideration to the properties of the object ground and the characteristics of the construction procedures based on the performance records or the results of field test.

(2) The drain method as a liquefaction countermeasure alleviates the severity of liquefaction in a manner that enhances the permeability of the ground as a whole with drains made of permeable materials constructed in the ground subjected to liquefaction. Drains are generally constructed in the form of piles but there are ideas to construct wall-shaped drains or drains surrounding the structures. The backfill of quaywalls can be considered as a type of drain when sand invasion prevention sheets are made of permeable materials. The materials generally used for drains are

crushed stones, gravel and other artificial materials such as synthetic resin perforated pipes. As mentioned above, there are a variety of drains used as liquefaction countermeasures.

(3) It is inevitable that the implementation of the drain method be accompanied by certain levels of increase in pore water pressure and settlement.

(4) There is another ground improvement method, called the gravel compaction method, which is a combination of the drain and compaction methods. In principle, the performance verification of the gravel compaction method can be carried out as specified in this section. However, the gravel compaction method shall be implemented with attention paid to clogging of the drains because the dynamic implementation of the method may cause portions of the original ground that are brought into contact with the drains to undergo local liquefaction due to vibration caused during the construction of the drains.

(5) The vibro-flotation method may be implemented in a manner that uses gravel or crushed stones as filling materials installed in the form of piles. In this case, the vibro-flotation method is common to the gravel drain method in that gravel is installed in the form of piles. However, the vibro-flotation method has a high risk of allowing soil around the piles to seep into the gravel during the implementation of the method. Thus, it is appropriate to consider that the vibro-flotation method which uses crushed stones does not fall under the category of the drain method described in this section.

(6) Steel sheet pile quaywalls constructed using the drain method were severely damaged by the Great East Japan Earthquake in 2011. The cause of this severe damage is thought to be not from liquefaction but due to the raising of the water levels in the ground at the back of the quay walls due to seawater flowing into the ground through exhaust basins in the drains when they were inundated by the tsunami[207), 208)] . For using the drain method as a liquefaction countermeasure, exhaust basins have often been used to exhaust air in drainage crushed stone layers above the drains during earthquakes, thereby enhancing the drainage of groundwater to the drainage gravel layers. However, considering the volume of air to be exhausted out of the drainage gravel layers by the inflow of groundwater into the layers, the drain method as a liquefaction countermeasure provided with adequate drainage crushed stone layers can be implemented without the exhaust basins having a risk of destabilizing the ground when hit by a tsunami[209)] .

(7) For the performance verification of the drain method as a liquefaction countermeasure, refer to the **Reference 210)**.

5.14 Well Point Method

(1) The performance verification of the well point method shall be appropriately carried out with due consideration to the properties of the object ground and the performance records.

(2) The well point method can be implemented in combination with the sand drain or prefabricated drain methods for the purpose of increasing effective stresses. However, the well point method has been used mostly for facilitating dry work in a manner that lowers the groundwater levels in sand or sandy silt layers (as shown in **Fig. 5.14.1**)[211)] .

(3) Effects on the surroundings

The well point method, which lowers groundwater levels, may cause the wells and the buildings in the vicinity of the construction sites to become dried up and undergo settlement, respectively. Therefore, the applicability of the well point method shall be determined with due consideration to its effects on the surroundings.

(4) For the basic theory and performance verification of the well point method, refer to the **Reference 211)**.

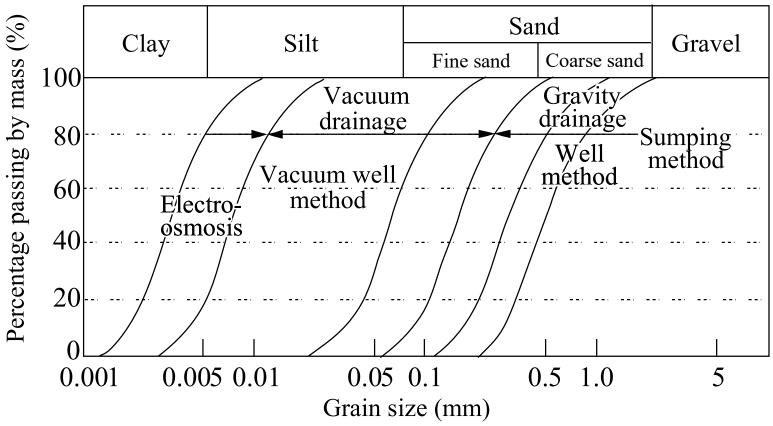


Fig. 5.14.1 Applicability of the Well Point Method by Soil Grain Size[211)]

5.15 Stabilization Method for Shallow Ground

(1) The selection and performance verification of the stabilization method for shallow ground shall be appropriately carried out with due consideration to the properties of the object ground and the performance records.

(2) Variations and characteristics of the stabilization method for shallow ground

The stabilization method for shallow ground has been implemented for securing the workability of construction equipment and increasing the bearing capacity of the surface layers in preparation for full-scale improvement of ground filled with soft or very soft cohesive soil. For reclaimed areas near residential districts, the shallow mixing method has often been implemented for the prevention of offensive odors, the elimination of puddles that act as sources of diseases and pests, and the containment of hazardous industrial waste[212)].

(3) The main variations of the stabilization method for soft shallow ground are as follows.

① **Spreading method**

The spreading method is an old method and the most common stabilization method for shallow ground which sequentially spreads sand or mountain soil. The spreading mechanisms and thicknesses vary but spread sand or mountain soil sinks into the surface layers of very soft ground to a greater or lesser extent and pushes the sludge of soft cohesive soil frontward. Thus, the spreading method has often been associated with difficulties in disposing of the sludge that becomes concentrates in the final corners of construction sites. In addition, the degrees of sand or mountain soil sinking into the surface layers vary widely depending on the location, thereby causing uneven settlement. Thus, there has been an increasing number of cases of the spreading method which use lightweight spreading materials so as to lessen the degrees of sand or mountain soil sinking into the surface layers.

② **Surface shielding method**

The surface shielding method is a type of physical stabilization method for shallow ground which preliminarily covers the surface layers of very soft ground with sheets, rope nets or bamboo nets so as to alleviate the spread sand from locally sinking into the surface layers. The surface shielding method can reduce the amounts and variations of sand sinking into the surface layers in a manner that enables the tensile force of the sheets to support the vertical loads of the spread sand. When using materials with low stiffness such as sheets or rope nets, it is necessary to rigidly fix the edges of these materials. When using materials with high stiffness, the surface shielding method can be implemented without paying particular attention to the fixation of the material edges.

③ **Shallow mixing stabilization method**

The shallow mixing stabilization method is to solidify the surface layers through chemical stabilization actions such as the pozzolanic reaction with soft soil of the surface layers mixed with chemical binders such as lime and cement. There are several types of binders and mixing mechanisms. Recently, the surface soil mixing-type stabilization method has often been implemented, not for the entire treatment of soft ground, but for local treatment as a measure to fix the edges of sheet materials as described in ② above and as partition weirs (stabilization slabs).

④ **Drying and drainage method**

The drying and drainage method is to naturally dry the surface layers of very soft cohesive ground so as to enable heavy equipment to be used directly on the dried surface layers. Because natural drying takes a long time, the drying and drainage method has often been combined with forced drainage, underdrainage or capillary drying to accelerate the drying time. However, the number of cases of actual implementation of the method is relatively few.

5.16 Chemical Grouting Method as a Liquefaction Countermeasure

5.16.1 Fundamentals of Performance Verification

(1) Scope of application

① This section describes the performance verification of ground improved through the chemical grouting method for liquefaction countermeasures. The specifications of grouting materials shall conform to the provisions in **Part II, Chapter 11, 8.5 Grouting Materials**. The variations of the chemical grouting method for a liquefaction countermeasure which have been developed include the permeable grouting, multi-points grouting and grouting methods, such as in the **References 44)**, **213)** and **214)**. This section describes the chemical grouting method as a liquefaction countermeasure with reference to the **Reference 214)**. When implementing other chemical grouting methods irrelevant to the **Reference 214)**, it is necessary to implement them with reference to the manuals of the respective methods with due consideration to the performance records as liquefaction countermeasures (or data obtained through verification tests if no performance records are available) as well as the items described in this section and in the **Reference 214)**.

② The definitions of the terms used in this section are as follows:

(a) Chemical: Water glass-based chemical solutions with degradation components removed to enhance durability;

(b) Activated silica: Solution-type activated silica grout with degradation components removed through ion-exchange resin (film) to enhance durability;

(c) Non-alkaline silica sol: Solution-type activated silica grout with alkaline degradation components removed through neutralization with acid to enhance durability;

(d) Permeation grouting: Injection of chemicals into the voids among soil particles without changing the structures of the soil particles in the ground;

(e) Gelling time: Time from when chemicals are mixed to when the chemicals lose fluidity with an increase in viscosity;

(f) Improvement area ratio: A ratio of the net improved volume to the entire volume of the improvement object ground expressed as a percentage;

(g) Target strength of the mix proportion test: The target value of the unconfined compressive strength in laboratory mix tests using soil sampled from original ground;

(h) Average field strength: The average of the field unconfined compressive strength of improved ground;

(i) Standard design strength: The target value of the field unconfined compressive strength used for performance verification;

(j) Strength ratio: A ratio of the average field strength to the target strength of mix proportion tests;

(k) Field overdesign factor: The overdesign factor of the standard design strength that takes into consideration the variations in the field strength of improved soil;

(l) Grout diameter: The diameter of an improved body assuming that the predetermined amount of chemicals spherically permeates the ground when injected into it;

(m) Grouting velocity: The amount of grout injected into a borehole per minute;

(n) Limit grouting velocity: The maximum grout speed at which the state of permeation grouting can be maintained without the occurrence of fracturing grout in the ground;

(o) Grout ratio: A ratio of the volume of grout chemicals to the net volume of the improvement body; and

(p) Grouting interval: The intervals of grout pipes installed in the ground.

③ While the construction procedure is almost identical to the general grouting method, as applying for liquefaction countermeasure, permanent chemical with longer gelling time is specially used together with special injection ports and injection methods in consideration of the necessity to improve wide areas.

④ According to the performance records, the chemical grouting method as a liquefaction countermeasure is generally applicable to ground with fine particle contents of about 40% or less.

⑤ For other items related to the performance verification and implementation of the chemical grouting method as a liquefaction countermeasure not described in this section, refer to the **Reference 214)**.

(2) Basic concepts

① In the performance verification, it is necessary to appropriately determine the required strength of the improved soil, the mix proportion of chemicals and the areas of improvement.

② In the examination of the stability of the ground with respect to slip circle failures and others, the improved soil shall be evaluated as c and c-ϕ materials so as to obtain examination results that are on the safe side.

③ The areas of improvement shall be determined based on the examination of the stability of the improved ground with respect to sliding failure with structures above the improved ground integrally considered as rigid bodies because, when the surrounding unimproved ground undergoes liquefaction in the event of an earthquake, the improved ground is expected to have significantly larger rigidity than the unimproved ground and behaves as a rigid body.

④ It is preferable to determine the characteristic value of the standard design strength of the improved ground and the areas of improvement in accordance with the procedures shown in **Fig. 5.16.1**.

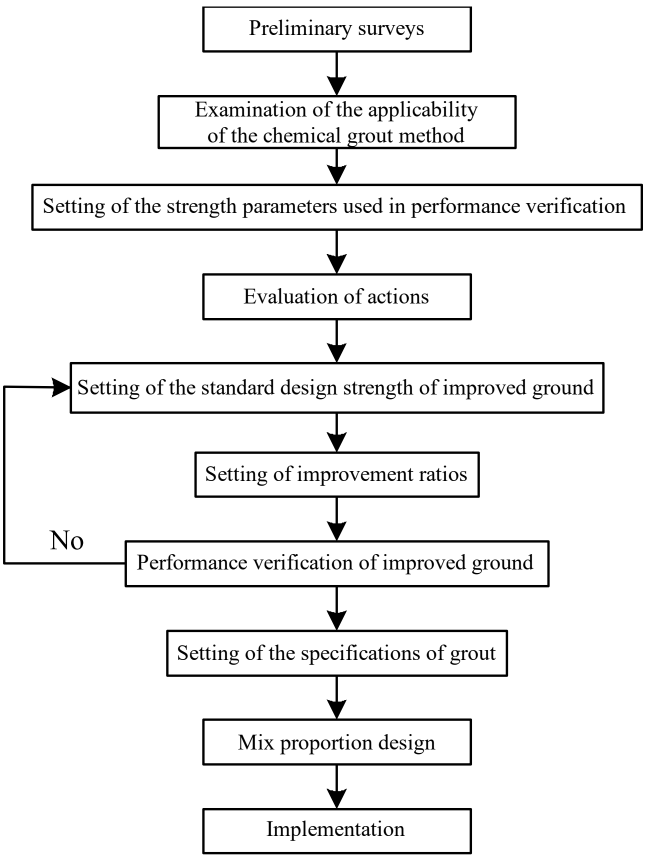


Fig. 5.16.1 Procedures of Performance Verification

⑤ It is necessary to determine the mix proportion of chemicals so as to obtain ground which does not liquefy.

⑥ The followings are the reasons for examining the stability of improved soil in two cases where the improved soil is considered as c and c-ϕ materials. The improved soil is considered to undergo undrained shear deformation because the coefficient of permeability of the improved soil are significantly reduced with voids among the particles of the original sandy soil filled with gel, thereby degrading the drainage performance of the original sandy soil before improvement. Thus, the examination of stability needs to consider the undrained shear strength of improved soil as c materials. However, when subjected to shear force in laboratory element tests, improved soil produces large negative pore water pressure which may cause the improved soil to show larger shear strength than drained shear strength. The prerequisite for the improved soil to reproduce such large shear strength is field conditions with sufficiently large groundwater pressure which can prevent cavitation. In addition, there are reports that the coefficient of permeability of the soil improved through the chemical grouting method are increased with the progress of shear. Thus, it has been decided to adopt the results of stability examinations using either undrained shear strength or drained shear strength, whichever is safer. Because actual cohesion is added to the soil improved with chemicals, the undrained shear strength is evaluated with the improved soil considered as c-ϕ materials. That is, the shear strength of improved soil can be expressed by the following **equations** (**5.16.1**) and (**5.16.2**).

$$\tau_f = c_u \tag{5.16.1}$$

$$\tau_f = c + \sigma' \tan\phi \tag{5.16.2}$$

where

τ_f : shear strength of improved soil (kN/m^2)

c_u : undrained shear strength (kN/m^2)

σ' : effective confining pressure (kN/m^2)

c : cohesion (kN/m^2)

ϕ : angle of shear resistance (°)

In addition, in the equations above, c_u is undrained shear strength obtained through consolidated and undrained triaxial compressive tests at field consolidation pressure, and c and ϕ correspond to cohesion c_d and the angle of shear resistance ϕ_d, respectively, which are obtained through consolidated and drained triaxial compressive tests.

⑦ The calculation of the earth pressure of improved ground acting on wall surfaces with improved soil considered as c-ϕ materials shall be carried out through the method specified in **Part III, Chapter 2, 5.18 Active Earth Pressure of Soils Treated with Hardeners.**

⑧ The chemicals for the chemical grouting method have been practically limited to water glass by the provisional guidelines issued by the then Ministry of Construction. However, it shall be noted that some chemicals conforming to the guidelines may degrade durability due to the eluviation of silica. The eluviation is explained to be caused by unreacted water glass left in the gel and sodium ion. There has been development of highly durable chemicals that no longer contain the substances responsible for eluviation and it is preferable to use these newly developed solution-type chemicals. It is also necessary to select the chemicals in consideration of the effects on environments including those in groundwater and marine water during the implementation of the method.

5.16.2 Preliminary Surveys

(1) It is necessary to appropriately evaluate the properties of the object soil for improvement through preliminary surveys and tests.

(2) The types of preliminary surveys and tests include standard penetration tests, soil particle density tests, soil water content tests, soil grain size tests, maximum and minimum density tests, pH tests, silica content tests, calcium content tests, soil consolidated and undrained triaxial tests, consolidated and drained triaxial tests and repeated triaxial tests.

5.16.3 Examination of the Applicability of the Chemical grouting method

It is necessary to determine the applicability of the chemical grouting method based on the preliminary survey results. One of the most important determining factors is the fine particle contents. Generally, the chemical grouting method cannot be applied to soil having fine particle contents of 40% or more. For soil having fine particle contents of 25 to 40%, the chemical grouting method needs to be implemented with particular attention to the increasing inhomogeneous nature of the improved soil. It shall also be noted that the homogeneity of improved ground and gelling time vary in the cases of ground with areas of improvement consisting of alternate layers of sandy and cohesive soil, containing shells or gravel, and having rapid groundwater flows.

5.16.4 Setting of the Strength Parameters used in Performance Verification

The strength parameters used in the performance verification are unconfined compressive strength, liquefaction strength, drained shear strength (ϕ_d and c_d), and the undrained shear strength (C_u) of improved soil. It is preferable to obtain the values of these strength parameters through laboratory mix tests with the density of test samples taken from construction sites adjusted to the field density, and the values of the liquefaction strength as well as cohesion c_d and C_u in relation to the unconfined compressive strength.

5.16.5 Actions

The main actions to be considered in the performance verification shall be surcharge, the self-weight of improved ground, buoyancy, earth pressure, residual water pressure, fender reaction force, actions due to seismic ground motions, waves, and so on.

5.16.6 Setting of the Standard Design Strength

There is a significant relationship between the liquefaction strength and unconfined compressive strength of improved soil. Thus, the characteristic values of the standard design strength (with unconfined compressive strength as an index) are preferably set so as to enable the liquefaction strength to exceed actions, and are based on the relationship with the unconfined compressive strength described in **Part III, Chapter 2, 5.16.4 Setting of Strength Parameters used in Performance Verification** so as to achieve the required improvement effects while taking into consideration the applicable scope and conditions.

5.16.7 Setting of Improvement area ratios

In principle, the improvement area ratios shall be set at 100%, which means that entire improvement area shall be subjected to the chemical grouting method. When reducing the improvement area ratios, careful measures such as model tests shall be taken to confirm that the reduced improvement area ratios do not cause serious settlement and deformation on structures.

5.16.8 Performance Verification of Improved Ground

(1) The areas of improvement shall be appropriately determined through examinations of the stability of the object facilities and entire ground while taking into consideration the structural types of the facilities and actions.

(2) The actions and resistance force acting on the facilities and improvement object ground to be considered in cases with and without the liquefaction of unimproved ground behind the improved ground shall be set in accordance with **Figs. 5.8.2** and **5.8.3** in **Part III, Chapter 2, 5.8 Premix Method**.

(3) It is necessary to examine the stability of the improved ground including the object facilities with respect to sliding failures during the actions of seismic ground motions. It is also necessary to examine the stability of the improved ground and facilities as a whole with respect to slip circle failures under a permanent action situation. When examining the stability, the methods specified in **Part III, Chapter 2, 5.8.6 (5) ① Examination of sliding failures during the actions of seismic ground motions** and **Part III, Chapter 2, 5.8.6 (5) ② Examination of stability with respect to slip circle failures under a permanent situation** can be used as references.

(4) When the stability of the structures cannot be secured, it is necessary to change the areas of improvement or increase the standard design strength by returning to the procedure specified in **Part III, Chapter 2, 5.16.6 Setting of the Standard Design Strength**.

5.16.9 Setting of the Specifications of Grouting

The following three items related to the soil and construction conditions shall be set as the specifications of grouting.

(1) Grouting ratios (the ratios of the volume of chemicals to the entire volume of the improvement bodies) can be calculated by the **equation** (**5.16.3**).

$$\lambda = \alpha \frac{n}{100} \tag{5.16.3}$$

where

λ : grouting ratio (%)

n : porosity of the ground (%)

α : void filling ratio (a volume ratio of chemicals to voids in the soil) (%)

(2) The grouting velocities and grouting pressure are preferably set by taking into consideration the properties of the object ground, overburden, groundwater pressure and the effects on neighboring facilities. These values have often been set based on the results of limit grouting velocity tests in previous performance records.

(3) Next, the diameters of improvement bodies and grouting intervals shall be determined. According to the **Reference 213)**, the diameters of improvement bodies can be determined based on the grouting ratios, grouting velocities and grouting work time using the **equation** (**5.16.4**).

$$D = 2\left(Q_p t \Big/ \left(\frac{4}{3}\pi \frac{\lambda}{100} 1000 \right) \right)^{\frac{1}{3}} \tag{5.16.4}$$

where

D : grouting diameter (m)

Q_p : grouting velocity (L/min)

t : grouting work time per improvement body (min)

λ : grouting ratio (%)

According to the **Reference 213)**, the grouting intervals can be obtained by the **equation** (**5.16.5**) as the lengths of the sides of cubic improved bodies equivalent to spherical improvement bodies.

$$L = D\left(\frac{\pi}{6a} \right)^{\frac{1}{3}} \tag{5.16.5}$$

where

L : grouting interval (m)

D : grouting diameter (m)

a : improvement area ratio

5.16.10 Mix Proportion Design

(1) The mix proportion design of chemicals, such as the types and concentration of the chemicals as well as the additive amounts of reaction materials, shall be determined so as to fulfill the target gelling time and mix proportion strength through the appropriate laboratory mix tests.

(2) The relationship between the strength of improved soil and the concentration of chemicals is largely affected by the types of soil and the conditions of the density tests and others. Thus, it is necessary to set the laboratory mix test conditions as close to the field conditions as possible.

(3) The gelling time can be set by taking into consideration the grouting work time used in **Part III, Chapter 2, 5.16.9 Setting of the Specifications of Grout**. The target mix proportion strength can be calculated by the **equation** (**5.16.6**) using the characteristic values of the standard design strength as specified in **Part III, Chapter 2, 5.16.6 Setting of the Standard Design Strength**.

$$q_{uL} = \frac{q_{uc_k}}{\kappa}\eta \tag{5.16.6}$$

where

q_{uL} : target mix proportion strength (kN/m^2)

q_{uc_k} : standard design strength (kN/m^2)

κ : strength ratio

η : field overdesign factor

In addition, κ is for correcting the difference between the laboratory and field strength, and η is for incorporating the effect of inhomogeneity of the field soil into the calculation.

5.17 Pneumatic Flow Mixing Method

5.17.1 Fundamentals of Performance Verification

(1) The pneumatic flow mixing method is to produce stabilized soil by mixing binders with the object soil for improvement such as dredged soil using the turbulence effect of plug flows generated inside pressure pipes while pneumatically transporting the object soil for improvement and placing the stabilized soil at predetermined locations. For the principles and characteristics of the pneumatic flow mixing method, refer to the **Manual on Pneumatic Flow Mixing Technology**[215] .

(2) For the production of stabilized soil, there has been development of several types of methods based on pneumatic transportation and solidifier adding methods[25), 216), 217)] . In addition, for the placement of stabilized soil, there are aerial and underwater placement methods. For the underwater placement method, it is necessary to pay attention to the correct use of tremie pipes so as not to drop stabilized soil directly into the water.

(3) The performance verification of the pneumatic flow mixing method shall be carried out by appropriately setting the required strength of improved ground and the areas of improvement based on the survey and test results on the object soil for improvement and stabilized soil as well as the applicable conditions for the method.

5.17.2 Performance Verification of the Pneumatic Flow Mixing Method

(1) Pneumatically stabilized soil is a soil stabilized with binders. The performance verification of the pneumatically stabilized soil can be carried out using the methods applicable to soil.

(2) The required performance of the pneumatically stabilized soil varies depending on the intended use as shown in **Table 5.17.1**. The pneumatically stabilized soil needs to ensure ground strength that satisfies the standard design strength when used for landfill, earth pressure reduction, seismic reinforcement and surface layer treatment, as well as reduce fluidity as much as possible to maintain steep slopes when used for embankment widening and increase fluidity to enhance self-compacting performance when used for backfilling.

Table 5.17.1 Required Performance of Pneumatically Stabilized Soil

Intended use	Required performance of pneumatically stabilized soil	Remarks
① Landfilling	Ground strength	
② Earth pressure reduction	Ground strength	
③ Seismic reinforcement	Ground strength	
④ Embankment widening	Low fluidity (to maintain steep slopes), ground strength	Water content of soil: Low
⑤ Surface layer treatment	Ground strength	
⑥ Underwater backfilling	High fluidity (to eliminate compaction), ground strength	Water content of soil: High

5.17.3 Mix Proportion Design

(1) The mix proportion design of pneumatically stabilized soil shall be determined so as to ensure the target strength and fluidity when pneumatically transported while taking into consideration the types and additive amounts of binders, adjusted water contents and material ages.

(2) It is preferable to conduct laboratory mix tests for the purpose of determining the fluidity, strength and curing conditions, and obtaining information on the relationships between the additive amounts of binders and unconfined compressive strength, the water-cement ratio (*W/C*) and unconfined compressive strength, and the adjusted water contents and flow values.

5.18 Active Earth Pressure When Using Soils Treated with Binders

5.18.1 General

(1) This section describes the concepts relevant to the performance verification of active earth pressure when using soils treated with binders such as cement as backing and backfilling materials.

The soils introduced in this section are those subjected to artificial stabilization with added binders such as cement and those having self-hardening properties. Those materials currently under development are listed below; recent trends have shown an increase in the number of types of similar materials.

① Premixed soil
② Lightweight treated soil
③ Cement-mixed treated soil other than ① and ② above
④ Coal ash with binders
⑤ Self-hardening coal ash
⑥ Granulated blast furnace slag used according to its stabilization characteristics

5.18.2 Active Earth Pressure

(1) Outline

① The active earth pressure of soils treated with binders acting on structures shall be appropriately calculated by taking into consideration the characteristics of the materials to be used and seismic ground motions.

② Generally, active earth pressure during the actions of seismic ground motions can be calculated as static earth pressure based on the seismic coefficient method, provided, however, that seismic response analyses shall be used when it is necessary to examine the earth pressure during earthquakes in detail. The following section describes the method for calculating earth pressure based on the seismic coefficient method while taking into consideration the material characteristics.

③ When stabilized soils in the areas of improvement are determined to have sufficiently large cohesion, the areas of improvement can be generally considered not to undergo liquefaction. Depending on the actions of seismic ground motions, it is generally thought that excess pore water pressure is not generated in the areas of improvement during the actions of seismic ground motions when unconfined compressive strength q_u is around 50 to 100 kN/m^2 or more.

(2) Strength parameters

The methods for setting the strength parameters of soils differ material by material. It is necessary to consider the appropriate cohesion and the internal friction angle in accordance with the characteristics of the soils. Generally, those soils including deep mixing treated soil, lightweight treated soil and stabilized coal ash are considered to be c materials. Premixed soil is considered to have the characteristics of both c and ϕ materials. Granulated blast furnace slag is generally considered to be ϕ material, provided, however, that it can be treated as c material when used with particular focus on its stabilization characteristics.

(3) Calculation of active earth pressure

① Generally, active earth pressure can be calculated in accordance with **Part II, Chapter 4, 2 Earth Pressure**, which means that the concept of earth pressure follows the Mononobe-Okabe theory, which uses Coulomb's earth pressure and enables earth pressure to be calculated from the balance of force acting on spheroidal soil masses causing ground failures.

② Although the earth pressure during earthquakes, particularly underwater earth pressure, has not been fully elucidated, the concept described in **Part II, Chapter 4, 2 Earth Pressure** has been applied to the performance verification of many structures and has achieved satisfactory results.

③ The **equation (5.18.1)** is an earth pressure calculation equation applicable to materials having characteristics of both c and ϕ materials. The equation can be obtained by extending the earth pressure calculation equation in **Part II**, **Chapter 4, 2 Earth Pressure** (refer to **Fig. 5.18.1**).

$$p_{a_i} = \left\{ \frac{\left(\sum \gamma_i h_i\right)\cos(\psi-\beta)}{\cos\psi} + \omega \right\} \frac{\sin(\zeta_i-\phi_i+\theta)\cos(\psi-\zeta_i)}{\cos\theta\cos(\psi-\zeta_i+\phi_i+\delta)\sin(\zeta_i-\beta)} - \frac{c_i\cos(\psi-\beta)\cos\phi_i}{\cos(\psi-\zeta_i+\phi_i+\delta)\sin(\zeta_i-\beta)}$$

$$2\zeta_i = \psi + \phi_i - \mu_i + 90°$$

$$\mu_i = \tan^{-1}\frac{B_i C_i + A_i\sqrt{B_i^{\,2} - A_i^{\,2} + C_i^{\,2}}}{B_i^{\,2} - A_i^{\,2}}$$

$$A_i = \sin(\delta+\beta+\theta) \qquad \textbf{(5.18.1)}$$

$$B_i = \sin(\psi+\phi_i+\delta-\beta)\cos\theta - \sin(\psi-\phi_i+\theta)\cos(\delta+\beta) + \frac{2c_i\cos(\psi-\beta)\cos\phi_i\cos(\delta+\beta)\cos\theta}{\dfrac{\left(\sum\gamma_i h_i\right)\cos(\psi-\beta)}{2\cos\psi}+\omega}$$

$$C = \sin(\psi+\phi+\delta-\beta)\sin\theta + \sin(\psi-\phi+\theta)\sin(\delta+\beta) - \frac{2c\cos(\psi-\beta)\cos\phi\sin(\delta+\beta)\cos\theta}{\dfrac{\left(\sum\gamma h\right)\cos(\psi-\beta)}{2\cos\psi}+\omega}$$

where

p_{a_i} : active earth pressure intensity acting on a wall face at the bottom of the ith layer (kN/m^2)

c_i : cohesion of soil in the ith layer (kN/m^2)

ϕ_i : angle of shear resistance of the ith layer (°)

γ_i : unit weight of the ith layer (kN/m^3)

h_i : thickness of the ith layer (m)

ψ : angle between the wall face and the vertical plane (°)

β : angle between the ground surface and the horizontal plane (°)

δ : friction angle on the wall face (°)

ζ_i : angle between the failure face of the ith layer and the horizontal plane (°)

ω : surcharge per unit area of the ground surface (kN/m^2)

θ : composite seismic angle (°) expressed by $\theta = \tan^{-1}k$ or $\theta = \tan^{-1}k'$

k : seismic intensity

k' : apparent seismic intensity

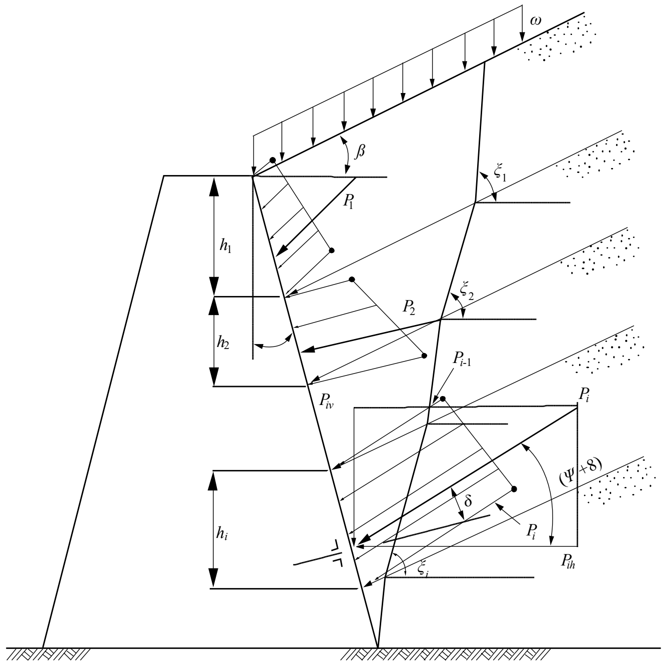


Fig. 5.18.1 Active Earth Pressure Acting on a Structure

④ Okabe's equation[218)] is expanded on in the **equation** (**5.18.1**)[219)] . Although the **equation** (**5.18.1**) does not have the strictness that Okabe's equation has, the solutions from the **equation** (**5.18.1**) when dealing with ϕ materials without the cohesion of soil or c materials without internal friction angle correspond to those of the equation in **Part II, Chapter 4, 2 Earth Pressure**.

⑤ When obtaining the earth pressure intensity and failure angles using the **equation** (**5.18.1**), these values shall be obtained at each boundary of layers with different soil properties on the assumption that the earth pressure intensity and failure line in each layer show a linear distribution. However, there are cases where the earth pressure intensity and failure lines show a curved distribution when these values in identical layers are obtained by dividing the layers, which contradicts the fact that Okabe's equation assumes linear failures based on Coulomb's earth pressure.

⑥ In using the above equation, there may be a necessity to consider the existence of cracks depending on the characteristics of the soils to be used.

(4) Cases of finite improvement widths

① When the equation proposed by Mononobe and Okabe cannot be simply applied to stabilized soils because the areas of improvement are finite, it is necessary to evaluate earth pressure through appropriate methods capable of evaluating the effects of the areas of improvement on earth pressure. In cases of finite areas of improvement, the slice method[220)] can be used for the evaluation of earth pressure. In the slice method, earth pressure can be calculated in a manner that assumes the slip surfaces at the back of the structures, slices earth mass between the slip surfaces and wall faces into pieces, and calculates the earth pressure by balancing self-weight, buoyancy, shear force on sliding surfaces and actions due to seismic ground motions acting on the respective sliced pieces. Although the above method does not always correspond to the modes of ground failures, it can be used when there are no other appropriate methods.

② The characteristics of earth pressure calculation based on the slice method introduced in this section are as follows.

(a) In the case of semi-infinite multilayer ground, the earth pressure calculated using the method almost corresponds to that calculated in accordance with **Part II, Chapter 4, 2 Earth Pressure**.

(b) In the case of finite multilayer ground, the method can calculate earth pressure compatible with the concept of earth pressure based on the Mononobe-Okabe theory.

(c) Following the conventional concept of the angle of wall friction, the method only assumes angles of wall friction of 15° and 0° for ground including ϕ materials and ground comprising c materials, respectively.

(d) There have been no clear definitions of the point of earth pressure on the entire spheroidal soil mass. Thus, the point of earth pressure shall be calculated based on the earth pressure distribution by depth.

(e) There may be cases where the modes of failures when obtaining earth pressure distribution do not correspond to those of entire systems. In such cases, due consideration shall be given to the earth pressure distribution to be used for the performance verification.

(f) In the slice method, the following three modes of failures are examined (**Fig. 5.18.2**).

(g) The earth pressure distribution can be calculated on the assumption that the earth pressure of a certain depth interval is equal to the differences in the earth pressure at the respective depths in the interval.

Mode 1: Failures with uniform slip failure surfaces throughout the backfill (shear resistant mode);

Mode 2: Failures with cracks down to the bottoms of stabilized soil layers (crack mode); and

Mode 3: Failures along the shapes of stabilized soil masses (friction resistance mode).

Note: The type of Mode 1 failures with slip surfaces that do not pass through stabilized soil masses is categorized as Mode 0.

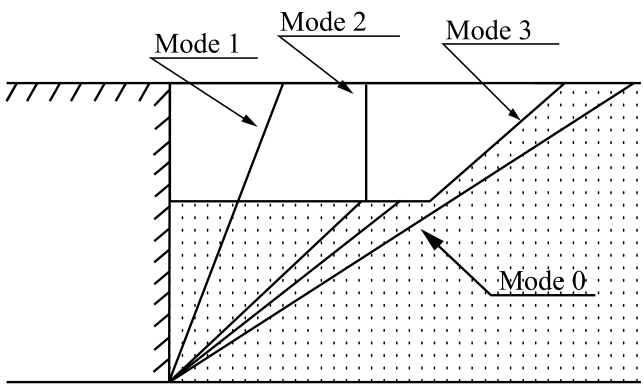


Fig 5.18.2 Three Failure Modes Considered in the Slice Method

5.19 Jet Grouting Method

5.19.1 General

(1) The jet grouting method is to improve ground in a manner that cuts the ground with the injection of highly pressurized fluid and mixes soil with slurry binders[22]. Generally, the jet grouting method has advantages in that it can be implemented in areas close to existing underground structures, enhance the adhesion of improved soil and can be implemented with compact equipment having small diameters. There are several variations of the jet grouting method depending on the construction specifications such as the types of high pressure fluid (water and binders), intensity of pressure, flow rates and injection methods[221].

(2) Depending on the injection patterns, the jet grouting method can be classified largely into three types as shown in **Fig. 5.19.1**[221, 222]. The single fluid type injects binders through nozzles on the side wall of a boring rod, the double fluid type injects binders with air added to it and the triple fluid type injects water with air added to it, in addition to the independent injection of binders. In the double fluid and triple fluid types, injected air is discharged through boring holes along boring rods, and cut soil is also discharged to the ground surface along with the air[223] . Thus, the implementation of the double fluid and triple fluid types requires the removal of slime and replacement of portions of original ground with slurry. In contrast, because no air is injected, the single fluid type discharges less cut soil than the double fluid and triple fluid types. Thus, the single fluid type shall be implemented with attention paid to deformations such as the heaving of ground.

(3) In the preliminary surveys, due consideration shall be given to bounding stones and underground residues which may affect the performance of the jet grouting method.

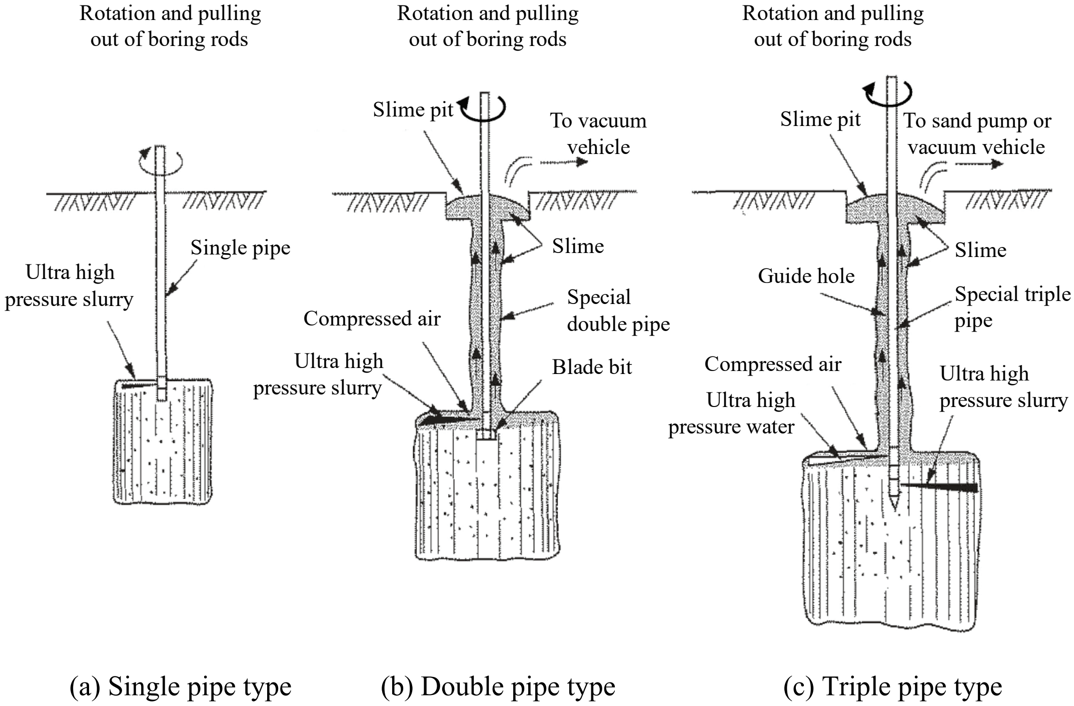


Fig. 5.19.1 Classification of the Types of Jet Grouting Method [221)]

5.19.2 Fundamentals of Performance Verification[223), 224)]

(1) The performance verification of ground improved through the jet grouting method shall be carried out by assuming the following dimensions of the improved bodies:

① Effective diameters;
② Physical properties; and
③ Arrangement patterns and minimum cross sections.

(2) When assuming the dimensions of the bodies improved through the jet grouting method, it is necessary to pay attention to the following items.

① The areas of improvement and improvement specifications shall be determined according to the purposes of improvement and ground conditions, respectively.

② There shall be no observations of a phenomenon where the ground surrounding the areas of improvement undergoes fractures with binders injected beyond the ground cutting areas.

③ Because the strength of the improved ground is adjustable by the types of binders, the binders shall be selected in accordance with the purposes of ground improvement.

④ When designing the jet grouting method for the areas of improvement which include layers of different properties (for example, cohesive and sandy soil layers), examinations shall be carried out with respect to the soil properties which lead to the weakest strength and the soil layers which lead to the least effective diameters in principle.

5.19.3 Dimensions of Improvement Bodies used in Performance Verification[224)]

(1) Effective diameters

In the jet grouting method, the design effective diameters are generally set for construction specifications such as the injection pressure, injection rates and pulling out speeds. In many cases, the design effective diameters are set in accordance with the type of method, classifications of soil properties of the improvement objective ground, *N*-

values and installation depths. In the cases of cohesive soil, the design effective diameters are generally set in consideration not only of the *N*-values but also of the cohesion of the ground. The design effective diameters of cohesive soil shall be set with particular attention to the scope of application because there may be cases where the design effective diameters cannot be secured when cohesive soil has high cohesion. Furthermore, for sand gravel, the design effective diameters shall be set with due consideration to the collapse of borehole walls and the generation of unimproved regions at the back of the gravel. For humus soil layers, the ground cutting performance may be reduced due to an inability to cut fibers. Thus, it is preferable to implement field test to confirm the effective diameters to the extent possible in the cases of sand gravel and humus soil layers.

(2) Physical properties of improved bodies

In the performance verification, the physical properties of the bodies improved through the jet grouting method to be used include the unconfined compressive strength, cohesion, adhesion, bending tensile strength and deformation coefficient. Because it is difficult to clarify the mix proportions of water, slurry and the object soil for improvement, mix proportion design has not generally been carried out for the jet grouting method, and, in many cases, the types of binders and standard design strength are shown in catalogues of the jet grouting method for the respective types of object soil for improvement. Because the standard design strength in the catalogues has been set based on the performance records, it is preferable to confirm the strength of the improved bodies through mix proportion tests or field test depending on the purposes, the level of importance and the sizes of improvement.

The strength of improvement bodies is larger in the jet grouting method than in other chemical stabilization methods. Recently, based on the assumption of application to soil improvement requiring lower strength, there has been development of a new jet grouting method where the construction specifications and use materials have been revised in view of enhancing economic performance[225), 226)].

(3) Improvement patterns

Typical improvement patterns of the jet grouting method include overlapped, ellipsoidal, wall-type, grid-type and pile-type improvements.

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6. Land Reclamation

6.1 General

(1) In the past, land reclamation in a sea area was commonly carried out by infilling only from the landside. However, as it has become necessary to extend the coastline and protect the local marine environment, at present there has been an increasing amount of planning and construction using the artificial island method. This Chapter deals with ground reclaimed by the artificial island method.

(2) It is often the case that soft sedimentary ground is found in ocean areas where land is to be reclaimed. Therefore, it is necessary to consider the possible occurrence of ground subsidence or liquefaction and to deal with the severe oceanographic weather conditions. Furthermore, at the same time, the impact on the natural environment and fisheries needs to be minimized.

(3) Performance verification of the ground to be reclaimed shall be carried out so that the required functional performance of the upper facilities can be secured during the working life set according to the purpose of the reclamation, taking into account the natural conditions, usage status and other various conditions, such as reclamation materials. Furthermore, it should also be certain that stability of the ground is not affected by construction of the upper facilities. The various conditions include natural conditions such as meteorological and oceanographic conditions, ground conditions and earthquakes. It is necessary to set these construction conditions adequately based on the results of thorough preliminary surveys and tests.

6.2 Survey and Condition-Setting for Land Reclamation

(1) Meteorological and Oceanographic Conditions

Meteorological and oceanographic conditions to be considered in land reclamation include winds, tide levels, waves, tsunamis, the movement of seawater, etc., estuarine hydraulics and littoral drift. If the existing data are not sufficient, new surveys must be carried out for an adequate period. For setting the conditions of the meteorological and oceanographic conditions, refer to **Part II, Chapter 2 Meteorology and Oceanography**.

(2) Ground Conditions

The contents of the surveys of ground conditions shall be that which is required to consider the soil profiles of the foundation ground, continuity of each layer, physical features, such as the unit weight of each soil layer, consistency and particle size distribution, as well as the mechanical features such as the consolidation characteristics and strength characteristics. If the reclamation load is large and settlement of the Pleistocene clay layer is expected, the physical and mechanical features of the Pleistocene clay layer shall also be surveyed. For the implementation of ground surveys and setting ground conditions, refer to **Part II, Chapter 3 Geotechnical Conditions**” and **Part II, Chapter 4 Earth Pressure and Water Pressure**.

In large-scale land reclamation, the intervals between ground survey points cannot be minimized to zero. For determining the number of survey points, the thickness and continuity of each soil layer constituting the foundation ground is important, and it is effective to confirm the layer thickness and continuity of each soil layer from past survey results, the land topography and by geophysical exploration methods such as acoustic exploration and surface wave exploration. When past survey results are utilized, it is necessary to set the preliminary survey points while considering their relative positional relationships to past survey points so that it can be verified whether the ground conditions have changed.

(3) Earthquakes

In order to accurately examine the stability of the reclamation revetments, the liquefaction of reclaimed ground and performance verification of upper facilities in case of earthquakes, information related to past earthquakes and active faults needs to be acquired. For setting earthquake motions and assessing liquefaction for these types of examinations, refer to **Part II, Chapter 6 Earthquakes** and **Part II, Chapter 7 Ground Liquefaction**.

(4) Material

The capacity to supply materials for reclamation directly affects the construction period of land reclamation. Therefore, it is necessary to select reclamation materials after thoroughly surveying the quarry sand and examining the supply plan. In selecting reclamation materials, the situations surrounding the reclamation revetments and the reclaimed ground during the working life need to be considered, and their physical properties need to be adequately assessed. The physical properties of reclamation materials include strength, unit weight and the friction coefficient. If the reclamation layer is thick, it is possible that the reclaimed ground is compressed due to its own weight;

therefore, it is desirable to confirm the compressibility of the reclamation materials, and the setting of the physical properties of the materials needs to be conducted carefully based on reliable data. Furthermore, the deterioration etc., of the materials due to environmental actions needs to be considered appropriately.

Reclamation materials include stones, quarry sand, sea sand, and recyclable resource materials including dredged soil. For stones and recyclable resource materials including dredged soil, refer to **Part II, Chapter 11, 5 Stones** and **Part II, Chapter 11, 7 Recyclable Resource Materials**. When materials are thrown into the sea, the reclaimed ground might not be uniform due to the different particle sizes of the reclamation materials. Particularly when a fine-grain fraction concentrates locally, the surface form the reclaimed ground and the function of the upper facilities might be affected. Therefore, for quarry sand, it is desirable to use gravelly soil which has a good grain size composition with less than 20% fine-grain fraction which shows less self-weight consolidation settlement after reclamation.

Liquefaction due to extrusion or earthquake motions might occur in ground reclaimed with sea sand, etc., so it is necessary to consider the possibility beforehand.

6.3 Performance Verification of Reclaimed Ground

6.3.1 General

(1) Performance verification of reclaimed ground needs to be carried out adequately while considering the purpose of reclamation, the utilization plan of the upper facilities, the construction period of reclamation and the characteristics of the reclamation materials.

(2) Reclaimed ground needs to have an adequate area, height and draining capacity according to its utilization plan. In addition, it is necessary to maintain an adequate ground height and flatness, and to make sure that it possesses liquefaction strength so that displacement, such as ground subsidence after reclamation due to reclamation load, or liquefaction due to an earthquake will not significantly damage the functions of the upper facilities.

(3) In the performance verification of reclaimed ground, the crown height of the reclaimed ground necessary for the functions of the upper facilities shall be set, the reclamation method shall be selected, the necessity for soil improvement of the foundation ground and reclaimed ground shall be assessed and the construction method shall be chosen based on the results of **Part III, Chapter 2, 6.2 Survey and Condition-Setting for Land Reclamation**.

6.3.2 Performance Verification of Reclaimed Ground

(1) The crown height of land reclaimed on soft ground changes due to settlement of the foundation ground even after the reclamation is completed. Therefore, the performance verification of reclaimed ground shall be carried out not only for the crown height at the time of the completion of the reclamation, but also for the reclamation layer thickness to secure the crown height of the reclaimed ground during the working life.

(2) The performance verification of land reclamation on soft ground shall be carried out for the following items.

① Consolidation settlement of foundation ground
② Uneven settlement
③ Compressive settlement of reclaimed ground
④ Liquefaction of reclaimed ground
⑤ Extrusion of reclamation materials

(3) The reclamation layer thickness shall be set as the difference between the necessary crown height at the end of the set working life and the height of the foundation ground at that time. Therefore, predictions of long-term consolidation settlement of the foundation ground are extremely important. Thus, if an Holocene clay layer or a Pleistocene clay layer, in which consolidation settlement can be expected, exists in the foundation ground at the planned location, it is necessary to carry out a thorough investigation concerning consolidation settlement and uneven settlement of the foundation ground during the construction period and the working life. Furthermore, a Pleistocene clay layer may cause long-term settlement even at a reclamation load smaller than the pre-consolidation load. Since it could be difficult to improve Pleistocene clay layers, the prediction of long-term settlement needs to be investigated with particular care.

(4) In order to prevent damage to the functions of the upper facilities during the working life, it is necessary to minimize uneven settlement of the reclaimed ground after the reclamation work is completed. In order to make the reclamation load on the foundation ground uniformly, the thickness of the reclamation layer shall be uniform in

principle. However, if the thickness of the clay layer which constitutes the foundation ground is uneven, a uniform reclamation load might cause uneven settlement. Therefore, if the thickness of the clay layer which constitutes the foundation ground is uneven, it is necessary to divide the land to be reclaimed into several sections and to predict long-term settlement and set the reclamation layer thickness for each section. If the reclamation layer thickness varies significantly between sections, it is necessary to consider installing a section for smoothing layer thickness in order to maintain adequate flatness of the reclaimed ground during the working life.

(5) The constructed crown height at the time of the completion of reclamation shall be set by adding the foundation ground height and the reclamation layer thickness at the time of completion. When the reclamation layer is thick, the reclaimed ground itself is compressed by its own weight. Therefore, if the compression of the reclaimed ground continues at the time of completion of the reclamation, the reclamation layer thickness needs to be determined while considering the remaining compression amount.

(6) In order to secure the stability of the foundation ground during construction and control uneven settlement after the completion of the reclamation, the necessity for soil improvement of the foundation ground shall be considered and the construction method shall be selected.

(7) It is desirable to confirm the validity of the settlement predictions and the effects of soil improvement by introducing a pilot construction area before the main construction starts. In addition, it is necessary to improve accuracy by reviewing the settlement predictions and the setting of the reclamation layer thickness by construction observation of the amount of consolidation settlement during the construction period.

(8) The necessity for soil improvement of the reclaimed ground shall be considered according to the required performance of the upper facilities.

(9) In order to control damage due to ground liquefaction at the time of an earthquake, adequate reclamation materials and construction methods shall be selected, the necessity for soil improvement of the reclaimed ground shall be considered and the construction method shall be chosen.

6.4 Selection of Soil Improvement Methods and Verification

For soil improvement of the foundation ground and reclaimed ground, a method to fulfill the utilization purpose of the reclaimed land and minimize the effects on the surrounding environment shall be selected while taking into account the construction period and cost. In a design, adequate methods shall be used with consideration given to the principles, construction methods and construction accuracy of the soil improvement methods. Some methods control the settlement amount and uneven settlement by stabilizing the ground while other methods control uneven settlement by reducing the residual settlement after the completion of reclamation by accelerating consolidation.

The major soil improvement methods for foundation ground and reclaimed ground used in past large-scale reclamation construction works are as follows. For the verification of soil improvement methods, refer to **Part III, Chapter 5 Soil Improvement Methods**.

(1) Soil Improvement of Foundation Ground

① Vertical drain method
② Sand compaction pile method
③ Deep mixing method
④ Replacement method

(2) Soil Improvement Method for Reclaimed Ground

① Compaction method for reclaimed ground (when the reclamation material is sandy soil)
② Pneumatic flow mixing method
③ Lightweight treated soil method
④ Surface soil stabilization method
⑤ Thin layer rolling compaction method

In the thin layer rolling compaction method, the ground shall be covered with a layer of material using a bulldozer and then compacted by a large vibrating roller. This method has an advantage where soil improvement can be carried out at the same time as the land reclamation work.

6.5 Selection of Reclamation Method

(1) When reclaiming land, the appropriate reclamation method needs to be selected while considering the reclamation materials, reclamation technique and utilization of reclaimed ground. The major reclamation methods are as follows. The reclamation work shall be carried out with a combination of these construction methods while switching between the construction methods according to the progress. It is important to pay attention to uneven settlement by carrying out the uniform construction accumulating divided thin layers over the entire reclaimed area or by minimizing the differences in construction periods in the neighboring construction areas, etc.

① **Direct feeding of quarry sand by hopper barges**

This method is suitable when the water is relatively deep and mountain soil is used as reclamation material.

② **Direct reclamation by sealed barges and unloader barges**

In this method, quarry sand for reclamation shall be fed directly from unloader barges because it is difficult for the barges to navigate in shallow waters. Since the ground under the water surface shall emerge due to reclamation, the load to the seabed will increase suddenly. Furthermore, the amount of area which can be reclaimed at one time is limited during construction with direct reclamation. Therefore, when the reclamation layer to be constructed by direct reclamation is thick, it is important to pay attention to the construction development so as to minimize uneven settlement after reclamation due to differences in construction periods as well as confirm the stability of the seabed, including the layer accumulated in previous stage of construction.

③ **Indirect reclamation by soil-heaping to revetments using unloader barges, transportation using dump trucks, the thin layer spreading method, and vibration compaction**

This process shall be carried out after creating land by direct reclamation. Here, the formation of uniform reclaimed ground is possible.

④ **Reclamation by pump dredgers**

In this construction method, the land is reclaimed by sucking up reclamation materials from the seabed or from barges utilizing pump dredgers. While this method ensures the uniformity of the reclaimed ground, it must be noted that a fine-grain fraction might concentrate locally if the reclamation material contains a fine-grain portion. After reclamation, it is necessary that scaffolding for soil improvement equipment be prepared using the surface treatment method and the vertical drain method shall be applied in order to secure the required ground strength.

(2) In addition to the methods mentioned above, the pneumatic flow mixing method, the light-weight treated soil method, the premixing-type stabilization method and the like shall be adopted in order to reduce the earth pressure and the reclamation load on the reclamation revetments.

6.6 Supervision of Construction Work

(1) Reclamation on a soft seabed often requires soil improvement of the foundation ground in order to secure its stability during construction. When methods are adopted where an increase in strength can be expected due to the consolidation of improved ground by the vertical drain method, phased construction with some suspension periods of loading for consolidation shall be adopted and the construction layer thickness at each construction phase shall be set while considering the development of the ground strength. For setting phased construction, refer to **Part III, Chapter2, 5.4 Vertical Drain Method**.

(2) When the seabed is soft ground, the supervision of subsequent construction work for safety construction shall gradually become easier because the ground strength shall increase as the effect of soil improvement induced by the reclamation load of the preceding construction. However, as the layer becomes thicker, the management of residual settlement, uneven settlement and the like of the reclaimed ground shall become even more important.

(3) Comprehension of filling forms by bathymetric survey

Comprehension of reclamation layer thickness underwater and management of construction periods and places shall affect the functions of the upper facilities. Particularly for large-scale construction works such as offshore artificial islands, it is desirable to conduct bathymetric surveys by depth-measuring sonars which can obtain wide-range data logically and effectively.

(4) Control of layer thickness and settlement

For reclamation on soft ground, control of reclamation layer thickness which provides load is essential in order to improve the accuracy of settlement control. It is important for the control of layer thickness to understand the filling locations of quarry sand by barges, filling sand amount and filling form. Since it is expected that the reclaimed land shall be compressed, the reclamation layer thickness shall be calculated not as the accumulation of the completed forms at each construction stage (assuming that the layer thickness at the completion of each construction stage remains unchanged), but as the difference between the reclaimed crown height and the foundation ground height. Therefore, settlement control by measuring settlement of the foundation ground is also important.

(5) Comprehension of the compression amount of the reclaimed ground

As the compression amount of the reclaimed ground is necessary for setting the construction layer thickness and the final construction crown height as well as for predicting the final required amount of sand, it is important to comprehend the amount during the construction period.

(6) Consideration of differences in construction periods between reclamation revetments and reclaimed ground

When working just behind a reclamation revetment in reclamation construction on soft ground, it is necessary to make efforts to understand the settlement amount and to make the residual settlement amount uniform in order to reduce the amount of uneven settlement as much as possible while considering differences in the construction periods between the reclamation revetment and the reclaimed ground.

For reclamation on soft ground, since the settlement of the preceding reclamation revetment shall progress, if the construction of reclaimed ground is carried out continuously solely based on the management of the crown height, the layer thickness just behind the revetment shall become thicker than the layer thickness in other parts. Therefore, attention must be paid to make the load (layer thickness) and settlement amount uniform in order to secure the stability, control deformation of the revetment, and control uneven settlement of the reclaimed part.

6.7 Maintenance

(1) For reclaimed ground on soft ground, settlement of the foundation ground might be unavoidable even after the reclamation is completed. It is desirable to make considerations at the design and construction stages of the reclaimed ground so that maintenance after the completion of the reclamation can be carried out appropriately. In addition, in order to confirm the ground height of the reclaimed land needed for land utilization, it is important to carry out measurement of the settlement amount continuously even after completion of the reclamation concerning the ground subsidence of the reclaimed land. In some cases, it is important to survey and measure the rising ground water level as the ground subsides.

(2) For long-term predictions of settlement, which are important for the maintenance of the reclaimed land, the settlement history from the start of construction and the speed of settlement is important in addition to the daily settlement amount. Therefore, it is important to install equipment to measure settlement before the start of the reclamation work to continuously measure the settlement during the construction period and after the completion of reclamation.

(3) If the gap between the actual and the predicted settlement amounts tends to increase, it is desirable to improve the accuracy of the long-term settlement prediction method by considering renewing the mechanical model used to predict settlement and by changing the model of soil profiles and the ground parameters.

Chapter 3 Waterways and Basins

1 General

[Ministerial Ordinance] (General Provisions)

Article 8

1 Waterways and basins shall be located appropriately in light of geotechnical characteristics, meteorological characteristics, sea states and other environmental conditions, as well as navigation channels and other usage conditions of water area around the facilities.

2 In waterways and basins where it is necessary to maintain the calmness of water area, measures shall be taken to mitigate the effects of waves, water currents, winds, etc.

3 In waterways and basins in which there is a risk of siltation by sediments, etc. measures shall be taken to prevent the occurrence thereof.

[Ministerial Ordinance] (Necessary Items concerning Waterways and Basins)

Article 12

The necessary matters for the enforcement of the requirements as specified in this chapter by the Minister of Land, Infrastructure, Transport and Tourism and other performance requirements for waterways and basins shall be provided by the Public Notice.

[Public Notice] (Waterways and Basins)

Article 29

The items to be specified by the Public Notices under Article 12 of the Ministerial Ordinance concerning the performance requirements of waterways and basins shall be as provided in the following Article through Article 32.

(1) In selecting the locations for basins or small craft basins exclusively used by hazardous cargo ships, the following should be considered: minimize encounters with general ships, especially passenger ships; isolate them from the facilities of which the surrounding environment should be protected, such as residential areas, schools and hospitals; and promptly deal with accidents including spills of hazardous cargo.

(2) For ensuring safety and efficiency in navigation and cargo handling, it is preferable to separate the basins and small craft basins for passenger ships, ferries and fishing boats from those for other types of ships.

(3) In principle, it is preferable to allocate specialized terminals for timber handling which are isolated from other general facilities.

(4) Measures to maintain the calmness of basins and small craft basins include protective facilities for harbors such as breakwater, and the installation of wave-dissipating work as well as alongshore wave protection work.

(5) Measures to prevent siltation due to sediment include the following:

① Installation of protective facilities for harbors such as sediment control groins and training jetties as well as other equivalent facilities

② Sediment control for facilities such as sand pockets by spot dredging around waterways and basins

③ Installation of facilities such as waterway revetments for slope protection

④ Excessive dredging

2 Navigation Channels

[Ministerial Ordinance] (Performance Requirements for Navigation Channels)

Article 9

The performance requirements for navigation channels shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied in light of geotechnical characteristics, waves, water currents and wind conditions, as well as the usage conditions of the surrounding water areas so as to secure safe and smooth navigations by ships.

[Public Notice] (Performance Criteria of Navigation Channels)

Article 30

The performance criteria of navigation channels shall be as prescribed respectively in the following items:

(1) Navigation channels shall have appropriate width that are equal to or greater than the lengths of the design ships in navigation channels where there is a possibility of ships passing each other or width that are equal to or greater than half of the length of the design ships in navigation channels where there is no possibility of ships passing each other, in light of the length and width of the design ship, the traffic volume of ships, the conditions of geotechnical characteristics, waves, water currents and winds, as well as the usage conditions of the surrounding water areas. Provided, however, that in cases where the mode of navigation is special, the width of the navigation channels can be reduced to the width that does not hinder the safe navigation of ships.

(2) Navigation channels shall have appropriate depths greater than the drafts of the design ships in consideration of the trim and the degree of ship motions of the design ship due to waves, water currents, winds, etc.

(3) Directions of navigation channels shall be such that safe ship navigation is not hindered, in light of the geotechnical conditions, waves, water currents and winds, as well as the usage conditions of the surrounding water areas.

(4) If navigation channels is remarkably congested, navigation channels shall be provided with lanes separated depending on the direction of movement or the sizes of ships.

[Interpretation]

9. Waterways and Basins

(1) Performance Criteria of Navigation Channels (Article 9 of the Ministerial Ordinance and the interpretation related to Article 30 of the Public Notice)

① The required performance of navigation channels shall be serviceability. Here, serviceability means the performance of navigation channels which enables ships to navigate safely and smoothly.

② **Widths of navigation channels**

Cases where the modes of navigation are special include navigation channels which require consideration for the use of tug boats or installation of water areas for refugees and navigation channels which have significantly shorter distances. The term "significantly shorter distances" can be applied to both the entire lengths of the navigation channels and their partial lengths.

③ **Directions of navigation channels**

The conditions of geotechnical characteristics are the ground and underground phenomena closely related to earthquakes, volcanic activities, uplift and settlement of ground, and the weather.

2.1 General

(1) Concept of Navigation Channels

Navigation channels are considered to be water areas, such as entrance and passage channels in shallow water areas, whose existence is clearly marked with buoys or other navigation aids so as to enable navigators to perform safe and smooth navigation of ships.

(2) Items to be Considered

The performance verification of navigation channels shall be carried out with due consideration to the design ships and navigation environment, particularly the depths, widths and alignments (bend sections) of navigation channels.

(3) Classification of Verification Methods

The verification methods for navigation channels can be classified as follows depending on whether or not design ships and navigation environment are specified. Because the Class 2 method below allows the navigation environment to be designated and changed, the method can be applied to cases for determining whether or not to allow ships to enter the existing navigation channels.

(a) Class 1: Cases where the design ship and navigation environment are not specified.

(b) Class 2: Cases where the design ship and navigation environment are specified.

(4) The performance verification of navigation channels can be based on the methods described in 1) and 2) below from **Part III, Chapter 3, 2.2 Depths of Navigation Channels** to **2.4 Alignment of Navigation Channels (Bends)**, which are proposed by the Japan Institute of Navigation Standard Committee and the Port and Harbor Department of the National Institute for Land and Infrastructure Management, the Ministry of Land, Infrastructure, Transport and Tourism.

(5) Performance Criteria of Navigation Channels

① **Depths of navigation channels (serviceability)**

(a) Cases where the design ship and navigation environment cannot be specified

For the performance verification of navigation channels in cases where the design ship and navigation environment are not specified, the following values can be used as appropriate depths greater than the maximum drafts of the design ships.

- In navigation channels inside harbors where the effects of waves such as swells are negligible: 1.10 times the maximum draft.
- In navigation channels outside harbors where the effects of waves such as swells are expected: 1.15 times the maximum draft.
- In navigation channels in the open sea where the effects of waves such as strong swells are expected: 1.20 times the maximum draft.

(b) Cases where the design ship and navigation environment are specified

In setting the water depths of navigation channels for performance verification of navigation channels in cases where the design ship and navigation environment are specified, appropriate consideration shall be given to the maximum drafts of the design ships, ship squatting due to ship waves or swells, and keel clearances.

(c) Cases where the modes of navigation are special

In setting the water depths for performance verification of navigation channels used for special modes of navigation such as navigation channels accommodating ships entering or leaving dry docks, or ships unloading at multiple ports during single voyages as part of normal operation, notwithstanding the items mentioned in (a) and (b) above, the water depths shall be set appropriately in consideration of the anticipated conditions of use of the navigation channels concerned. For example, when ships are expected to enter a port in significantly light load conditions, it is preferable to identify the design ships and navigation environment and evaluate the effects of waves such as swells on ship squatting. In such cases, the literature 3) can be used as a reference.

② **Widths of navigation channels (serviceability)**

(a) Cases where the design ship and navigation environment are not specified

1) Appropriate widths of navigation channels where it is possible for ships to pass each other

In the performance verification of navigation channels where it is possible for ships to pass each other in cases where the design ships and navigation environment cannot be designated, the following values can be used as appropriate widths greater than the lengths overall of the design ships.

- For comparatively long navigation channels: 1.5 times the lengths overall of the design ships.

- For navigation channels with design ships frequently passing each other during their navigation: 1.5 times the lengths overall of the design ships.
- For comparatively long navigation channels with design ships frequently passing each other during their navigation: 2.0 times the lengths overall of the design ships.

2) Appropriate widths of navigation channels where there is no possibility for ships to pass each other

In the performance verification of navigation channels where there is no possibility for ships to pass each other in cases where the design ships and navigation environment are not designated, the appropriate widths shall be 0.5 times the lengths overall of the design ships or greater. Provided, however, that in cases where the widths of the navigation channels are less than the lengths overall of the design ships, adequate countermeasures to ensure safe navigation, such as the provision of facilities to support ship navigation, shall be examined.

(b) Cases where the design ship and navigation environment are specified

When setting the widths of navigation channels in the performance verification of navigation channels in cases where the design ships and navigation environment are specified, appropriate consideration shall be given to the basic ship maneuvering widths and the widths necessary to cope with the effects of the side walls of the navigation channels, ships passing each other and ships overtaking other ships.

(c) Cases with special modes of navigation

Cases where there are special modes of navigation include navigation channels which require consideration for the use of tug boats or installation of water areas for refugees and navigation channels with significantly shorter distances. The term “significantly shorter distances” can be applied to both the entire lengths and the partial lengths (subject to examination) of navigation channels.

③ Directions of navigation channels (serviceability)

(a) Whenever possible, the directions of navigation channels shall be straight, provided, however, that in cases where bends are required in navigation channels, the intersection angles of the centerlines of the navigation channels at the bends shall not exceed roughly 30°.

(b) Cases where the intersection angles of the centerlines of the navigation channels at bends exceed 30°

1) Cases where the design ship and navigation environment such as rudder angles are not specified

In the performance verification of navigation channels in cases where the intersection angles of the centerlines of the navigation channels at bends exceed 30° and the design ships and features of the navigation environment such as rudder angles are not designated, the inner side of the bends shall be provided with appropriate corner cutoffs and the curvature radius of the centerlines of the navigation channels at the bends set to roughly 4 times the lengths between perpendiculars of the design ships or greater.

2) Cases where the design ship and navigation environment such as rudder angles are specified

In the performance verification of navigation channels in cases where the intersection angles of the centerlines of the navigation channels at bends exceed 30° and the design ships and features of the navigation environment such as rudder angles are designated, the inner side of the bends shall be provided with appropriate corner cutoffs and the curvature radius of the centerlines of the navigation channels at the bends set appropriately in consideration of turning performance factors considering the turning characteristics of design ships.

(c) When expanding the widths of navigation channels at bends, the planar shapes of the inner sides of the bends can be curved, except for corner cutoffs, in consideration of the installation of buoys.

2.2 Depths of Navigation Channels

2.2.1 Fundamentals of Performance Verification

(1) The following values can be used as the required water depths for Class 1 navigation channels.

①Navigation channel in a port where waves including swell does not affect ship motion : $D = 1.10\,d$
②Navigation channel out of a port where waves including swells affect ship motion : $D = 1.15\,d$
③Navigation channel in open water where waves including swells exist : $D = 1.20\,d$ **(2.2.1)**

where

D : depth of the navigation channel (m);

d : maximum draft of a moored design ship in still water (m);

(2) The required water depths for Class 2 navigation channels can be calculated by **equation (2.2.2)**.

$$D = d + D_1 + \max(D_2, D_3) + D_4 \qquad \textbf{(2.2.2)}$$

where

D : depth of the navigation channel (m);

d : maximum draft of a moored design ship in still water (m);

D_1 : bow sinkage during navigation (m);

D_2 : bow sinkage due to heaving and pitching motions (additional item in the case of $\lambda > 0.45\,L_{pp}$) (m);

D_3 : bilge keel sinkage due to heaving and rolling (as shown in **Fig. 2.2.1**) (additional item in the case of $TR \approx TE$) (m);

D_4 : underkeel clearance (m);

λ : length of the wave such as the swell (m);

L_{pp} : length between perpendiculars (m);

TR : natural rolling period of the design ship (s);

TE : encounter period of the design ship and design wave such as the swell (s).

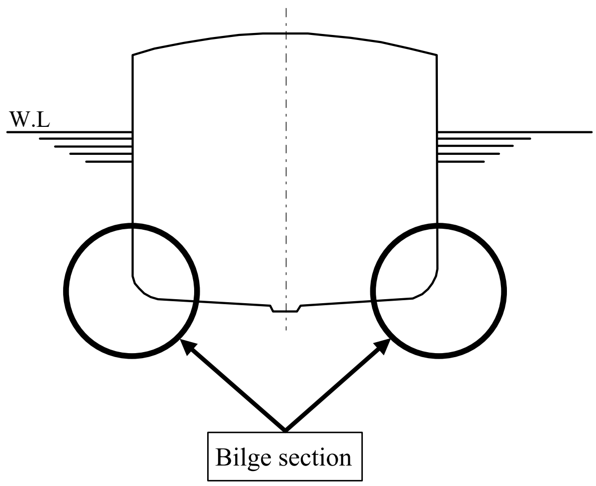


Fig. 2.2.1 Bilge Section

(3) Common items

① Swells subject to performance verification is determined by the relationship between the lengths overall of the design ships and the wavelengths in the water areas concerned.

② The maximum drafts d of maximum draft of a moored design ship in still water (m) is deemed to be, in the maximum, $d = d_0$ (d_0: full load draft), however, it can be $d < d_0$ depending on the operational condition concerned.

③ The following items can be considered when setting the depths D of the navigation channels:

(a) Tide levels: Tide levels during navigation are generally above the lowest astronomical tide and the difference between the tidal levels and the lowest astronomical tide can be considered as a factor which increases the actual water depths of the navigation channels. When considering the tide levels in the water depths of the navigation channels, it is necessary to ensure that tide levels are sufficiently high as per the consideration. The literature 4) introduces the actual situation of the use of tide levels with items to be considered in the implementation of their use.

(b) Accuracy of water depths: Errors in bathymetric data in the nautical charts may pose a danger to navigation, but in general, the actual water depths are deeper than the planning depths when dredging is implemented. Thus, excessive water depths below the planning depths can also be considered as a factor which increases the actual water depths of the navigation channels, provided, however, that such excessive water depths are confirmed through sufficient bathymetric surveys.

(c) Other: It is preferable to consider atmospheric pressure, bottom sediment, obstructions at the seabed, the specific gravity of seawater, etc., as needed. In the case of the existence of a soft clay layer with high water content close to the seafloor, it is necessary to pay attention to a possible discrepancy between the measurement results when using echo sounders and the measurement results when using sounding lead, which causes the underestimation of the depth measurement results when using echo sounders.[5) 6)]

2.2.2 Performance Verification of Class 2 Navigation Channels

(1) The required depths of Class 2 navigation channels can be calculated using the following methods.

① The bow sinkage D_1 during navigation can be calculated by **equation (2.2.3)** proposed by Yoshimura.[7)]

$$\left.\begin{aligned} D_1 &= \left(0.7+1.5\frac{d}{D}\right)\left(\frac{C_b}{L_{pp}/B}\right)\frac{U^2}{g}+15\frac{d}{D}\left(\frac{C_b}{L_{pp}/B}\right)^3\frac{U^2}{g} \\ C_b &= DT/(L_{pp}\,B_d\gamma) \end{aligned}\right\} \qquad \textbf{(2.2.3)}$$

where

D : depth of the navigation channel (m);

d : maximum draft of a moored design ship in still water (m);

L_{pp} : length between perpendiculars (m);

B : molded breadth (m);

C_b : block coefficient;

DT : displacement tonnage of the design ship (t);

γ : density of seawater (t/m^3);

U : ship speed (m/s);

g : gravitational acceleration (m/s^2).

When block coefficients C_b (representing the degree of fatness or slenderness of the hulls) are unknown, the values in **Table 2.2.1** can be used.[8)]

Table 2.2.1 Block Coefficient C_b[8)]

Design ship	50% value	Standard deviation
Cargo ship	0.804	0.0712
Container ship	0.668	0.0472
Tanker	0.824	0.0381
Roll on/roll off (RORO) vessel	0.667	0.0939
Pure Car Carrier (PCC) ship	0.594	0.0665
LPG ship	0.737	0.0620
LNG ship	0.716	0.0399
Passenger ship	0.591	0.0595
Short-to-medium distance ferry	0.548	0.0452
Long distance ferry	0.516	0.0295

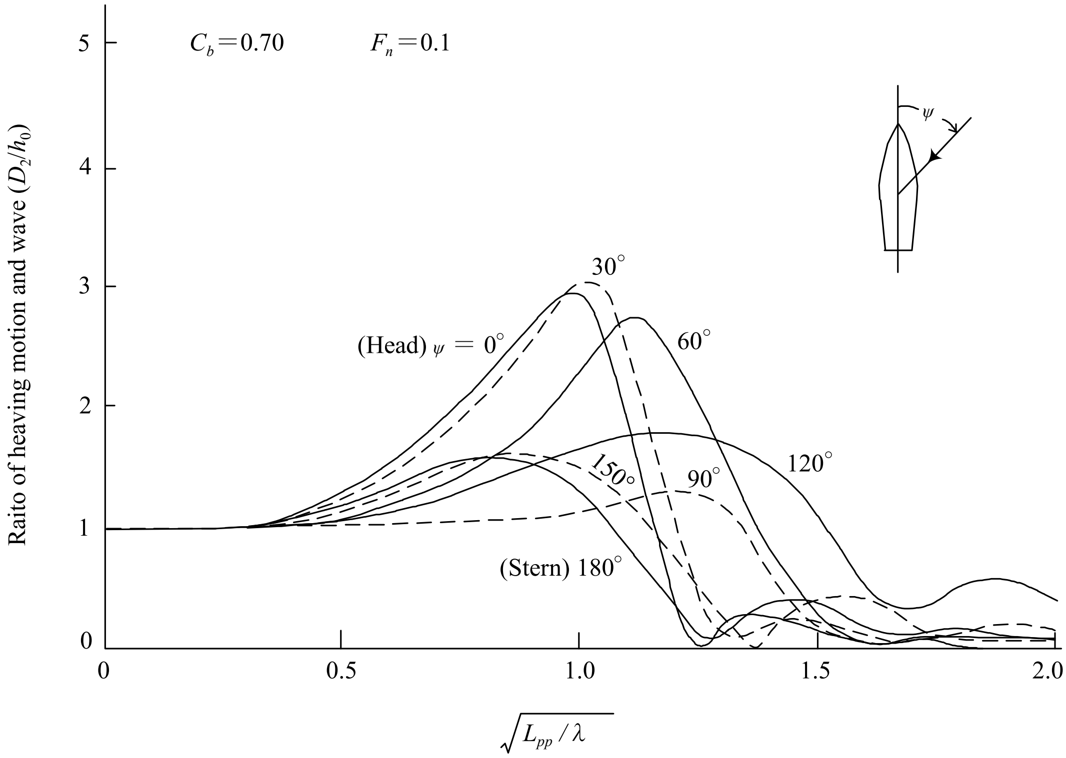
② The maximum value D_2 of bow sinkage due to heaving and pitching motions and the maximum value D_3 of bilge keel sinkage due to heaving and rolling do not occur at the same time. Therefore, as expressed by max (D_2, D_3) in **equation (2.2.2)**, D_2 or D_3, whichever is greater, shall be used.

where

D_2 : bow sinkage due to heaving and pitching motions (additional item in the case of $\lambda > 0.45\ L_{pp}$) (m);

D_3 : bilge keel sinkage due to heaving and rolling (additional item in the case of $TR \approx TE$) (m).

(a) In the case of $\lambda > 0.45\ L_{pp}$, D_2 can be calculated by the value of D_2/h_0 taken from **Fig. 2.2.2**.



Note: The figure above shows a case where C_b = 0.7 and F_n = 0.1 only. However, because the case represents phenomena in deep water areas which have larger values than shallow water areas (safer values for shallow water areas), the relationship can be applied to all cases regardless of the values of C_b and F_n (Froude number: $F_n = U/(L_{pp}g)^{0.5}$).

Fig. 2.2.2 Relationship between Heaving and Pitching Motions and Encounter Waves [9)]

where

λ : length of the wave such as the swell (m);

h_0 : amplitude of the wave such as the swell ($h_0 = H/2$) (m);

H : height of the wave such as the swell ($H = 0.7\ H_{1/3}$) (m);

$H_{1/3}$: significant height of the wave such as the swell (m);

L_{pp} : length between perpendiculars (m);

ψ : encounter angle (°) between the moving direction of the ship and the propagation direction of the design wave such as the swell.

(b) In cases where the natural rolling periods TR of design ships are almost equal to the encounter periods TE of design ships with design waves such as swells, D_3 can be calculated by **equation (2.2.4)**.[10)]

$$\left.\begin{aligned} & D_3 = 0.7\left(\frac{H_{1/3}}{2}\right) + \left(\frac{B}{2}\right)\sin\Theta \\ & \text{provided that} \\ & \Theta = \mu\gamma\,\Phi \\ & \Phi = 360(0.35H_{1/3}/\lambda)\sin\psi \end{aligned}\right\} \quad \textbf{(2.2.4)}$$

where

D_3 : bilge keel sinkage due to heaving and rolling (additional item in the case of $TR \approx TE$) (m);

$H_{1/3}$: significant height of the wave such as the swell (m);

B : molded breadth (m);

Θ : maximum rolling angle of the design ship (°);

μ : rolling amplitude in regular waves

γ : effective wave slope coefficient;

Φ : maximum inclination angle of the surface wave (°) with respect to the line perpendicular to the fore and aft direction;

λ : length of the wave such as the swell (m);

ψ : encounter angle (°) between the moving direction of the ship and the propagation direction of the design wave such as the swell.

Here, $\mu\gamma$ can generally be a value up to 3.5 in the maximum.[11)]

1) TR and TE can be calculated by **equation (2.2.5)**.

$$\begin{aligned} & TR = 0.8B/(GM)^{0.5} \\ & TE = \lambda/(\lambda/TW + U\cos\psi) \end{aligned} \quad \textbf{(2.2.5)}$$

where

TR : natural rolling period of the design ship (s);

TE : encounter period of the design ship and the design wave such as the swell (s);

B : molded breadth (m);

GM : distance between the gravity center and metacenter of the ship (m);

TW : period of the wave such as the swell (s);

λ : length of the wave such as the swell (m);

U : ship speed (m/s);

ψ : encounter angle (°) between the moving direction of the ship and the propagation direction of the design wave such as the swell.

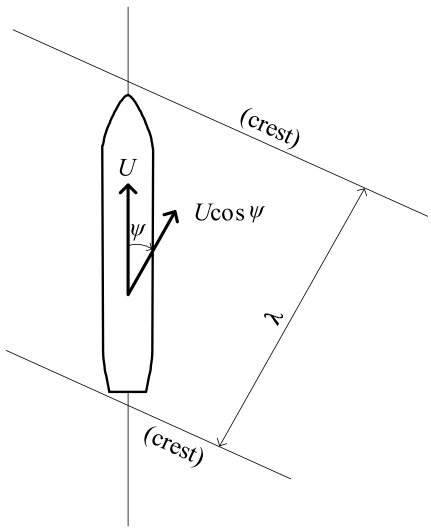


Fig. 2.2.3 Setting Method of ψ

2) It is considered appropriate to calculate the distance GM between the gravity center and metacenter of a ship by $GM = B/25$. However, considering that the actual GM varies, GM can be set with reference to the value calculated by **equation (2.2.6)**.

$$GM = a\left(B/25\right) \qquad \textbf{(2.2.6)}$$

where

GM : distance between the gravity center and metacenter of the ship (m);

B : molded breadth (m);

a : 0.5 to 2.0.

③ D_4 is allowance for sinkage due to hull inclination when the ship is steered and can be calculated by **equation (2.2.7)**.

$$\left.\begin{array}{ll} D_4 = 0.5 \ \ \text{(m)} & d \leq 10\text{m} \\ D_4 = 0.05d \ \ \text{(m)} & d > 10\text{m} \end{array}\right\} \qquad \textbf{(2.2.7)}$$

where

D_4 : allowance in under keel clearance (m);

d : maximum draft of a moored design ship in still water (m).

(2) Convergence Calculation for the Design Depths of Newly Planned Navigation Channels

The depth of a navigation channel D needs to be input into **equation (2.2.3)** to calculate a bow sinkage during navigation D_1, which is the basic element to calculate the depth of a navigation channel D. Thus, in calculating the depth of a newly planned navigation channel, it is necessary to repeat convergence calculation until the initially set depth of the navigation channel D becomes equal to the depth of the navigation channel D calculated by **equation (2.2.8)**.

$$D = d + D_1 + \max\left(D_2, D_3\right) + D_4 \qquad \textbf{(2.2.8)}$$

(3) Application to the Design Changes of Existing Navigation Channels

In cases involving a design change of the design ships and navigation environment of existing navigation channels, the depths of these channels can be the depths of navigation channels D to be input into **equation (2.2.3)** to calculate bow sinkage during navigation D_1. Then, the calculated depths of navigation channels D can be evaluated by **equation (2.2.9)**.

$$D \text{ (depth of an existing navigation channel)} \geq D \text{ (calculated depth of a navigation channel)} \qquad \textbf{(2.2.9)}$$

In cases where the above equation cannot be satisfied, the design changes need to be reviewed or the design depths need to be deepened to the depths obtained through the convergence calculation as is the case for new navigation channels.

2.3 Performance Verification of the Widths of Navigation Channels

2.3.1 Fundamentals of Performance Verification

(1) Generally, the following values can be used as the required widths of Class 1 navigation channels.

① In the cases of navigation channels which do not allow ships to pass each other, the appropriate widths can generally be set at 0.5 L_{oa} or more. For widths less than 1.0 L_{oa}, it is advisable to take adequate safety measures such as the installation of navigation aids.

② In the cases of navigation channels which allow ships to pass each other, the appropriate widths can generally be set at 1.0 L_{oa} or more, provided, however, that:

(a) those navigation channels which have comparatively long lengths : $W = 1.5L_{oa}$

(b) those navigation channels which have frequent navigation of design ships passing each other : $W = 1.5L_{oa}$

(c) those navigation channels which have comparatively long lengths and frequent navigation of design ships passing each other : $W = 2.0L_{oa}$ **(2.3.1)**

where

W : width of the navigation channel (m);

L_{oa} : length overall of the design ship (m).

(2) The values calculated by the following equation can be used as the required widths of Class 2 navigation channels.

① Navigation channels without accounting for two-ship interaction in passing (**Fig. 2.3.1** One-Way Navigation Channels)

$$W = W_{b1} + W_{m0} + W_{b2} \qquad \textbf{(2.3.2)}$$

② Navigation channels with accounting for two-ship interaction in passing (**Fig. 2.3.2** Two-Way Navigation Channels)

$$W = W_{b1} + W_{m1} + W_c + W_{m2} + W_{b2} \qquad \textbf{(2.3.3)}$$

③ Navigation channels with accounting for two-ship interaction in both passing and overtaking (**Fig. 2.3.3** Two-Way Navigation Channels That Allow Ships to Overtake Other Ships)

$$W = W_{b1} + W_{m1-1} + W_{ov1} + W_{m1-2} + W_c + W_{m2-1} + W_{ov2} + W_{m2-2} + W_{b2} \qquad \textbf{(2.3.4)}$$

where

W : width of the navigation channel (m);

W_{mi} : width of basic or fundamental ship maneuvering lane (m);

W_{bi} : additional width to account for bank effect (m);

W_c : additional width to account for two-ship interaction in passing (m);

W_{ovi} : additional width to account for two-ship interaction in overtaking (m).

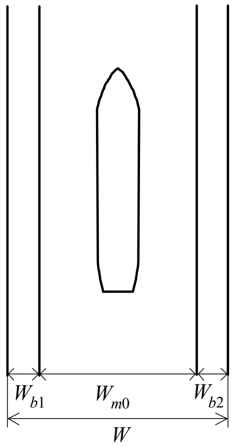


Fig. 2.3.1 One-Way Navigation Channels

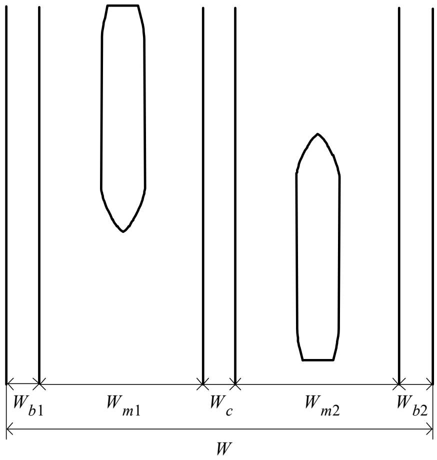


Fig. 2.3.2 Two-Way Navigation Channels

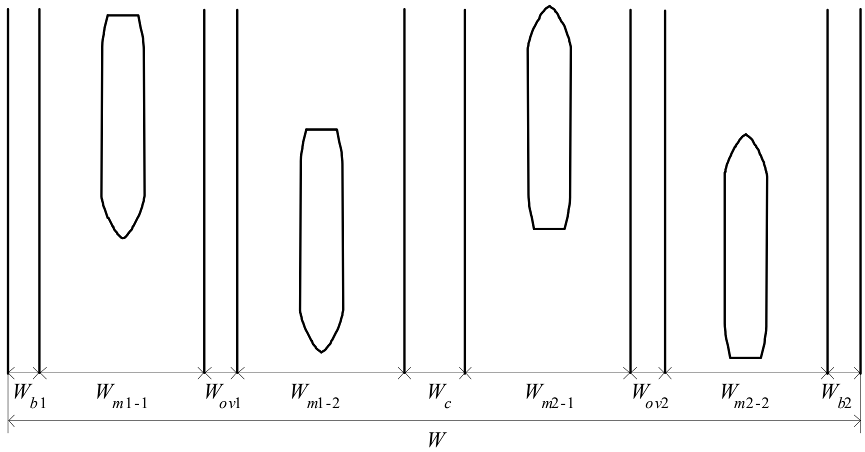


Fig. 2.3.3 Two-Way Navigation Channels to Account for Two-Ship Interaction in Overtaking

2.3.2 Performance Verification of Class 2 Navigation Channels

(1) The required widths of Class 2 navigation channels can be calculated by the following methods.

① Width of basic ship maneuvering lane W_{mi} can be obtained from the following two elements:

1) $W_m(\beta, y)$: additional widths to account for wind forces, current forces and yawing motion; and

2) $W_m(S)$: additional width to account for drift detection.

Here, the width of basic ship maneuvering lane W_{mi} can be calculated by **equation (2.3.5)** in the form of the maximum widths measured from the centerline to either edge.

$$0.5W_{mi} = W_m(S) + 0.5W_m(\beta, y) \tag{2.3.5}$$

Therefore, the width of basic ship maneuvering lane W_{mi} can be calculated by **equation (2.3.6)**.

$$W_{mi} = 2W_m(S) + W_m(\beta, y) \tag{2.3.6}$$

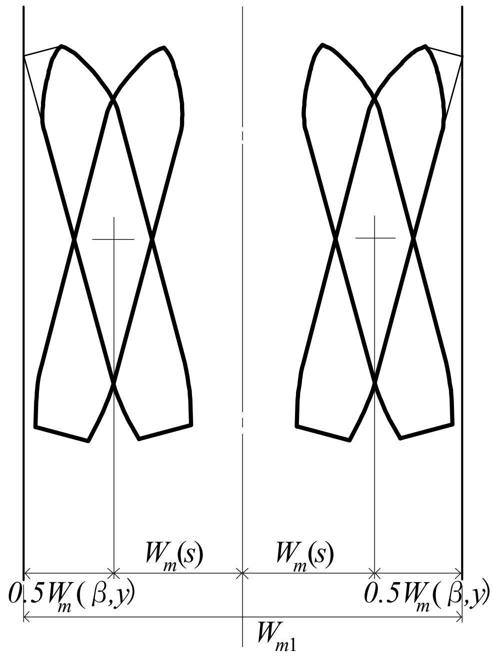


Fig. 2.3.4 Concept of Basic Ship Maneuvering Width

(a) The additional widths W_m (β, y) to account for wind forces, current forces and yawing motion can be calculated by the following procedure.

1) Basic concept of calculation

i. Calculate rudder angles δ to compensate for the effects of winds and drift angles β based on the design ships and navigation environment conditions. Note, however, that the rudder angles shall be up to 15° to compensate for the effects of winds. In cases where the check rudder angles exceed 15°, it is necessary to revise the setting of the maximum wind speed as one of the port call conditions. In addition, the drift angles to compensate for the effects of tidal currents need to be calculated based on the tidal component whose direction is perpendicular to the centerlines of the navigation channels as one of the navigation environmental conditions.

ii. The required widths of the navigation channels to cope with the effects of winds and tidal currents can be calculated from the drift angles obtained as a result of combining the drift angles to compensate for the effects of winds and tidal currents, respectively. Then, the required widths to cope with the effects of winds, tidal currents and yawing can be calculated by adding the widths required to compensate for meandering due to yawing to the above drift angles.

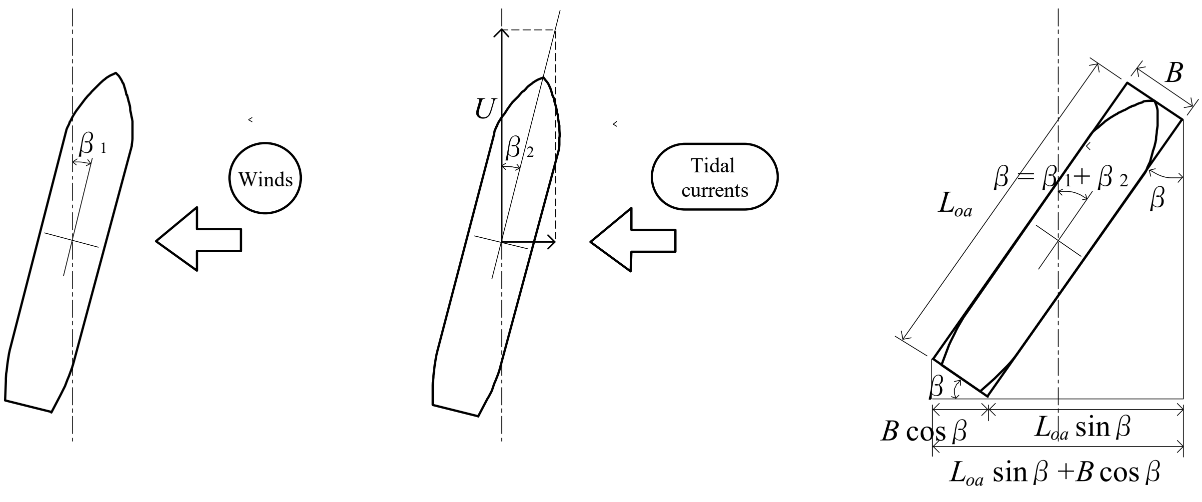


Fig. 2.3.5 Concept of Drift Angles to Cope with the Effects of Winds and Tidal Currents

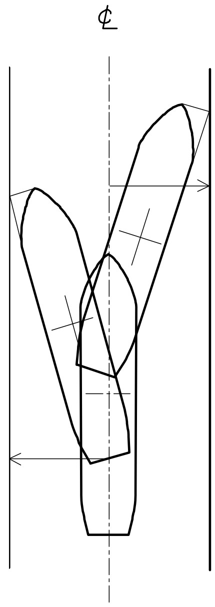


Fig. 2.3.6 Meandering Width Due to Yawing

2) Standard calculation method and calculation equation

i. Calculation of drift angles β_1 due to the effects of winds

Generally, the drift angles due to the effects of winds can be calculated by the following phased calculation method.

Setting of design ship dimensions, ship speeds and navigation environment conditions such as anticipated natural conditions
↓
Calculation of the coefficients of fluid dynamic forces
↓
Calculation of the coefficients of wind pressure resistance and wind pressure moment
↓
Calculation of rudder and drift angles based on the motion equation in an equilibrium situation

Fig. 2.3.7 Calculation of Drift Angle β_1 Due to the Effects of Winds

ii. Calculation of the coefficients of fluid dynamic forces

a) The coefficients of fluid dynamic forces with respect to ships can be calculated by **equation (2.3.7)**[1] based on the equation proposed by Hirano et al.[12] in consideration of the stabilization effects of rudders.

$$\left.\begin{aligned} Y'_\beta &= \frac{\pi}{2}\frac{k}{\dfrac{d}{2D}k+\left\{\dfrac{\pi d}{2D}\cot\left(\dfrac{\pi d}{2D}\right)\right\}^{2.3}}+1.4C_b\frac{B}{L}-0.4Y'_\delta \\ N'_\beta &= \frac{k}{\dfrac{d}{2D}k+\left\{\dfrac{\pi d}{2D}\cot\left(\dfrac{\pi d}{2D}\right)\right\}^{1.7}}+0.49\left(0.4Y'_\delta\right) \\ &\text{provided that} \\ k &= \frac{2d}{L} \end{aligned}\right\} \qquad \textbf{(2.3.7)}$$

where

Y'_β : dimensionless value of the coefficient Y_β for the reaction force in a transverse direction from a fluid when a ship diagonally navigates with a drift angle β;

N'_β : dimensionless value of the coefficient N_β for the turning reaction moment from a fluid when a ship diagonally navigates with a drift angle β;

D : depth of the navigation channel (m);

d : maximum draft of a moored design ship in still water (m);

L : length between perpendiculars (= L_{pp}) (m);

B : molded breadth (m);

Y'_δ : dimensionless value of a transverse force coefficient Y_δ generated by a rudder set at rudder angle δ;

C_b : block coefficient, $C_b = DT/(L_{pp}Bd\gamma)$;

DT : displacement tonnage of the design ship;

γ : density of seawater (t/m^3).

b) The coefficients of fluid dynamic forces with respect to the rudder force corresponding to the respective combinations of the number of propellers and shafts can be calculated by **equations (2.3.8)**[1)] and **(2.3.9)**[1)] based on the equations proposed by Fujii et al.[13)] in consideration of the hull wake and propeller slip effects.

(When 1 shaft and 1 propeller, or 2 shafts and 2 propellers)

$$\left.\begin{aligned} Y'_\delta &= -\frac{6.13\lambda_a}{(\lambda_a+2.25)}\frac{A_R}{L_{pp}d}(1+a_H)1.1 \\ N'_\delta &= -0.5Y'_\delta \end{aligned}\right\} \qquad \textbf{(2.3.8)}$$

(When 2 shafts and 1 propeller)

$$\left.\begin{aligned} Y'_\delta &= -\frac{6.13\lambda_a}{(\lambda_a+2.25)}\frac{A_R}{L_{pp}d}(1+a_H)0.7 \\ N'_\delta &= -0.5Y'_\delta \end{aligned}\right\} \qquad \textbf{(2.3.9)}$$

where

Y'_δ : dimensionless value of the coefficient Y_δ for the lateral force generated by a rudder at rudder angle;

N'_δ : dimensionless value of the coefficient N_δ for the rudder force moment generated by a rudder at rudder angle;

λ_a : effective aspect ratio of the rudder (a ratio b/c of a vertical length b and a lateral length c of the rudder);

A_R : rudder area (m^2), where A_R shall be doubled in the case of 2 rudders;

$A_R/(L_{pp}d)$: rudder area ratio;

α_H : interference coefficient of a rudder.

c) In many cases, the effective aspect ratios λ_a of rudders are set at 1.4 to 1.9.[10)]

d) When the rudder area ratios $A_R/(L_{pp}d)$ of the design ships are unknown, the following values can be used as references.

- High speed cargo ships: 1/35 to 1/40
- Conventional cargo ships: 1/45 to 1/60
- Tankers: 1/60 to 1/75

e) When the interference coefficients of the rudders α_H are unknown, the values estimated from the **Fig. 2.3.8** proposed by Kose et al.[14)] can be used as references.

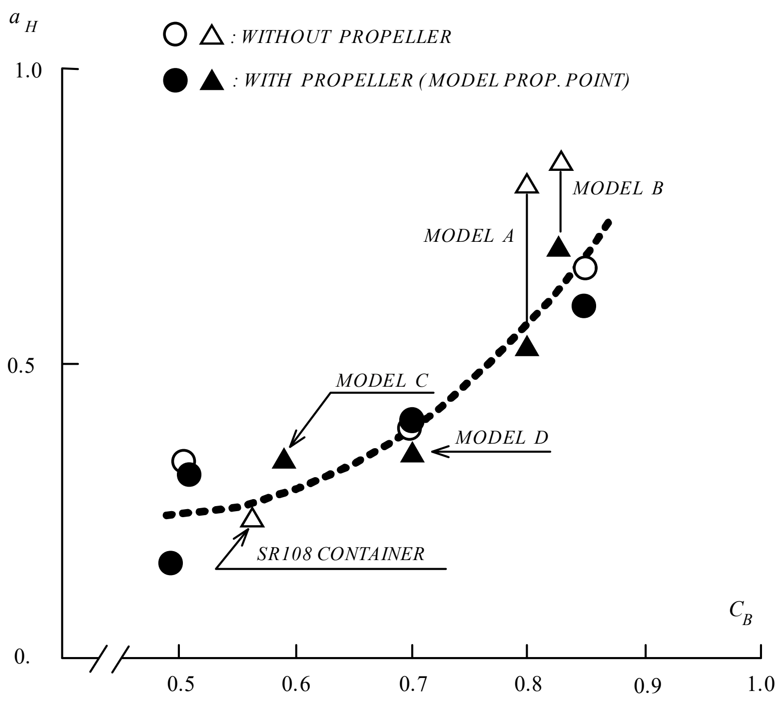


Fig. 2.3.8 Estimation of α_H Based on C_b[13)]

f) **Table 2.3.1** summarizes the dimensions of 22 types of ships and the calculation results of their coefficients of fluid dynamic forces (Y'_β, N'_β, Y'_δ, N'_δ). The coefficients of fluid dynamic forces calculated in the table are also required when calculating not only W_m, but also W_b, W_c and W_{ov}. Specific calculation examples are shown in the literature 15).

Table 2.3.1 Calculation Examples of Coefficients of Fluid Dynamic Forces by Ship Type (D/d_0 = 1.2)

	Ship type	*GT/DWT*	L_{oa} (m)	L_{pp} (m)	B (m)	d_0 (m)	C_b	Y'_β	N'_β	Y'_δ	N'_δ
1	Cargo ship	5,000 GT	109.0	103.0	20.0	7.00	0.7402	1.688	0.590	-0.0723	0.0362
2	Cargo ship (small size)	499 GT	63.8	60.4	11.2	4.20	0.5395	1.653	0.597	-0.0881	0.0441
3	Container ship (14,000 TEU)	150,166 DWT	368.8	352.0	51.0	15.50	0.6845	1.253	0.419	-0.0526	0.0263
4	Container ship (10,000 TEU)	99,563 DWT	336.0	318.3	45.8	14.04	0.6437	1.252	0.416	-0.0691	0.0345
5	Container ship (6,000 TEU OVER PANAMAX)	77,900 DWT	299.9	283.8	40.0	14.00	0.6472	1.340	0.457	-0.0720	0.0360
6	Container ship (4,000 TEU PANAMAX)	59,500 DWT	288.3	273.0	32.2	13.25	0.6665	1.312	0.449	-0.0781	0.0391
7	Bulk carrier (VLOC)	297,736 DWT	327.0	318.0	55.0	21.40	0.8698	1.689	0.585	-0.0730	0.0365
8	Bulk carrier (CAPESIZE)	172,900 DWT	289.0	279.0	45.0	17.81	0.8042	1.612	0.562	-0.0699	0.0350
9	Bulk carrier (NEW PANAMAX)	98,681 DWT	240.0	236.0	38.0	14.48	0.8528	1.591	0.543	-0.0794	0.0397
10	Bulk carrier	74,000 DWT	225.0	216.0	32.3	13.50	0.8383	1.587	0.553	-0.0696	0.0348
11	Bulk carrier (small size)	10,000 DWT	125.0	119.2	21.5	6.90	0.8057	1.551	0.519	-0.0773	0.0387
12	Tanker (VLCC)	280,000 DWT	333.0	316.0	60.0	20.40	0.7941	1.658	0.564	-0.0880	0.0440
13	Tanker (small size)	6,000 DWT	100.6	92.0	20.0	7.00	0.7968	1.835	0.640	-0.0811	0.0406
14	Pure car carrier (PCC) (VLCC)	25,272 DWT	199.8	190.9	36.5	10.60	0.5866	1.466	0.504	-0.0671	0.0336
15	Pure car carrier (PCC) (large size)	21,500 DWT	199.9	190.0	32.2	10.06	0.6153	1.417	0.484	-0.0731	0.0365
16	Pure car carrier (PCC)	18,000 DWT	190.0	180.0	32.2	8.20	0.5470	1.287	0.427	-0.0753	0.0376
17	LNG ship	69,500 DWT	283.0	270.0	44.8	10.80	0.7000	1.213	0.382	-0.0762	0.0381
18	LPG ship	54,500 DWT	225.0	220.0	36.6	12.00	0.7422	1.469	0.495	-0.0741	0.0371
19	Refrigerated cargo carrier	10,000 GT	152.0	144.0	23.5	7.00	0.7526	1.372	0.451	-0.0705	0.0353
20	Passenger ship (large, 2 shafts, 2 propellers)	142,714 GT	330.0	306.0	38.4	8.30	0.6800	0.908	0.269	-0.0780	0.0390
21	Passenger ship (2 shafts, 2 propellers)	28,700 GT	192.8	160.0	24.7	6.60	0.6030	1.214	0.387	-0.1000	0.0500
22	Ferry boat (2 shafts, 1 propeller)	18,000 GT	192.9	181.0	29.4	6.70	0.5547	1.125	0.354	-0.0875	0.0437

iii. Calculation of the coefficients for wind pressure resistance and wind pressure moment

a) The coefficients for wind pressure resistance and wind pressure moment can be calculated by **equation (2.3.10)** proposed by Yamano et al.[16)]

$$\left.\begin{aligned} C_x &= C_{x_0} + C_{x_1}\cos\theta_w + C_{x_2}\cos 2\theta_w + C_{x_3}\cos 3\theta_w + C_{x_4}\cos 4\theta_w + C_{x_5}\cos 5\theta_w \\ C_y &= C_{y_1}\sin\theta_w + C_{y_2}\sin 2\theta_w + C_{y_3}\sin 3\theta_w \\ C_m &= 0.1\left(C_{m_1}\sin\theta_w + C_{m_2}\sin 2\theta_w + C_{m_3}\sin 3\theta_w\right) \end{aligned}\right\} \quad \textbf{(2.3.10)}$$

where

C_x : coefficient for longitudinal wind pressure resistance;

C_y : coefficient for lateral wind pressure resistance;

C_m : coefficient for wind pressure moment around the central axis of the hull;

θ_w : wind direction with respect to the bow (°).

Each coefficient is the sum of A_y/L^2, X_g/L, L/B and A_y/A_x multiplied by the respective coefficients in **Table 2.3.2**. For example, using the coefficients in the table, C_{x0} can be calculated by $C_{x0} = -0.0358 + 0.925\ A_y/L^2 + 0.0521\ X_g/L$. In the table, the meanings of the respective symbols are as follows:

L : length between perpendiculars (= L_{pp}) (m);

A_x : projected front area above the water line (m^2);

A_y : projected lateral area above the water line (m^2);

X_g : distance between the centroid of A_y and F.P. (m).

(F.P.: fore perpendicular [refer to **Part II, Chapter 8, 1 Main Dimensions of Design Ships**])

Table 2.3.2 Regression Coefficients (When $L = L_{pp}$)

C_x	*Const.*	A_y/L^2	X_g/L	L/B	A_y/A_x
C_{x0}	-0.0358	0.925	0.0521		
C_{x1}	2.58	-6.087		-0.1735	
C_{x2}	-0.97		0.978	0.0556	
C_{x3}	-0.146			-0.0283	0.0728
C_{x4}	0.0851			-0.0254	0.0212
C_{x5}	0.0318	0.287		-0.0164	
Cy	*Const.*	A_y/L^2	X_g/L	L/B	A_y/A_x
C_{y1}	0.509	4.904			0.022
C_{y2}	0.0208	0.230	-0.075		
C_{y3}	-0.357	0.943		0.0381	
C_m	*Const.*	A_y/L^2	X_g/L	L/B	A_y/A_x
C_{m1}	2.650	4.634	-5.876		
C_{m2}	0.105	5.306			0.0704
C_{m3}	0.616		-1.474	0.0161	

b) A_x and A_y can be calculated by **equation (2.3.11)** proposed by Akakura and Takahashi.[17)]

$$\log(Y) = \alpha_w + \beta_w \log(X) \tag{2.3.11}$$

where

Y : A_x or A_y;

X : DWT or GT of the design ship;

α_w, β_w : coefficients (refer to **Table 2.3.3**).

Table 2.3.3 Coefficients to Estimate A_x and A_y

Fully Loaded Condition

	Object unit	Coefficient when Y is A_x				Coefficient when Y is A_y			
		α_w	β_w	R^2	σ	α_w	β_w	R^2	σ
Cargo ship	DWT	-0.228	0.666	0.929	0.0451	0.507	0.616	0.824	0.1302
Bulk carrier	DWT	0.944	0.370	0.823	0.0497	1.218	0.425	0.841	0.0729
Container ship	DWT	0.136	0.609	0.812	0.0598	0.417	0.703	0.949	0.0675
Tanker	DWT	0.469	0.474	0.901	0.0625	0.556	0.558	0.931	0.0708
RORO ship	DWT	1.029	0.435	0.901	0.0469	1.453	0.464	0.719	0.1453
Passenger ship	GT	0.947	0.426	0.956	0.0715	0.059	0.680	0.998	0.0552
Ferry	GT	0.728	0.473	0.891	0.0578	0.564	0.674	0.974	0.0391
Gas carrier	GT	0.423	0.553	0.960	0.0593	0.705	0.613	0.939	0.0706

Ballast Condition

	Object unit	Coefficient when Y is A_x				Coefficient when Y is A_y			
		α_w	β_w	R^2	σ	α_w	β_w	R^2	σ
Cargo ship	DWT	0.099	0.615	0.935	0.0365	0.479	0.662	0.906	0.1007
Bulk carrier	DWT	0.629	0.469	0.935	0.0376	0.970	0.530	0.956	0.0460
Container ship	DWT	0.574	0.526	0.696	0.0741	0.731	0.625	0.819	0.1016
Tanker	DWT	0.251	0.551	0.962	0.0408	0.650	0.592	0.984	0.0333
RORO ship	DWT	0.917	0.473	0.910	0.0453	1.541	0.456	0.792	0.1123
Passenger ship	GT	0.986	0.419	0.953	0.0746	0.656	0.666	0.996	0.0466
Ferry	GT	0.710	0.484	0.901	0.0557	0.569	0.679	0.976	0.0377
Gas carrier	GT	0.503	0.547	0.980	0.0468	0.828	0.604	0.976	0.0420

c) When the distances X_G between the centroids of the lateral areas and F.P. are unknown, X_G can be calculated with reference to the average (0.517) and deviation (0.032) of X_G/L_{pp} in the literature 15).

iv. Calculation of rudder angles and drift angles β_1 based on the motion equation in an equilibrium situation

a) The motion equation of ships in an equilibrium situation under steady winds with rudder angles δ and drift angles β can be expressed by **equation (2.3.12)**.

$$\left.\begin{aligned} &Y_\beta \beta + Y_\delta \delta + C_y \left(\frac{\rho_a}{\rho_w}\right)\left(\frac{A_y}{Ld}\right)\left(\frac{U_a}{U}\right)^2 = 0 \\ &N_\beta \beta + N_\delta \delta + C_m \left(\frac{\rho_a}{\rho_w}\right)\left(\frac{A_y}{Ld}\right)\left(\frac{U_a}{U}\right)^2 = 0 \end{aligned}\right\} \qquad \textbf{(2.3.12)}$$

where

Y_β : coefficient for the reaction force in a transverse direction from a fluid when a ship diagonally navigates with a drift angle β;

N_β : coefficient for the turning reaction moment from a fluid when a ship diagonally navigates with a drift angle β;

Y_δ : coefficient for the lateral force generated by a rudder at rudder angle δ;

N_δ : coefficient for the rudder force moment generated by a rudder at rudder angle δ;

U : ship speed (m/s);

U_a : wind speed (m/s);

ρ_w : density of seawater (t/m^3);

ρ_a : density of air (t/m^3);

d : maximum draft of a moored design ship in still water (m);

L : length between perpendiculars (= L_{pp}) (m);

A_y : projected lateral area above the water line (m^2);

C_y : coefficient for lateral wind pressure resistance;

C_m : coefficient for wind pressure moment around the central axis of the hull.

b) Based on the equation above, the rudder and drift angles can be calculated by **equation (2.3.13)**.

$$
\left.
\begin{aligned}
&\text{Rudder angle: } \delta = -\left(\frac{\rho_a}{\rho_w}\right)\left(\frac{U_a}{U}\right)^2\left(\frac{A_y}{Ld}\right)\left(\frac{C_m Y'_\beta - C_y N'_\beta}{Y'_\beta N'_\delta - Y'_\delta N'_\beta}\right) \\
&\text{Drift angle: } \beta = \left(\frac{\rho_a}{\rho_w}\right)\left(\frac{U_a}{U}\right)^2\left(\frac{A_y}{Ld}\right)\left(\frac{C_m Y'_\delta - C_y N'_\delta}{Y'_\beta N'_\delta - Y'_\delta N'_\beta}\right) \\
&\text{provided that} \\
&\quad Y'_\beta = Y_\beta \big/ (0.5\rho_w LdU^2) \\
&\quad N'_\beta = N_\beta \big/ (0.5\rho_w L^2 dU^2) \\
&\quad Y'_\delta = Y_\delta \big/ (0.5\rho_w LdU^2) \\
&\quad N'_\delta = N_\delta \big/ (0.5\rho_w L^2 dU^2)
\end{aligned}
\right\} \qquad \textbf{(2.3.13)}
$$

where

Y'_β : dimensionless value of the coefficient Y_β for the reaction force in a transverse direction from a fluid when a ship diagonally navigates with a drift angle β;

N'_β : dimensionless value of the coefficient N_β for the turning reaction moment from a fluid when a ship diagonally navigates with a drift angle β;

Y'_δ : dimensionless value of the coefficient Y_δ for the lateral force generated by a rudder at rudder angle δ;

N'_δ : dimensionless value of the coefficient N_δ for the rudder force moment generated by a rudder at rudder angle δ;

d : maximum draft of a moored design ship in still water (m);

L : length between perpendiculars (= L_{pp}) (m);

A_y : lateral projected area above the water line (m^2);

C_y : coefficient for lateral wind pressure resistance;

C_m : coefficient for wind pressure moment around the central axis of the hull;

ρ_w : density of seawater (t/m^3);

ρ_a : the density of air (t/m^3).

c) **Table 2.3.4** shows the calculation results of the required check rudder angles and drift angles β_1 for the types of ships listed in **Table 2.3.1** in a case where the ratio of wind speeds to ship speeds U_a/U (called as the "K value") with respect to the respective wind directions is 1.0. Here, the wind direction angles are measured from the bows of the respective ships. As is evident in **equation (2.3.13)**, the required check rudder and drift angles in cases where K values are other than 1 can be calculated by multiplying the require check rudder and drift angles when the *K* value is 1 by the square of the *K* values. As already mentioned in "Fundamentals of Performance Verification," the check rudder angles to compensate for the effect of winds shall be up to 15°. In cases where the check rudder angles exceed 15°, it is necessary to reassess the setting of maximum wind speed as one of the port call conditions, the normal direction of navigation channels, etc. The calculation results of the required check rudder and drift angles when *K* values are 1 to 7 are shown in **Reference (Part III), Chapter 4, 3 Required Check Rudder Angles and Drift Angles by Ship Type and Wind Direction Angle When *K* Values Are 1 to 7**. Cases with check rudder angles larger than 30° are eliminated from the calculation results. The values in **Table 2.3.1** can be used as references when calculating the approximate values of the required widths to cope with the effects of winds in cases where the types and sizes of design ships are equivalent to those listed in **Table 2.3.1**.

Table 2.3.4 Required Check Rudder and Drift Angles by Ship Type and Wind Direction Angle (D/d = 1.2)

Ship type			Ratio of wind speed to ship speed	Wind direction angle (°)												
				0	15	30	45	60	75	90	105	120	135	150	165	180
1	Cargo ship	Check rudder angle (°)	K = 1	0.000	0.017	0.049	0.102	0.169	0.233	0.276	0.284	0.257	0.204	0.138	0.068	0.001
		Drift angle (°)	K = 1	0.000	0.003	0.007	0.011	0.014	0.017	0.017	0.015	0.011	0.007	0.003	0.001	0.000
2	Cargo ship (small size)	Check rudder angle (°)	K = 1	0.000	0.028	0.069	0.128	0.199	0.267	0.313	0.325	0.300	0.245	0.170	0.087	0.001
		Drift angle (°)	K = 1	0.000	0.006	0.011	0.017	0.021	0.024	0.024	0.021	0.016	0.011	0.006	0.003	0.000
3	Container ship (14,000 TEU)	Check rudder angle (°)	K = 1	0.000	0.191	0.406	0.660	0.952	1.258	1.531	1.711	1.738	1.567	1.191	0.644	0.000
		Drift angle (°)	K = 1	0.000	0.040	0.074	0.099	0.113	0.116	0.109	0.095	0.077	0.058	0.039	0.019	0.000
4	Container ship (10,000 TEU)	Check rudder angle (°)	K = 1	0.000	0.113	0.245	0.406	0.593	0.787	0.956	1.063	1.070	0.958	0.723	0.389	0.000
		Drift angle (°)	K = 1	0.000	0.030	0.057	0.077	0.088	0.091	0.086	0.075	0.061	0.045	0.029	0.014	0.000
5	Container ship (6,000 TEU OVER PANAMAX)	Check rudder angle (°)	K = 1	0.000	0.082	0.178	0.293	0.425	0.559	0.671	0.736	0.732	0.648	0.485	0.261	0.002
		Drift angle (°)	K = 1	0.000	0.019	0.036	0.049	0.056	0.059	0.056	0.049	0.040	0.029	0.019	0.009	0.000
6	Container ship (4,000 TEU PANAMAX)	Check rudder angle (°)	K = 1	0.000	0.070	0.143	0.220	0.303	0.387	0.461	0.510	0.517	0.468	0.357	0.195	0.002
		Drift angle (°)	K = 1	0.000	0.015	0.029	0.038	0.042	0.043	0.040	0.036	0.030	0.023	0.016	0.008	0.000
7	Bulk carrier (VLOC)	Check rudder angle (°)	K = 1	0.000	0.012	0.035	0.072	0.119	0.164	0.195	0.202	0.184	0.147	0.100	0.050	0.000
		Drift angle (°)	K = 1	0.000	0.002	0.005	0.008	0.010	0.012	0.012	0.010	0.008	0.005	0.002	0.001	0.000
8	Bulk carrier (CAPESIZE)	Check rudder angle (°)	K = 1	0.000	0.015	0.039	0.077	0.124	0.169	0.199	0.206	0.189	0.153	0.105	0.053	0.000
		Drift angle (°)	K = 1	0.000	0.002	0.005	0.008	0.010	0.012	0.012	0.010	0.008	0.005	0.003	0.001	0.000
9	Bulk carrier (NEW PANAMAX)	Check rudder angle (°)	K = 1	0.000	0.014	0.036	0.070	0.112	0.153	0.180	0.186	0.170	0.137	0.094	0.047	0.000
		Drift angle (°)	K = 1	0.000	0.003	0.005	0.008	0.011	0.012	0.012	0.011	0.008	0.005	0.003	0.001	0.000
10	Bulk carrier	Check rudder angle (°)	K = 1	0.000	0.015	0.036	0.067	0.104	0.139	0.162	0.167	0.153	0.124	0.085	0.043	0.000
		Drift angle (°)	K = 1	0.000	0.002	0.004	0.006	0.008	0.009	0.009	0.008	0.006	0.004	0.002	0.001	0.000
11	Bulk carrier (small size)	Check rudder angle (°)	K = 1	0.000	0.027	0.070	0.135	0.217	0.296	0.351	0.367	0.340	0.278	0.194	0.099	0.001
		Drift angle (°)	K = 1	0.000	0.006	0.012	0.018	0.024	0.027	0.026	0.023	0.018	0.012	0.006	0.003	0.000
12	Tanker (VLCC)	Check rudder angle (°)	K = 1	0.000	0.008	0.027	0.059	0.102	0.143	0.170	0.174	0.157	0.123	0.082	0.040	0.000
		Drift angle (°)	K = 1	0.000	0.002	0.005	0.008	0.011	0.013	0.013	0.011	0.008	0.005	0.002	0.001	0.000
13	Tanker (small size)	Check rudder angle (°)	K = 1	0.000	0.015	0.044	0.095	0.160	0.223	0.264	0.272	0.245	0.193	0.129	0.064	0.001
		Drift angle (°)	K = 1	0.000	0.003	0.007	0.011	0.014	0.017	0.017	0.015	0.011	0.007	0.003	0.001	0.000
14	Pure car carrier (PCC) (VLCC)	Check rudder angle (°)	K = 1	0.000	0.264	0.550	0.873	1.234	1.611	1.954	2.189	2.235	2.029	1.551	0.842	0.000
		Drift angle (°)	K = 1	0.000	0.059	0.110	0.147	0.168	0.172	0.161	0.141	0.114	0.085	0.056	0.028	0.000
15	Pure car carrier (PCC) (large size)	Check rudder angle (°)	K = 1	0.000	0.159	0.340	0.556	0.806	1.067	1.298	1.450	1.470	1.324	1.006	0.546	0.005
		Drift angle (°)	K = 1	0.000	0.041	0.076	0.103	0.118	0.122	0.115	0.100	0.080	0.059	0.038	0.019	0.000
16	Pure car carrier (PCC)	Check rudder angle (°)	K = 1	0.000	0.161	0.353	0.593	0.877	1.176	1.440	1.609	1.626	1.458	1.104	0.598	0.006
		Drift angle (°)	K = 1	0.000	0.051	0.097	0.132	0.152	0.158	0.149	0.130	0.104	0.076	0.048	0.024	0.000
17	LNG ship	Check rudder angle (°)	K = 1	0.000	0.092	0.211	0.374	0.573	0.780	0.952	1.049	1.040	0.914	0.680	0.364	0.003
		Drift angle (°)	K = 1	0.000	0.033	0.063	0.087	0.103	0.109	0.105	0.091	0.072	0.052	0.032	0.015	0.000

Ship type			Ratio of wind speed to ship speed	Wind direction angle (°)												
				0	15	30	45	60	75	90	105	120	135	150	165	180
18	LPG ship	Check rudder angle (°)	$K = 1$	0.000	0.053	0.125	0.223	0.341	0.459	0.550	0.591	0.570	0.487	0.353	0.185	0.000
		Drift angle (°)	$K = 1$	0.000	0.012	0.024	0.034	0.041	0.044	0.043	0.037	0.029	0.020	0.012	0.005	0.000
19	Refrigerated cargo carrier	Check rudder angle (°)	$K = 1$	0.000	0.036	0.089	0.164	0.255	0.342	0.405	0.425	0.397	0.328	0.231	0.119	0.001
		Drift angle (°)	$K = 1$	0.000	0.008	0.015	0.023	0.028	0.032	0.031	0.028	0.022	0.015	0.008	0.004	0.000
20	Passenger ship (large, 2 shafts, 2 propellers)	Check rudder angle (°)	$K = 1$	0.000	0.298	0.600	0.920	1.285	1.706	2.154	2.540	2.729	2.588	2.044	1.132	0.000
		Drift angle (°)	$K = 1$	0.000	0.151	0.276	0.358	0.392	0.386	0.354	0.308	0.257	0.202	0.141	0.073	0.000
21	Passenger ship (2 shafts, 2 propellers)	Check rudder angle (°)	$K = 1$	0.000	0.174	0.363	0.578	0.826	1.097	1.361	1.561	1.629	1.507	1.169	0.643	0.006
		Drift angle (°)	$K = 1$	0.000	0.082	0.152	0.201	0.226	0.227	0.212	0.185	0.151	0.115	0.078	0.039	0.000
22	Ferry boat (2 shafts, 1 propeller)	Check rudder angle (°)	$K = 1$	0.000	0.113	0.253	0.438	0.662	0.900	1.111	1.244	1.257	1.126	0.851	0.460	0.004
		Drift angle (°)	$K = 1$	0.000	0.053	0.100	0.136	0.158	0.164	0.155	0.135	0.108	0.078	0.050	0.024	0.000

v. Calculation of drift angles β_2 due to the effects of tidal currents

Drift angles can be calculated by **equation (2.3.14)** using ship speeds and the abeam components of tidal constituent speeds.

$$\beta_2 = \arctan(U_c/U) \tag{2.3.14}$$

where

β_2 : drift angle due to the effects of tidal currents (°);

U : ship speed (m/s);

U_c : abeam component of the tidal constituent speed with respect to the centerline of the navigation channel (m/s).

vi. Calculation of the required widths W_m (β) to account for wind forces and current forces

The required widths to account for wind forces and current forces can be calculated by **equation (2.3.15)** using the drift angles β to compensate for these forces, and are obtainable by combining the drift angles β_1 and β_2 to compensate for the effects of winds and tidal currents, respectively.

$$\left.\begin{aligned} &\beta = \beta_1 + \beta_2 \\ &W(\beta) = L_{oa} \sin\beta + B\cos\beta \end{aligned}\right\} \tag{2.3.15}$$

where

$W(\beta)$: required widths to account for wind forces and current forces (m);

L_{oa} : length overall of the ship (m);

B : molded breadth (m);

B : drift angle to compensate for the effects of winds and tidal currents (°);

β_1 : drift angle to compensate for the effect of winds (°);

β_2 : drift angle to compensate for the effect of tidal currents (°).

vii. Calculation of the required widths $W(y)$ to account for yawing motion

a) The required widths $W(y)$ to account for yawing motion can be calculated by **equation (2.3.16)** using the maximum meandering amount (either side).

$$W(y) = U\int_0^{T_y/4} \sin\varphi(t)dt = \frac{1}{4}UT_y \sin\varphi_0 \tag{2.3.16}$$

where

$W(y)$: required widths to account for yawing motion (m);

U : ship speed (m/s);

T_y : yawing period (s);

φ_0 : maximum yawing angle (°);

$\varphi(t)$: yawing amount at clock time t, $\varphi(t) = \varphi_0 \sin(2\pi t/T_y)$ (m).

b) When yawing periods T_y and the maximum yawing angle φ_0 are unknown, T_y = 12s and φ_0 = 4° can be used as values on the safe side.

viii. Calculation of the required widths $W_m(\beta, y)$ to account for wind forces, current forces and yawing motion

The required widths $W_m(\beta, y)$ to account for wind forces, current forces and yawing motion can be calculated by **equation (2.3.17)**.

$$W_m(\beta, y) = W(\beta) + 2W(y) = L_{oa}\sin\beta + B\cos\beta + 0.5UT_y\sin\phi_0 \quad \textbf{(2.3.17)}$$

where

$W_m(\beta, y)$: required width to account for wind forces, current forces and yawing motion (m);

$W(\beta)$: required width to account for wind forces and current forces (m);

$W(y)$: required width to account for yawing motion (m).

(b) The required widths $W_m(S)$ to account for drift detection can be calculated by the following procedure.

1) Basic concept of calculation

i. In general, a ship sailing in the fairway more or less makes some amount of lateral deviation from its course line even if the ship handler does believe that his ship is running on the right course line. This drift may hardly be detected within small amount of deviation. However, the ship handler can recognize the drift when the lateral deviation from the fairway center line becomes considerable amount. The required widths $W_m(S)$ are to enable navigators to recognize a lateral drift of ships through a deviation from the predetermined courses.

ii. Estimations of the required width $W_m(S)$ for the drift detection are provided for the following four types of on-board navigation equipment, which are currently available in the actual ship operation.

a) Recognition of lateral drift through the visual identification of buoys on both sides of the navigation channels $W_m(S) = W_m(\alpha)$

b) Recognition of lateral drift through the identification of buoys on both sides of the navigation channels by radar $W_m(S) = W_m(R)$

c) Recognition of lateral drift using GPS $W_m(S) = W_m(GPS)$ or $W_m(D \cdot GPS)$

d) Recognition of lateral drift using guide marks (lights) $W_m(S) = W_m(L)$

(2.3.18)

The appropriate methods for drift detection shall be selected depending on the design ships and navigation channels.

2) Standard calculation methods and calculation equations

i. Calculation of the required width $W_m(a)$ for drift detection by observing light buoys on both sides of the navigation channels with naked eyes

a) The required width $W_m(a)$ for drift detection by observing light buoys on both sides of the navigation channels with naked eye can be calculated by **equation (2.3.19)** proposed by the West Japan Society of Naval Architects.[18)]

$$\left.\begin{aligned}
&\theta = 2\arctan\left(\frac{W_{buoy}}{2LF}\right)\\
&\alpha_r = 0.00044\theta^2 + 0.0002\theta + 0.55343\\
&\alpha_{\max} = 4\alpha_r\\
&W_m(\alpha) = LF\tan(\alpha_{\max})
\end{aligned}\right\} \quad \textbf{(2.3.19)}$$

where

θ : angle between the lines connecting the ship with anterior buoys on both sides of the navigation channel (°);

W_{buoy} : distance between anterior buoys on both sides of the navigation channel (m);

LF : distance between the ship and the anterior buoy (m);

$W_m(a)$: required width for drift detection by observing light buoys on both sides of the navigation channels with naked eye (m);

α_r : observation error of middle point (°);

α_{max} : maximum observation error of middle point (the maximum error with which 99.8% of navigators can recognize lateral deflection) (°).

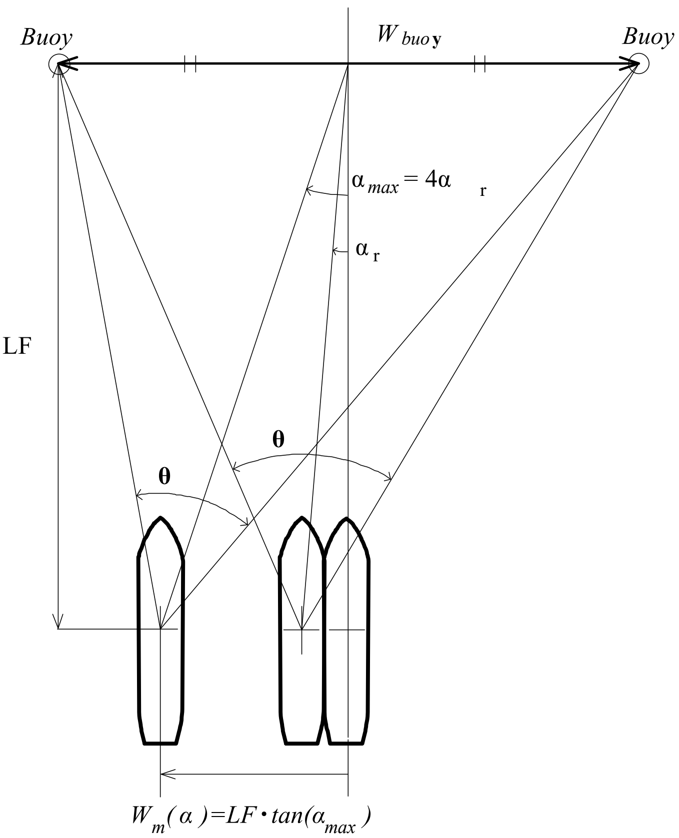


Fig. 2.3.9 Concept of the Required Molded Breadth $W_m(a)$ for the Recognition of Lateral Deflection

b) The distances LF from the ships to the anterior buoys can be set as shown below based on the concept shown in the literature 1).

One-way navigation channel $LF = 7L_{oa}$ **(2.3.20)**

Two-way navigation channel $LF = 3.5L_{oa} \sim 7L_{oa}$ **(2.3.21)**

where

L_{oa} : length overall.

c) For existing navigation channels, LF can be the distances between anterior buoys in the case of one-way navigation channels and 0.5 to 1.0 times the distances between anterior buoys in the case of two-way navigation channels.

ii. Calculation of the required widths $W_m(R)$ for drift detection by observing light buoys on both sides of the navigation channels with radar

a) The required widths $W_m(R)$ for drift detection by observing light buoys on both sides of the navigation channels with radar can be calculated by **equations (2.3.22)** and **(2.3.23)** proposed by the West Japan Navigation Technology Study Group.[18]

The maximum deflection error in the lateral direction when confirming the positions of ships through cross bearings by measuring the buoys on both sides of the ships by radar can be expressed by the following equation, provided that the orientation error of radar observation is 2°.

$$\text{Maximum drift error in the lateral direction} = \frac{W_{buoy}\sin 2^\circ}{\sin\theta} \tag{2.3.22}$$

Therefore,

$$W_m(R) = 0.0349\frac{W_{buoy}}{\sin\theta} \quad \text{(When the measurement error is 2°)} \tag{2.3.23}$$

where

$W_m(R)$: required width for drift detection by observing light buoys on both sides of the navigation channels with radar (m);

W_{buoy} : distance between anterior buoys (m);

θ : angle between the lines connecting the ship with the anterior buoys on both sides of the navigation channel (°);

where

$$\theta = 2\arctan\left(\frac{W_{buoy}}{2LF}\right) \tag{2.3.24}$$

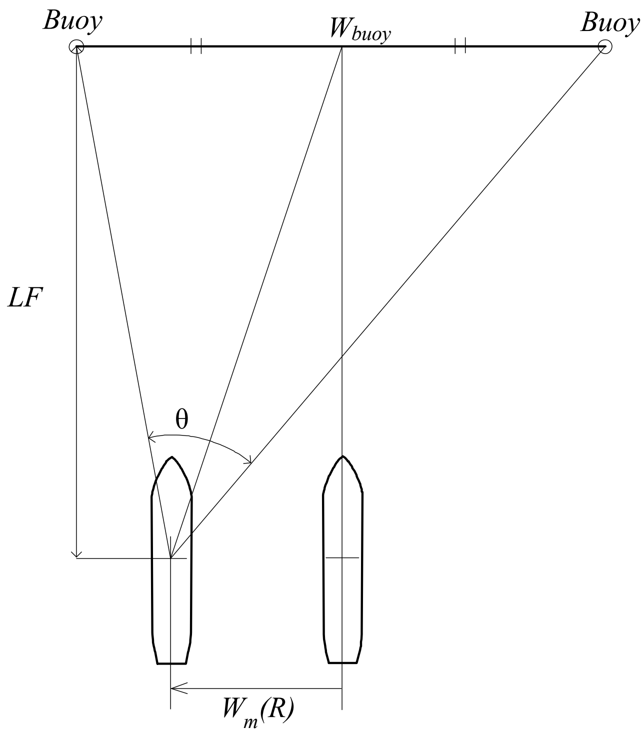


Fig. 2.3.10 Concept of the Required Molded Breadth $W_m(R)$ for the Drift Detection

b) It has been said that the measurement error of radar observation has been reduced to 1° due to the development of radar observation technologies since the West Japan Navigation Technology Study Group first proposed the equations in 1977.

Thus, for design ships equipped with accurate radar observation devices (with an orientation error of 1°), the required widths for the recognition of lateral drift can be calculated by **equation (2.3.25)**.

$$W_m(R) = 0.0175\frac{W_{buoy}}{\sin\theta} \quad \text{(When the measurement error is 1°)} \qquad \textbf{(2.3.25)}$$

c) As is the case with $W_m(a)$, the distance LF from ships to anterior buoys can be set as shown below based on the concept shown in **Reference 1)**.

One-way navigation channel $LF = 7L_{oa}$ **(2.3.26)**

Two-way navigation channel $LF = 3.5L_{oa} \sim 7L_{oa}$ **(2.3.27)**

where

L_{oa} : length overall.

d) For existing navigation channels, LF can be the distances between anterior buoys in the case of one-way navigation channels and 0.5 to 1.0 times the distances between anterior buoys in the case of two-way navigation channels.

iii. Calculation of the required widths $W_m(GPS)$ and $W_m(D{\cdot}GPS)$ for drift detection by GPS

a) The maximum measurement error of a GPS (single GPS) shall be set at 28 m as observed by the Japan Coast Guard.[19] For the maximum measurement error of a Differential GPS (D·GPS), the following values can be used based on the correction information by the Japan Coast Guard GPS Center.

- Error for single GPS = 30 m

- Error for D·GPS = 1 m

b) The use of GPS information in positioning is based on the assumption that electronic nautical charts accurately show measurement results. Considering that navigators actually determine ship positions based on the images of GPS information shown on displays, $W_m(GPS)$ and $W_m(D{\cdot}GPS)$ can be set as follows based on the display image magnification ratios of the ECDIS (Electronic Chart Display and Information System). For the calculation of $W_m(D{\cdot}GPS)$, the error of 1 m for D·GPS measurements can be ignored.

$$\left.\begin{array}{ll} \text{(For single GPS)} & : W_m\left(GPS\right) = 0.5B + 30 \quad (\mathrm{m}) \\ \text{(For D·GPS)} & : W_m\left(D\cdot GPS\right) = 0.5\mathrm{B} \quad (\mathrm{m}) \end{array}\right\} \qquad \textbf{(2.3.28)}$$

where

B : molded breadth (m).

iv. Calculation of the required widths $W_m(L)$ for drift detection by guide marks (lights)

a) Following the method adopted by the Hydrographic Service of the Royal Netherlands Navy, the required widths $W_m(L)$ for drift detection by guide marks (lights) can be calculated based on vertical angles θ_v and horizontal angles θ_h.

b) Calculation of vertical angles θ_v

The vertical angle θ_v between the lines connecting the navigator of the ship with the anterior and posterior navigation marks (lights) can be calculated by **equations (2.3.29)** and **(2.3.30)**.

$$\theta_v = \theta_{v1} - \theta_{v2} - \theta_{v3} \qquad \textbf{(2.3.29)}$$

$$\left.\begin{array}{l} \theta_{v1} = \arctan\left(\dfrac{H_H - H_L}{L_H}\right) \\ \theta_{v2} = \arctan\left(\dfrac{H_L - H_h}{L_L}\right) - \arctan\left(\dfrac{1\ H_L - H_h}{L_H}\right) \\ \theta_{v3} = \arctan\left(\dfrac{(1-K)L_D}{2R}\right) = 2.27\cdot 10^{-4} L_D \end{array}\right\} \qquad \textbf{(2.3.30)}$$

(Terms to correct the effects of the curvature of the Earth, where R: the radius of the Earth [6,360 km] and K: the refraction coefficient [0.16])

where

θ_v : vertical angle between the lines connecting the navigator of the ship with the anterior and posterior navigation marks (lights) (minutes);

H_H : height of the posterior guide mark (light) (m);

H_L : height of the anterior guide mark (light) (m);

H_h : eye height of the navigator on the ship (m);

L_H : distance between the ship and the posterior guide mark (light) (m);

L_L : distance between the ship and the anterior guide mark (light) (m);

L_D : distance between the posterior and anterior guide marks (lights) (= $L_H - L_L$) (m).

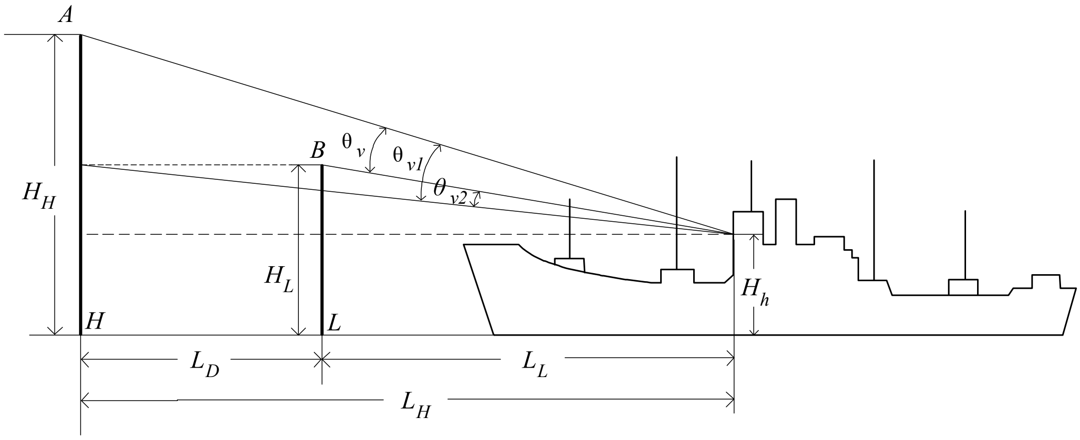


Fig. 2.3.11 Positional Relationship between the Design Ship and Guide Marks (Lights) (Elevation)

c) Calculation of vertical angles θ_h

The horizontal angles θ_h (minutes), with which navigators having healthy eyesight can recognize lateral deflection based on the breakup effects of the guide marks (lights), can be obtained from **Fig. 2.3.12** in relation to the calculated vertical angles θ_v (minutes).

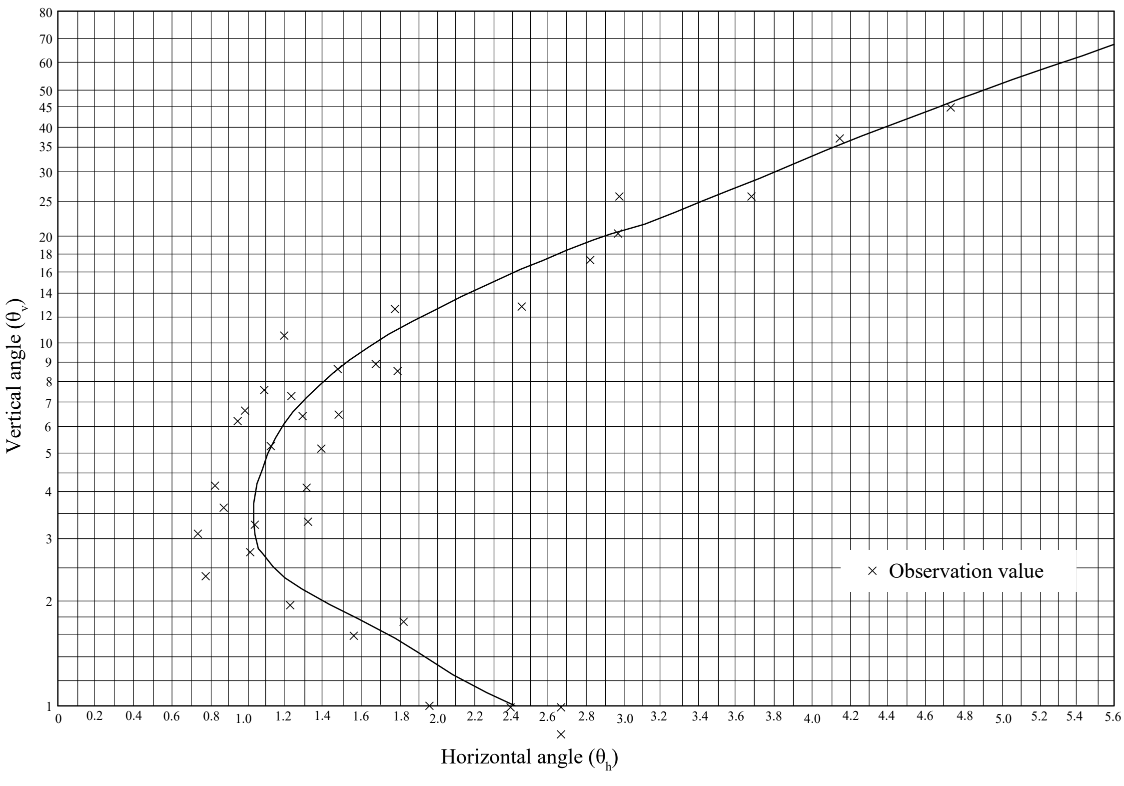


Fig. 2.3.12 Relationship Diagram between Vertical Angle (θ_v) and Horizontal Angle (θ_h) of Guide Marks (Lights)

d) Calculation of the required widths $W_m(L)$ for drift detection by guide marks (lights)

Based on the horizontal angles θ_h obtained from **Fig. 2.3.12**, the required widths $W_m(L)$ for drift detection by guide marks (lights) can be calculated by the following equation.

$$W_m(L) = \frac{L_H L_L \sin(\theta_h)}{L_D} \tag{2.3.31}$$

where

$W_m(L)$: required width for drift detection by guide marks (lights) (m);

L_H : distance between the ship and the posterior guide mark (light) (m);

L_L : distance between the ship and the anterior guide mark (light) (m);

L_D : distance between the posterior and anterior guide marks (lights) (= $L_H - L_L$) (m);

θ_h : horizontal angle with which a navigator having healthy eyesight can recognize lateral deflection on the basis of the breakup effects of the guide marks (lights) (minutes).

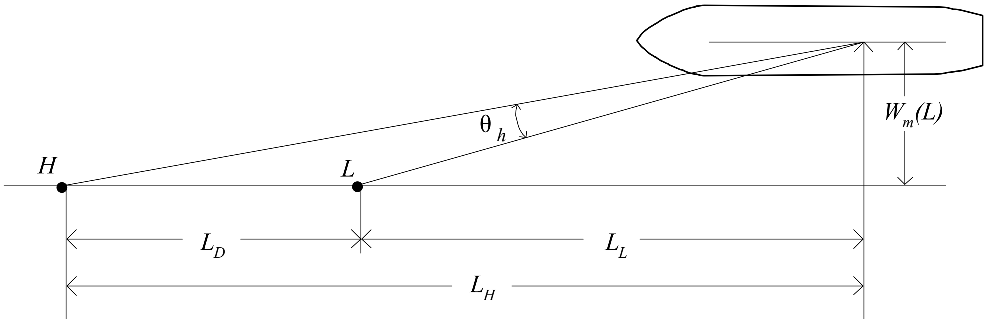


Fig. 2.3.13 Positional Relationship between the Design Ship and Guide Marks (Lights) (Plan)

② Widths W_{bi} considering an allowance for bank effect forces (the required widths to account for bank effect forces of navigation channels) can be calculated through the methods described in (a) and (b) 1) to 5) below.

(a) Basic concept of calculation

Because the bank effect forces of navigation channels are continuous, the required widths can be calculated as the distances from the side walls which enable ships to account for bank effect forces with a maximum check rudder angle of 5°.

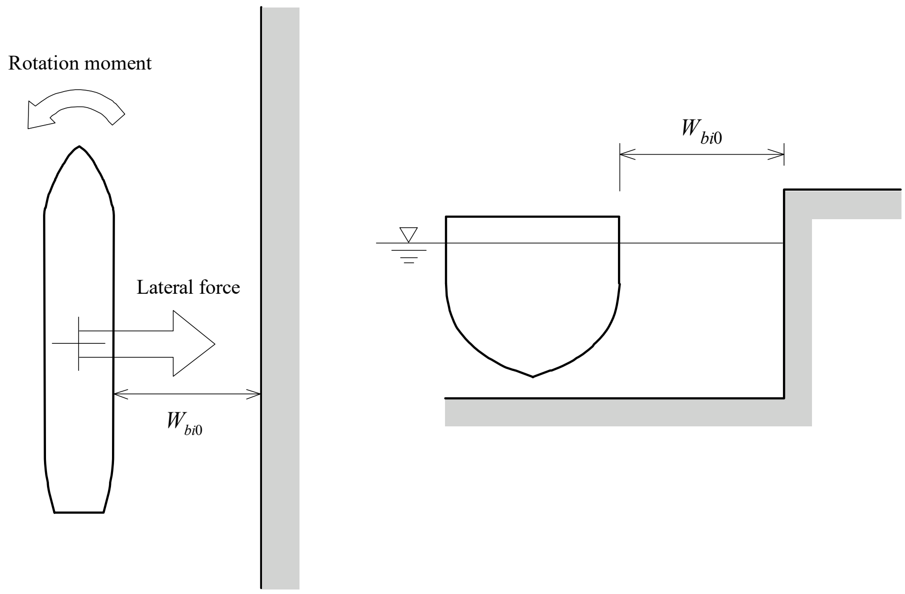


Fig. 2.3.14 Concept of Width to Account for Bank Effect Forces

(b) Standard calculation method and calculation equation

1) Calculation of the widths W_{bi} (subscript b: bank) to account for bank effect forces

The widths W_{bi} to account for bank effect forces can be calculated by the following phased calculation method.

Setting of the dimensions of design ships and navigation environment conditions

↓

Calculation of the lateral force and rotation moment acting on hulls due to vertical walls (either side)

↓

Calculation of rudder angles based on the motion equation in an equilibrium situation

↓

Calculation of widths to cope with the effects of side walls with a required rudder angle of 5°

↓

Correction of widths to cope with the effects of side walls based on the shapes of the side walls

Fig. 2.3.15 Calculation of Width to Account for Bank Effect Forces

2) Calculation of the lateral force and rotation moment acting on hulls due to bank walls (either side)

The values of the lateral force C_F and rotation moment C_M acting on the hulls of ships navigating close to bank walls can be obtained through **Fig. 2.3.16** proposed by Kijima et al.[20] according to S_p/L (= S_{pb}/L) values. In the figure, C_F (= C_{Fb}) and C_M (= C_{Mb}) are defined by **equation (2.3.32)**.

$$\left.\begin{aligned} C_{Fb} &= \frac{F_b}{0.5\rho_w L d U^2} \\ C_{Mb} &= \frac{M_b}{0.5\rho_w L^2 d U^2} \end{aligned}\right\} \qquad \textbf{(2.3.32)}$$

where

S_{pb} : distance from the centerline of the o the bank wall (S_p in **Fig. 2.3.16**) (m);

L : length between perpendiculars L_{pp} (m);

F_b : lateral force acting on the hull of a ship navigating close to a bank wall (kg·m/s^2);

C_{Fb} : dimensionless value of the lateral force acting on the hull of a ship navigating close to a bank wall;

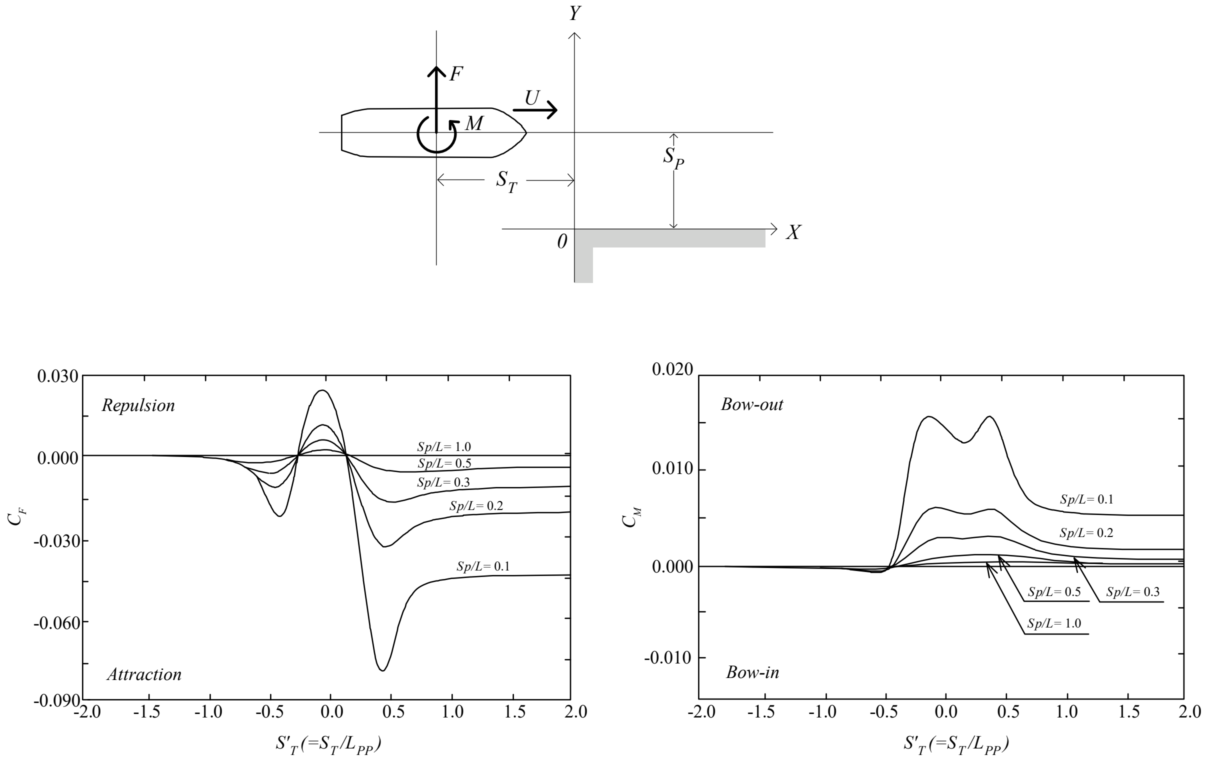
M_b : rotation moment acting on the hull of a ship navigating close to a bank wall (N·m);

C_{Mb} : dimensionless value of the rotation moment acting on the hull of a ship navigating close to a bank wall;

U : ship speed (m/s);

d : maximum draft of a moored design ship in still water (m);

ρ_w : density of seawater (kg/m^3).



(a) (b)

Fig. 2.3.16 Suction Force and Repulsive Moment[20] Acting on Ships Navigating Close to Bank Walls (in the figure, $S_p = S_{pb}$)

3) Calculation of the check rudder angle based on the motion equation in an equilibrium situation

The motion equation of ships navigating with rudder angles δ and drift angles β in an equilibrium situation can be expressed by **equation (2.3.33)** in the coordination system of **Fig. 2.3.16**.

$$\left.\begin{aligned} -C_{Fb} + Y'_\beta \beta + Y'_\delta \delta = 0 \\ -C_{Mb} + N'_\beta \beta + N'_\delta \delta = 0 \end{aligned}\right\} \qquad \textbf{(2.3.33)}$$

where

C_{Fb} : dimensionless value of the lateral force acting on the hull of a ship navigating close to a bank wall;

C_{Mb} : dimensionless value of the rotation moment acting on the hull of a ship navigating close to a bank wall;

Y'_β : dimensionless value of the coefficient Y_β for the reaction force in a transverse direction from a fluid when a ship diagonally navigates with a drift angle β;

N'_β : dimensionless value of the coefficient N_β for the turning reaction moment from a fluid when a ship diagonally navigates with a drift angle β;

Y'_δ : dimensionless value of a transverse force coefficient Y_δ generated by a rudder set at rudder angle δ;

N'_δ : dimensionless value of the coefficient N_δ for a rudder force moment generated by a rudder at rudder angle δ.

Then, rudder angles δ and drift angles β can be calculated by **equation (2.3.34)** obtained by solving **equation (2.3.33)**.

$$\left.\begin{aligned}\delta &= \frac{C_{Mb}Y'_\beta - C_{Fb}N'_\beta}{Y'_\beta N'_\delta - Y'_\delta N'_\beta}\\ \beta &= -\frac{C_{Mb}Y'_\delta - C_{Fb}N'_\delta}{Y'_\beta N'_\delta - Y'_\delta N'_\beta}\end{aligned}\right\} \quad \textbf{(2.3.34)}$$

where

δ : rudder angle (rad);

β : drift angle (rad).

However, it is noted that the unit system of δ and β needs to be changed to degrees (°) for the following calculations.

Here, in **Fig. 2.3.16 (a)**, C_F (= C_{Fb}) values are –0.044, –0.021 and –0.012 when S_{pb}/L values are 0.1, 0.2 and 0.3, respectively, in the range of a steady situation ($S'_T = S_T/L > 1.5$). Also, in **Fig. 2.3.16 (b)**, C_M (= C_{Mb}) values are 0.0050, 0.0012 and 0.0002 when S_{pb}/L values are 0.1, 0.2 and 0.3, respectively.

4) Calculation of the widths W_{bi0} to account for bank effect forces when the check rudder angle is 5°

The widths W_{bi0} can be calculated in a manner that obtains a regression equation of S_{pb}/L with rudder angles δ as a variable from the calculation results of δ corresponding to S_{pb}/L, calculates S_{pb}/L values by substituting δ = 5° into the regression equation, and calculates W_{bi0} by substituting the obtained S_{pb}/L value into **equation (2.3.35)**. However, in case the calculated rudder angles with S_{pb}/L exceeds 30° which is unrealistically large, the value of S_{pb}/L shall be ignored for developing the regression equation.

$$W_{bi0} = S_{pb} - 0.5B \quad \textbf{(2.3.35)}$$

where

W_{bi0} : width to account for bank effect forcess (m);

S_{pb} : distance from the centerline of a ship to the side wall (m);

B : molded breadth of the design ship (m).

The calculation results of the widths to account for bank effect forces for the type of ships listed in **Table 2.3.1** are shown in **Table 2.3.5**, and the values in the table can be used as approximate values of W_{bi0}. It shall be noted that W_{bi0} values are not affected by ship speeds.

Table 2.3.5 W_{bi0} by Ship Type

(Required Width to Cope with the Effects of Vertical Side Walls with a Check Rudder Angle of 5° and D/d = 1.2)

	Ship type	L_{pp}	B	S_{pb}/L_{pp}	S_{pb}	W_{bi0}	W_{bi0}/B
1	Cargo ship	103.0	20.0	0.267	27.4	17.4	0.87
2	Cargo ship (small size)	60.4	11.2	0.255	15.4	9.8	0.87
3	Container ship (14,000 TEU)	352.0	51.0	0.282	99.4	73.9	1.45
4	Container ship (10,000 TEU)	318.3	45.8	0.268	85.2	62.3	1.36
5	Container ship (6,000 TEU OVER PANAMAX)	283.8	40.0	0.266	75.5	55.5	1.39
6	Container ship (4,000 TEU PANAMAX)	273.0	32.2	0.261	71.3	55.2	1.71
7	Bulk carrier (VLOC)	318.0	55.0	0.266	84.5	57.0	1.04
8	Bulk carrier (CAPESIZE)	279.0	45.0	0.269	75.1	52.6	1.17
9	Bulk carrier (NEW PANAMAX)	236.0	38.0	0.260	61.3	42.3	1.11
10	Bulk carrier	216.0	32.3	0.269	58.1	41.9	1.30

	Ship type	L_{pp}	B	S_{pb}/L_{pp}	S_{pb}	W_{bi0}	W_{bi0}/B
11	Bulk carrier (small size)	119.2	21.5	0.261	31.1	20.3	0.95
12	Tanker (VLCC)	316.0	60.0	0.252	79.7	49.7	0.83
13	Tanker (small size)	92.0	20.0	0.259	23.8	13.8	0.69
14	Pure car carrier (PCC) (VLCC)	190.9	36.5	0.271	51.7	33.5	0.92
15	Pure car carrier (PCC) (large size)	190.0	32.2	0.265	50.4	34.3	1.06
16	Pure car carrier (PCC)	180.0	32.2	0.263	47.3	31.2	0.97
17	LNG ship	270.0	44.8	0.260	70.1	47.7	1.07
18	LPG ship	220.0	36.6	0.264	58.1	39.8	1.09
19	Refrigerated cargo carrier	144.0	23.5	0.267	38.4	26.6	1.13
20	Passenger ship (large, 2 shafts, 2 propellers)	306.0	38.4	0.256	78.3	59.1	1.54
21	Passenger ship (2 shafts, 2 propellers)	160.0	24.7	0.239	38.3	25.9	1.05
22	Ferry boat (2 shafts, 1 propeller)	181.0	29.4	0.250	45.2	30.5	1.04

5) Corrections for widths to account for bank effect forces based on wall shapes

i. When the shape of a side wall is as shown in **Fig. 2.3.17**, it is necessary to set a correction coefficient h_f based on the ratios of the depths outside the navigation channels to navigation channel depths (h_1: navigation channel depth ratio). The correction coefficients can be calculated by **equation (2.3.36)** proposed by Kijima et al.[21)]

$$h_f = \exp\left(-2\frac{h_1}{1-h_1}\right) \qquad \textbf{(2.3.36)}$$

where

h_f : correction coefficient corresponding to the navigation channel depth ratio h_1;

h_1 : navigation channel depth ratio (= depth outside navigation channel D_{out}/navigation channel depth D).

In the case of **Fig. 2.3.14**, $h_1 = 0$. For navigation channels with no bank walls, $h_1 = 0.9999$.

ii. The widths to account for bank effect forces W_{bi} corresponding to navigation channel depth ratios can be calculated by multiplying the widths account for vertical bank wall effect forces W_{bi0} by the correction coefficients h_f.

$$W_{bi} = W_{bi0} h_f \qquad \textbf{(2.3.37)}$$

where

W_{bi} : width to account for bank effect forces when the bank wall is not vertical (m);

W_{bi0} : width to account for vertical bank wall effect forces with a required check rudder angle of 5° (m);

h_f : correction coefficient based on the ratio of the navigation channel depth to the depth outside the navigation channel (h_1).

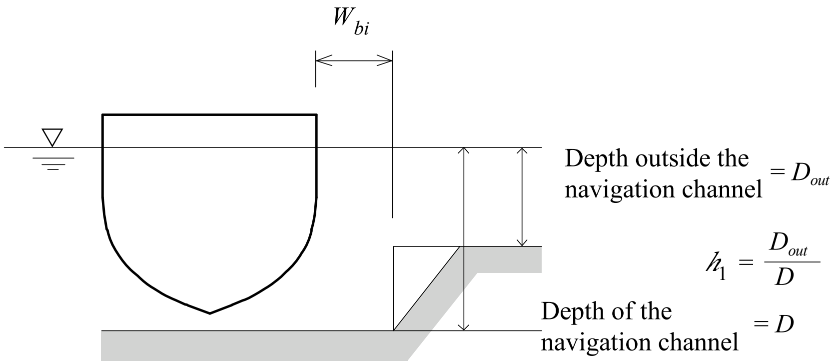


Fig. 2.3.17 Concept of Width to Account for Bank Effect Forces Based on Wall Shapes

iii. In cases where the slopes of side walls are gentle ($D_\theta \leq 45°$), as shown in **Fig. 2.3.18**, the depths outside the navigation channels can be modified to D_{out}' using **equation (2.3.38)** for calculating the widths to account for bank effect forces.

Modified depth outside the navigation channel : $D_{out}' = 0.5\ (D + D_{out})$ (m) **(2.3.38)**

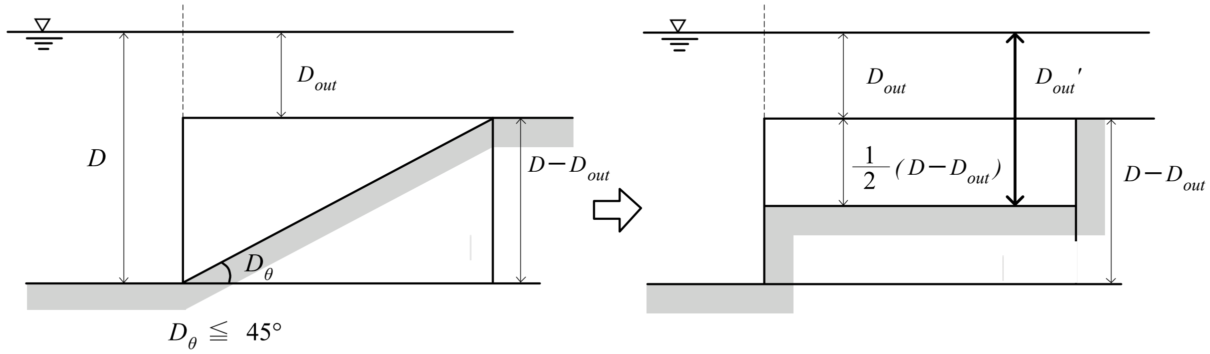


Fig. 2.3.18 Modified Depth outside the Navigation Channel When the Slopes of the Bank Walls are Gentle ($D_\theta \leq 45°$)

③ The widths to account for edges effect forces W_p (required widths to cope with the effects of the edges of breakwaters and jetties) can be calculated by the following procedures for special cases of side walls.

(a) Basic concept of calculation

Because the duration of the effects of the edges of breakwaters and jetties on ship navigation is considered to be relatively short, the required widths to account for edges effect forces can be calculated as the distances from the edges necessary to compensate for the effects with a maximum check rudder angle of 15°.

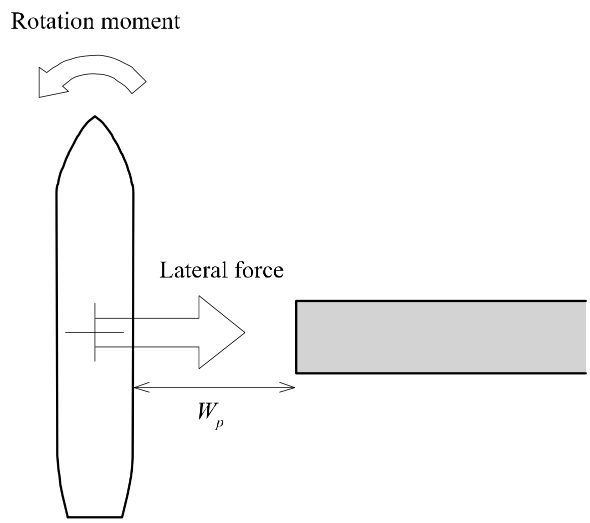


Fig. 2.3.19 Concept of Widths to Account for Edges Effect Forces

(b) Calculation of standard widths to account for edges effect forces W_p

1) Calculation of the widths to account for edges effect forces (subscript *p*: pier) when the crossing angles between the navigation channels and breakwaters or jetties are 90°

The widths to account for edges effect forces W_p can be calculated by the following phased calculation method.

Setting of the dimensions of design ships and navigation environment conditions

↓

Calculation of lateral force and rotation moment acting on hulls due to edges

↓

Calculation of check helm angles based on the motion equation in an equilibrium situation

↓

Calculation of widths to cope with the effects of edges with a required check helm angle of 15°

Fig. 2.3.20 Widths to Account for Edges Effect Forces

2) Calculation of the lateral force and rotation moment acting on hulls due to edges

The values for the lateral force C_F and rotation moment C_M acting on the hulls of ships navigating close to the edges of breakwaters and jetties for each S_p/L (= S_{pp}/L) can be obtained from **Fig. 2.3.21** proposed by Kijima et al.[20] Here, C_F (= C_{Fp}) and C_M (= C_{Mp}) are defined by **equation (2.3.39)**.

$$\left.\begin{aligned} C_{F_p} &= \frac{F_p}{0.5\rho_w L d U^2} \\ C_{M_p} &= \frac{M_p}{0.5\rho_w L^2 d U^2} \end{aligned}\right\} \quad \textbf{(2.3.39)}$$

where

S_{pp} : distance from the centerline of the navigation channel to the edge (S_p in **Fig. 2.3.21**) (m);

L : length between perpendiculars L_{pp} (m);

F_n : lateral force acting on the hull of a ship navigating close to the edge of a breakwater or jetty (N);

C_{Fp} : dimensionless value of the lateral force acting on the hull of a ship navigating close to the edge of a breakwater or jetty;

M_p : rotation moment acting on the hull of a ship navigating close to the edge of a breakwater or jetty (N·m);

C_{Mp} : dimensionless value of the rotation moment acting on the hull of a ship navigating close to the edge of a breakwater or jetty;

U : ship speed (m/s);

d : maximum draft of a moored design ship in still water (m);

ρ_w : density of seawater (kg/m^3).

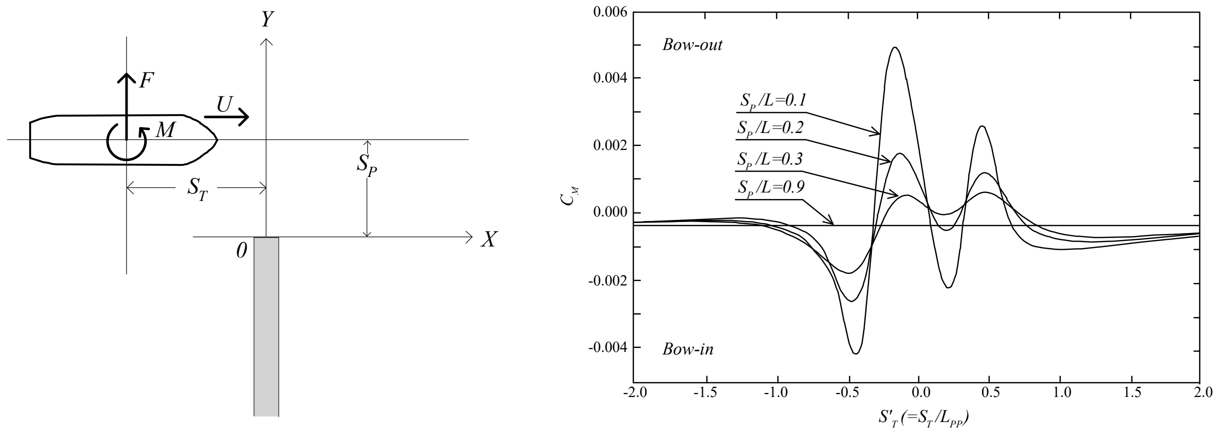


Fig. 2.3.21 Repulsive Moment on Ships Navigating Close to Edges [Shallow Water Areas, *D/d* = 1.2]

*Partial modification is made on the figure shown in the literature 20) ($S_p = S_{pp}$ in the figure)

3) Calculation of check rudder angles based on the motion equation in an equilibrium situation

Unlike the continuous effects of side walls, the duration of the effects of the edges of breakwaters and jetties on ship navigation is relatively short. Therefore, it is not likely that ships moving straight ahead will suddenly undergo oblique navigation with steady drift angles. Considering that ships have less chance of developing drift angles due to the effects of edges, the motion equation in an equilibrium situation in the coordinate system of **Fig. 2.3.21** as expressed by **equation (2.3.40)** can be changed to **equation (2.3.41)** by setting $\beta = 0$.

$$\left.\begin{aligned} -C_{Fp} + Y'_\beta \beta + Y'_\delta \delta = 0 \\ -C_{Mp} + N'_\beta \beta + N'_\delta \delta = 0 \end{aligned}\right\} \quad \textbf{(2.3.40)}$$

$$-C_{Mp} + N'_\delta \delta = 0 \quad \textbf{(2.3.41)}$$

Thus, rudder angle δ can be calculated by **equation (2.3.42)**.

$$\delta = \frac{C_{Mp}}{N'_{\delta}} \tag{2.3.42}$$

where

- C_{Fp} : dimensionless value of the lateral force acting on the hull of a ship navigating close to the edge of a breakwater or jetty;
- C_{Mp} : dimensionless value of the rotation moment acting on the hull of a ship navigating close to the edge of a breakwater or jetty;
- Y'_{β} : dimensionless value of the coefficient Y_{β} for the reaction force in a transverse direction from a fluid when a ship diagonally navigates with a drift angle β;
- N'_{β} : dimensionless value of the coefficient N_{β} for the turning reaction moment from a fluid when a ship diagonally navigates with a drift angle β;
- Y'_{δ} : dimensionless value of a transverse force coefficient Y_{δ} generated by a rudder set at rudder angle δ;
- N'_{δ} : dimensionless value of the coefficient N_{δ} for a rudder force moment generated by a rudder at rudder angle δ;
- δ : rudder angle (rad);
- β : drift angle (rad).

It is noted that the unit system of δ and β needs to be changed to degrees (°) for the following calculations.

Here, among the figures in the literature 20), **Fig. 2.3.21** shows the case of C_M (= C_{Mp}) only with a coordinate system converted into that of **Fig. 2.3.16**. In **Fig. 2.3.21**, the maximum values of C_M (= C_{Mp}) are 0.0050, 0.0018 and 0.00056 when the values of S_{pp}/L are 0.1, 0.2 and 0.3, respectively.

4) **Calculation of the widths W_p to account for edges effect forces when the required check rudder angle is 15°**

The widths W_p can be calculated in a manner that obtains a regression equation of S_{pp}/L with rudder angles δ as a variable from the calculation results of δ corresponding to S_{pp}/L, calculates S_{pp}/L values by substituting δ = 15° into the regression equation, and calculates W_p by substituting the obtained S_{pp}/L values into **equation (2.3.43)**. However, in case the calculated rudder angles with S_{pp}/L exceeds 30° which is unrealistically large, the value of S_{pp}/L shall be ignored for developing the regression equation.

$$W_p = S_{pp} - 0.5B \tag{2.3.43}$$

where

- W_p : width to cope with the effects of an edge (m);
- S_{pp} : distance from the centerline of the ship to the edge (m);
- B : molded breadth (m).

However, as will be seen after obtaining the calculation results of **equation (2.3.43)**, in cases where the widths of breakwaters and jetties are not wide (widths ≤ 0.1 L_{pp}), their edges generally have fairly minor hydrodynamic interference effects on ships navigating nearby. In such cases, it is reasonable to consider that $W_p \approx 0$.

(c) **Calculation** of widths to account for edges effect forces W_p when the widths of breakwaters or jetties are large compared to the ship lengths or when the crossing angles between navigation channels and breakwaters or jetties are other than 90°

1) Basic concept of calculation

Realistically, there are breakwaters and jetties which have widths larger than the ship lengths (widths > 0.1 L_{pp}), and the crossing angles between the navigation channels and breakwaters or jetties are not always 90°. Although even in such cases, the widths to account for edges effect forces W_p can be basically calculated in the same way as in the case of (b) above, and **Fig. 2.3.22** proposed by Kijima et al.[20] can be used in place of **Fig. 2.3.21**.

2) Standard calculation method and calculation equation

i. **Fig. 2.3.22** shows C_M (= C_{Mp}) acting on ships in navigation channels that have wedge-shaped side walls with tip angles (E_β) ranging from 0 to 180°. The case of E_β = 180° in **Fig. 2.3.22** is equivalent to the case of a steady state with S'_T > 1.5 in **Fig. 2.3.16 (b)**, and both cases show the value of 0.0050 for C_M (= C_{Mp}) when S_p/L (= S_{pp}/L) = 0.1. In contrast, when there are breakwaters along the moving directions of ships in navigation channels (E_β = 0°), C_M (= C_{Mp}) is maximized and the hydrodynamic interference effects on ships are considered to be significant. Furthermore, the C_M (= C_{Mp}) value of 0.015 when E_β = 90° and S_p/L (= S_{pp}/L) = 0.1 in **Fig. 2.3.22** is larger than that of 0.0050 when the crossing angle is 90° in **Fig. 2.3.21**.

ii. **Fig. 2.3.22** shows only the case of C_M (= C_{Mp}) when S_p/L (= S_{pp}/L) = 0.1. Thus, in order to obtain the values of C_M (= C_{Mp}) corresponding to the cases of S_p/L (= S_{pp}/L) = 0.2, 0.3 and 0.5, their approximated values need to be calculated in a manner that obtains ratios of the values of C_M (= C_{Mp}) at E_β respective of the set conditions in **Fig. 2.3.21** to the peak value of C_M (= C_{Mp}) when E_β = 90°, and multiplies the peak values in **Fig. 2.3.16 (b)** in the respective states of S'_T by the ratios.

iii. In calculating the widths of the navigation channels to account for edges effect forces when the widths of the breakwaters or jetties are narrow and the crossing angles between the navigation channels and the breakwaters or jetties are almost 90°, with particular focus on uncertain factors such as the effects of unsteady actions (wind pressure, wave drift forces, tidal currents, etc.) and errors in identifying the positions of the edges, **equation (2.3.44)** can be used in accordance with the type of ships.

$$W_p = aB \tag{2.3.44}$$

where

W_p : width to account for edges effect forces (m);

B : molded breadth (m);

a : 0.5 to 1.0.

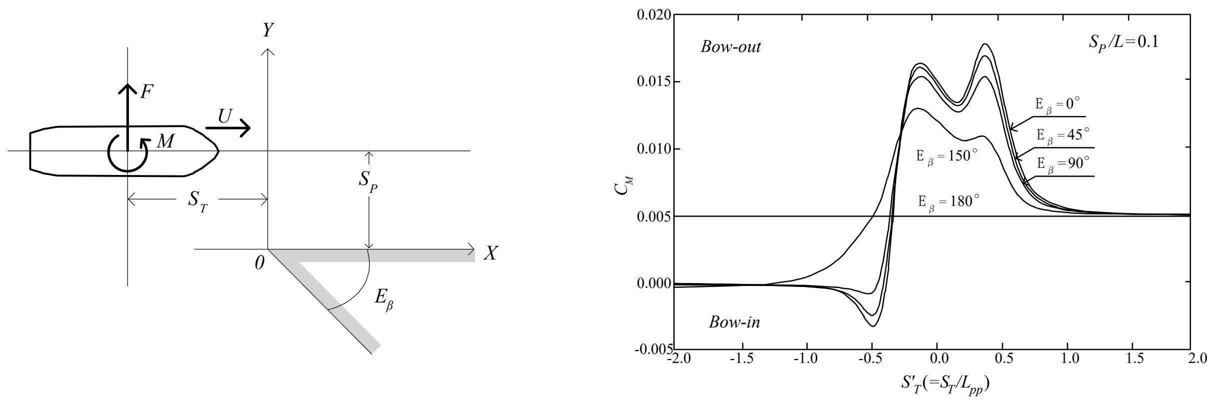


Fig. 2.3.22 Repulsive Moment on Ships Navigating Close to Edges [Shallow Water Areas, *D/d* = 1.2][20]
(S_p = S_{pp} in the figure)

④ Widths to account for two-ship interaction in passing W_c (the required widths to cope with the effects of ships passing each other) can be calculated by the following procedures.

(a) Basic concept of calculation

1) Because the duration of the effects due to ships passing each other is relatively short, the required widths can be calculated as safe inter-ship distances enabling ships to account for two-ship interaction in passing with a maximum check rudder angle of 15°.

2) In this calculation, the inter-ship distances are determined by assuming a dangerous positional relationship between ships passing each other as shown in the dotted lines in **Fig. 2.3.23**, even when the ships move diagonally in an actual situation.

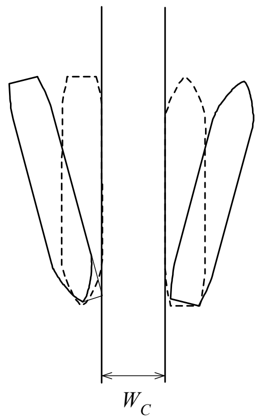


Fig. 2.3.23 Concept of Widths to Account for Two-ship Interaction in Passing

(b) Standard calculation method and calculation equation

1) Calculation of widths to account for two-ship interaction in passing W_c (subscript c: center)

Widths to account for two-ship interaction in passing W_c when the ships have identical types and speeds can be calculated by the following phased calculation method.

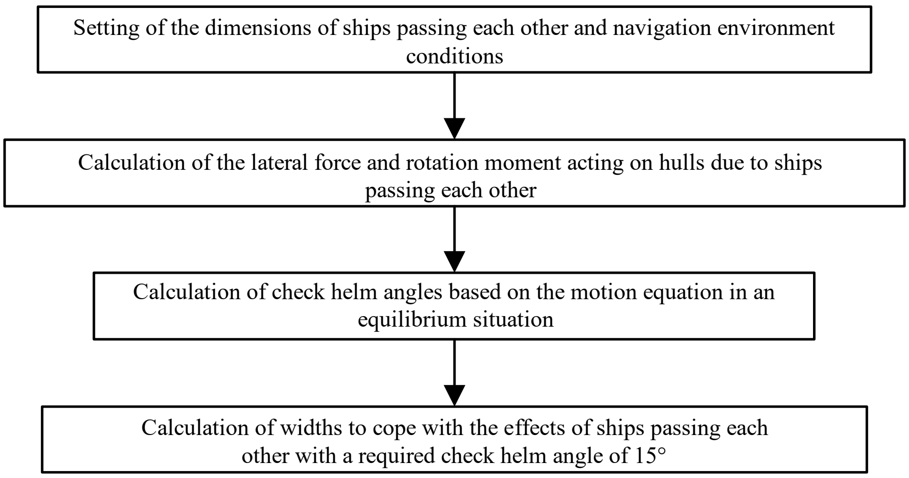


Fig. 2.3.24 Calculation of Width to Account for Two-ship Interaction in Passing

2) Calculation of the lateral force and rotation moment acting on the hulls of ships in passing

The values for the lateral force C_F and rotation moment C_M acting on the hulls of ships in passing for the respective S_p/L (= S_{pc}/L) values can be obtained from **Fig. 2.3.25** proposed by Kijima et al.[22] Here, C_F (= C_{Fc}) and C_M (= C_{Mc}) are the values defined by **equation (2.3.45)**.

$$\left.\begin{aligned} C_{Fc} &= \frac{F_c}{0.5\rho_w L d U^2} \\ C_{Mc} &= \frac{M_c}{0.5\rho_w L^2 d U^2} \end{aligned}\right\} \qquad \textbf{(2.3.45)}$$

where

S_{pc} : distance between the centerlines of the ships (S_p in **Fig. 2.3.25**)

L : length between perpendiculars (= L_{pp}) (m);

F_c : lateral force acting on the hulls of the respective ships in passing (N);

C_{Fc} : dimensionless value of the lateral force acting on the hulls of the respective ships in passing;

M_c : rotation moment acting on the hulls of the respective ships in passing (N·m);

C_{Mc} : dimensionless value of the rotation moment acting on the hulls of the respective ships in passing;

U : ship speed (m/s);

d : maximum draft of a moored design ship in still water (m);

ρ_w : density of seawater (kg/m^3).

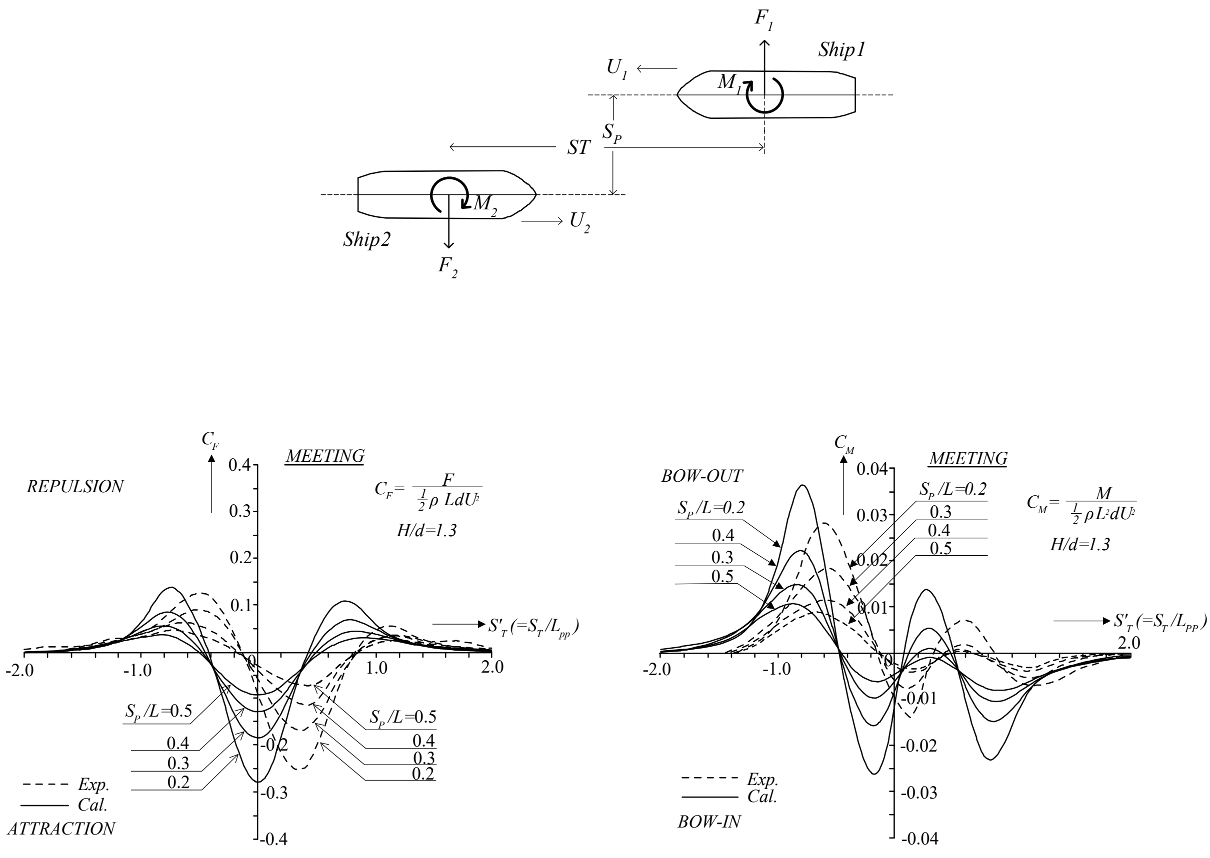


Fig. 2.3.25 Suction Force and Repulsive Moment on Ships in Passing (S_p = S_{pc} in the figure)[22)]

3) Calculation of the check rudder angle based on the motion equation in an equilibrium situation

Unlike the continuous effects of bank walls, the duration of the effects of ships in passing is relatively short. Therefore, it is not likely that ships moving straight ahead will suddenly undergo oblique navigation with steady drift angles. Considering that ships have less chance of developing drift angles due to the effects of ships in passing, the motion equation in an equilibrium situation in the coordinate system of **Fig. 2.3.25** as expressed by **equation (2.3.46)** can be changed to **equation (2.3.47)** by setting $\beta = 0$.

$$\left.\begin{array}{l} -C_{Fc} + Y'_{\beta}\beta + Y'_{\delta}\delta = 0 \\ -C_{Mc} + N'_{\beta}\beta + N'_{\delta}\delta = 0 \end{array}\right\} \quad \textbf{(2.3.46)}$$

$$-C_{Mc} + N'_{\delta}\delta = 0 \quad \textbf{(2.3.47)}$$

Therefore, rudder angle δ can be calculated by **equation (2.3.48)**.

$$\delta = \frac{C_{Mc}}{N'_{\delta}} \quad \textbf{(2.3.48)}$$

where

C_{Fc} : dimensionless value of the lateral force acting on the hulls of the respective ships in passing;

C_{Mc} : dimensionless value of the rotation moment acting on the hulls of the respective ships in passing;

Y'_{β} : dimensionless value of the coefficient Y_{β} for the reaction force in a transverse direction from a fluid when a ship diagonally navigates with a drift angle β;

N'_{β} : dimensionless value of the coefficient N_{β} for the turning reaction moment from a fluid when a ship diagonally navigates with a drift angle β;

Y'_{δ} : dimensionless value of the coefficient Y_{δ} for the lateral force generated by a rudder at rudder angle δ;

N'_{δ} : dimensionless value of the coefficient N_{δ} for the rudder force moment generated by a rudder at rudder angle δ;

δ : rudder angle (rad);

β : drift angle (rad).

However, it is noted that the unit system of δ and β needs to be changed to degrees (°) for the following calculations.

In **Fig. 2.3.25**, which shows both test and calculation results, the calculation results which are larger than the test results (evaluated to represent more dangerous cases than in the test cases) can be used. In **Fig. 2.3.25**, the maximum values of C_M (= C_{Mc}) are 0.023, 0.015 and 0.011 when the values of S_{pc}/L are 0.3, 0.4 and 0.5, respectively.

4) Calculation of the widths W_c to account for two-ship interaction in passing when the required check rudder angle is 15°

The widths W_c can be calculated in a manner that obtains a regression equation of S_{pc}/L with rudder angles δ as a variable from the calculation results of δ corresponding to S_{pc}/L, calculates S_{pc}/L values by substituting δ = 15° into the regression equation, and calculates W_c by substituting the obtained S_{pc}/L values into **Equation (2.3.49)**. However, in case the calculated rudder angles with S_{pc}/L exceeds 30° which is unrealistically large, the value of S_{pc}/L shall be ignored for developing the regression equation.

$$W_c = S_{pc} - (0.5B + 0.5B) = S_{pc} - B \quad \textbf{(2.3.49)}$$

where

W_c : width to account for two-ship interaction in passing (m);

S_{pc} : distance between the centerlines of the ships (m);

B : molded breadth (m).

The calculation results of the widths to account for two-ship interaction in passing for the type of ships listed in **Table 2.3.1** are shown in **Table 2.3.6**. The values in the table can be used as approximate values of W_c. It shall be noted that W_c values are not affected by ship speeds.

Table 2.3.6 W_c by Ship Type (Required Width to Account for Two-ship Interaction in Passing with a Check Rudder Angle of 15° and D/d = 1.3)

	Ship type	L_{pp}	B	S_{pc}/Lpp	S_{pc}	W_c	W_c/B
1	Cargo ship	103.0	20.0	0.511	52.6	32.6	1.63
2	Cargo ship (small size)	60.4	11.2	0.477	28.8	17.6	1.57
3	Container ship (14,000 TEU)	352.0	51.0	0.551	194.1	143.1	2.81
4	Container ship (10,000 TEU)	318.3	45.8	0.517	164.6	118.8	2.59
5	Container ship (6,000 TEU OVER PANAMAX)	283.8	40.0	0.511	145.0	105.0	2.63
6	Container ship (4,000 TEU PANAMAX)	273.0	32.2	0.498	135.8	103.6	3.22
7	Bulk carrier (VLOC)	318.0	55.0	0.510	162.0	107.0	1.95
8	Bulk carrier (CAPESIZE)	279.0	45.0	0.516	143.8	98.8	2.20
9	Bulk carrier (NEW PANAMAX)	236.0	38.0	0.496	117.1	79.1	2.08
10	Bulk carrier	216.0	32.3	0.516	111.3	79.0	2.45
11	Bulk carrier (small size)	119.2	21.5	0.501	59.7	38.2	1.77
12	Tanker (VLCC)	316.0	60.0	0.478	151.0	91.0	1.52
13	Tanker (small size)	92.0	20.0	0.492	45.2	25.2	1.26
14	Pure car carrier (PCC) (VLCC)	190.9	36.5	0.521	99.5	63.0	1.73
15	Pure car carrier (PCC) (large size)	190.0	32.2	0.510	96.8	64.6	2.01
16	Pure car carrier (PCC)	180.0	32.2	0.504	90.6	58.4	1.81
17	LNG ship	270.0	44.8	0.502	135.5	90.7	2.03
18	LPG ship	220.0	36.6	0.507	111.5	74.9	2.05
19	Refrigerated cargo carrier	144.0	23.5	0.514	74.0	50.5	2.15
20	Passenger ship (large, 2 shafts, 2 propellers)	306.0	38.4	0.499	152.7	114.3	2.98
21	Passenger ship (2 shafts, 2 propellers)	160.0	24.7	0.453	72.4	47.7	1.93
22	Ferry boat (2 shafts, 1 propeller)	181.0	29.4	0.478	86.5	57.1	1.94

(c) Calculation method when ships of different types pass each other

When ships of different types (type 1 and type 2) pass each other, the widths to account for two-ship interaction in passing can be the average of the widths (W_{c1} and W_{c2}) respectively calculated by assuming two cases where type 1 and type 2 ships each pass ships of identical types.

⑤ Widths to account for two-ship interaction in overtaking W_{ov} (the required widths to cope with the effects of ships overtaking other ships) can be calculated by the following procedures.

(a) Basic concept of calculation

1) The required widths can be calculated as safe inter-ship distances enabling ships to cope with the maximum rotation moment as an effect of ships overtaking other ships with a maximum check rudder angle of 15°.

2) In this calculation, the inter-ship distances are determined by assuming the most dangerous positional relationship between ships overtaking other ships as shown in the dotted lines in **Fig. 2.3.26**, even when the ships navigate diagonally in an actual situation.

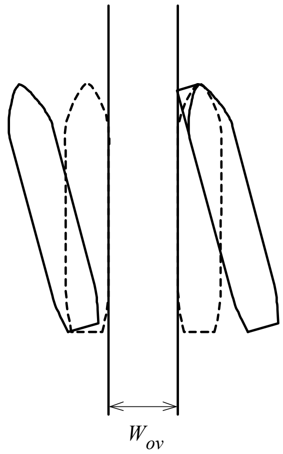


Fig. 2.3.26 Concept of Widths to Account for Two-ship Interaction in Overtaking

(b) Standard calculation method and calculation equation when ships in overtaking other ships of identical types

1) Widths to cope with the effects of ships overtaking other ships W_{ov} (subscript *ov*: overtake) when the ships have identical types can be calculated by the following phased calculation method.

Setting of the dimensions of ships overtaking other ships and navigation environment conditions

↓

Calculation of the lateral force and rotation moment acting on the hulls due to ships overtaking other ships

↓

Calculation of check helm angles based on the motion equation in an equilibrium situation

↓

Calculation of widths to cope with the effects of ships overtaking other ships with a required check helm angle of 15°

Fig. 2.3.27 Calculation of Width to Account for Two-ship Interaction in Overtaking

2) Calculation of the lateral force and rotation moment acting on the hulls of ships in overtaking

The values for the lateral force C_{Fi} and rotation moment C_{Mi} acting on the hull of a ship *i* overtaking another ship for the respective S_{p12}/L_i (= S_{pov12}/L_i) value can be those obtained from two types of graphs for the respective values in **Fig. 2.3.28** proposed by Lee et al.[23)] and the other based on additional calculation results—whichever is larger. Here, C_{Fi} (= C_{Fovi}) and C_{Mi} (= C_{Movi}) are the values defined by **equation (2.3.50)**.

$$\left.\begin{aligned} C_{Fovi} &= \frac{F_{ovi}}{0.5\rho_w L_i d_i U_i^2} \\ C_{Movi} &= \frac{M_{ovi}}{0.5\rho_w L_i d_i U_i^2} \end{aligned}\right\} \qquad \textbf{(2.3.50)}$$

where

S_{pov12} : distance between the centerlines of the ships (m) (S_{p12} in **Fig. 2.3.28**);

L : length between perpendiculars L_{pp} (m);

F_{ovi} : lateral force acting on the hull of a ship i in overtaking (N);

C_{Fovi} : dimensionless value of the lateral force acting on the hull of a ship i in overtaking;

M_{ovi} : rotation moment acting on the hull of a ship in overtaking (N·m);

C_{Movi} : dimensionless value of the rotation moment acting on the hull of a ship i in overtaking ;

U : ship speed (m/s);

d : maximum draft of a moored design ship in still water (m);

ρ_w : density of seawater (kg/m^3).

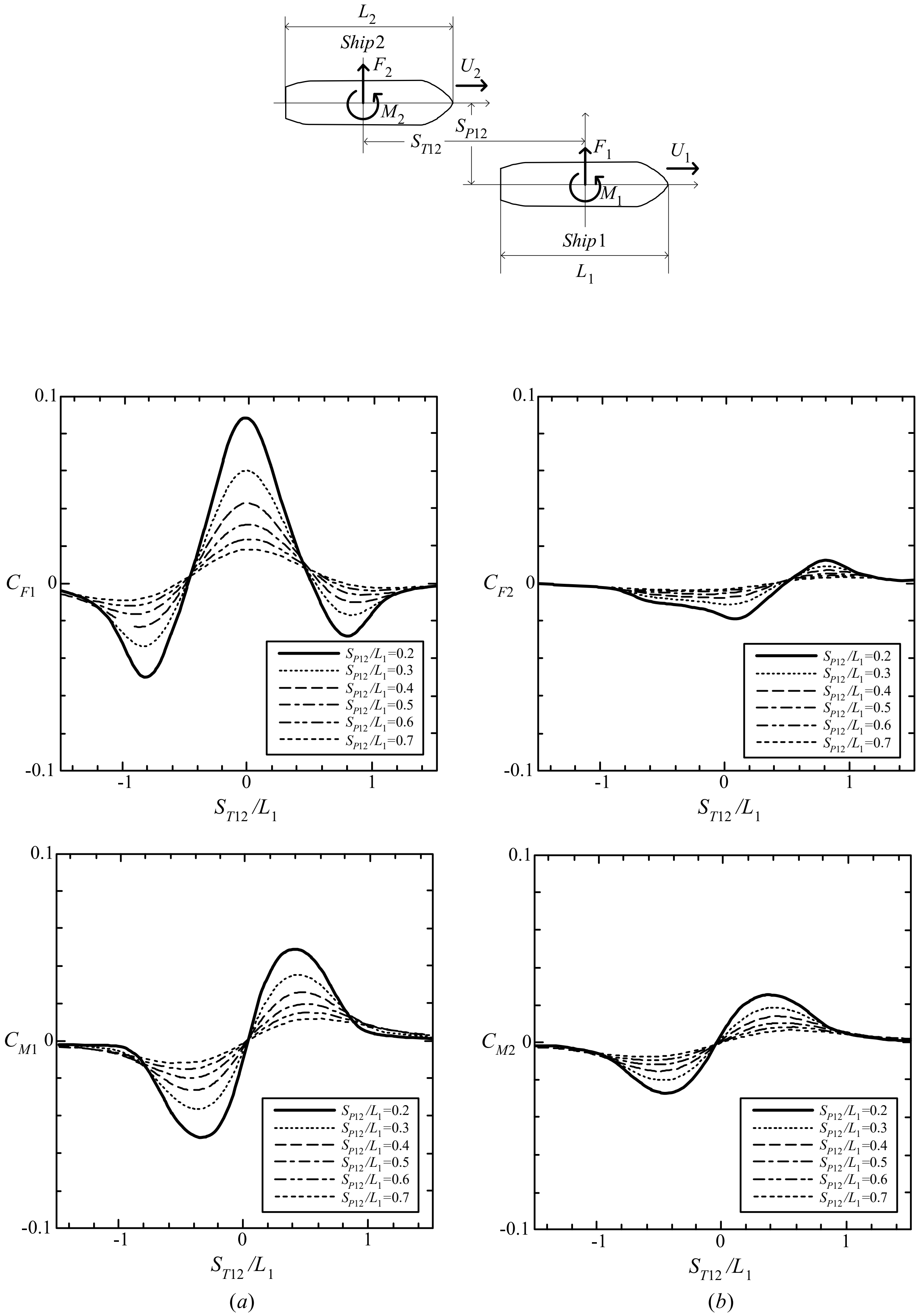


Fig. 2.3.28 Suction Force and Repulsive Moment on Ships in Overtaking (h/d = 1.2, U_2/U_1 = 1.2)[18)]

3) Calculation of rudder and drift angles based on the motion equation

Although the effects of ships in overtaking are continuous, the duration of the effects is relatively short. Therefore, it is not likely that ships moving straight ahead will suddenly undergo oblique navigation with steady drift angles. Considering that ships have less chance of developing drift angles due to the effects of ships overtaking others, the motion equation in an equilibrium situation in the coordinate system of **Fig. 2.3.28** as expressed by **equation (2.3.51)** can be changed to **equation (2.3.52)** by setting $\beta = 0$.

$$\left.\begin{aligned} -C_{Fovi} + Y'_{\beta i}\beta_i + Y'_{\delta i}\delta_i = 0 \\ -C_{Movi} + N'_{\beta i}\beta_i + N'_{\delta i}\delta_i = 0 \end{aligned}\right\} \quad \textbf{(2.3.51)}$$

$$-C_{Movi} + N'_{\delta i}\delta_i = 0 \quad \textbf{(2.3.52)}$$

Therefore, rudder angles δ_i can be calculated by **equation (2.3.53)**.

$$\delta_i = \frac{C_{Movi}}{N'_{\delta i}} \quad \textbf{(2.3.53)}$$

where

C_{Fovi} : dimensionless value of the lateral force acting on the hulls of ships in overtaking;

C_{Movi} : dimensionless value of the rotation moment acting on the hulls of ships in overtaking;

$Y'_{\beta i}$: dimensionless value of the coefficient Y_β for the reaction force in a transverse direction from a fluid when a ship diagonally navigates with a drift angle β;

$N'_{\beta i}$: dimensionless value of the coefficient N_β for the turning reaction moment from a fluid when a ship diagonally navigates with a drift angle β;

$Y'_{\delta i}$: dimensionless value of the coefficient $Y_{\delta i}$ for the lateral force generated by a rudder of a ship i at rudder angle δ;

$N'_{\delta i}$: dimensionless value of the coefficient $N_{\delta i}$ for the rudder force moment generated by a rudder of a ship i at rudder angle δ;

δ : rudder angle (rad);

β : drift angle (rad).

However, it is noted that the unit system of δ and β needs to be changed to degrees (°) for the following calculations.

In **Fig. 2.3.28** showing C_M (= C_{Mov}) in the case of D/d = 1.2, U_1 = 10 (kt) and U_2/U_1 = 1.2, the values of C_{M1} (= C_{Mov1}) can be selected because $C_{M1} > C_{M2}$ (= $C_{Mov1} > C_{Mov2}$). Furthermore, in **Fig. 2.3.28**, the maximum values of C_{M1} (= C_{Mov1}) are –0.0190, –0.0144 and –0.0111 when the values of S_{p12}/L_1 are 0.5, 0.6 and 0.7, respectively.

4) Calculation of the widths W_{ov} to account for two-ship interaction in overtaking when the required check rudder angle is 15°

The widths W_{ov} can be calculated in a manner that obtains a regression equation of S_{pov}/L with rudder angles δ as a variable from the calculation results of δ corresponding to S_{pov}/L, calculates S_{pov}/L values by substituting δ = 15° into the regression equation, and calculates W_{ov} by substituting the obtained S_{pov}/L values into **equation (2.3.54)**. However, in case the calculated rudder angles with S_{pov}/L exceeds 30° which is unrealistically large, the value of S_{pov}/L shall be ignored for developing the regression equation.

$$W_{ov} = S_{pov12} - (0.5B + 0.5B) = S_{pov12} - B \quad \textbf{(2.3.54)}$$

where

W_{ov} : width to account for two-ship interaction in overtaking (m);

S_{pov12} : distance between the centerlines of the ships (m);

B : molded breadth of the design ship (m).

The calculation results of widths to account for two-ship interaction in overtaking for the types of ships listed in **Table 2.3.1** are shown in **Table 2.3.7**. The values in the table can be used as approximate values of W_{ov}.

Table 2.3.7 W_{ov} by Ship Type (Required Width to Account for Two-ship Interaction in Overtaking with a Check Rudder Angle of 15° and D/d = 1.2)

	Ship type	L_{pp}	B	S_{pov}/L_{pp}	S_{pov}	W_{ov}	W_{ov}/B
1	Cargo ship	103.0	20.0	0.735	75.7	55.7	2.79
2	Cargo ship (small size)	60.4	11.2	0.683	41.2	30.0	2.68
3	Container ship (14,000 TEU)	352.0	51.0	0.800	281.5	230.5	4.52
4	Container ship (10,000 TEU)	318.3	45.8	0.746	237.4	191.6	4.18
5	Container ship (6,000 TEU OVER PANAMAX)	283.8	40.0	0.737	209.1	169.1	4.23
6	Container ship (4,000 TEU PANAMAX)	273.0	32.2	0.716	195.4	163.2	5.07
7	Bulk carrier (VLOC)	318.0	55.0	0.732	232.8	177.8	3.23
8	Bulk carrier (CAPESIZE)	279.0	45.0	0.743	207.2	162.2	3.60
9	Bulk carrier (NEW PANAMAX)	236.0	38.0	0.711	167.8	129.8	3.42
10	Bulk carrier	216.0	32.3	0.744	160.7	128.4	3.98
11	Bulk carrier (small size)	119.2	21.5	0.719	85.7	64.2	2.98
12	Tanker (VLCC)	316.0	60.0	0.683	215.7	155.7	2.60
13	Tanker (small size)	92.0	20.0	0.705	64.9	44.9	2.24
14	Pure car carrier (PCC) (VLCC)	190.9	36.5	0.752	143.5	107.0	2.93
15	Pure car carrier (PCC) (large size)	190.0	32.2	0.732	139.1	106.9	3.32
16	Pure car carrier (PCC)	180.0	32.2	0.725	130.4	98.2	3.05
17	LNG ship	270.0	44.8	0.722	194.9	150.1	3.35
18	LPG ship	220.0	36.6	0.729	160.4	123.8	3.38
19	Refrigerated cargo carrier	144.0	23.5	0.741	106.7	83.2	3.54
20	Passenger ship (large, 2 shafts, 2 propellers)	306.0	38.4	0.716	219.0	180.6	4.70
21	Passenger ship (2 shafts, 2 propellers)	160.0	24.7	0.644	103.0	78.3	3.17
22	Ferry boat (2 shafts, 1 propeller)	181.0	29.4	0.686	124.1	94.7	3.22

(c) Calculation method when ships overtake other ships of different types or different relative speed ratios

1) Basic concept of calculation

When ships overtake others, there may be cases where the types of ships overtaking or being overtaken are different from each other or are at several relative speed ratios. Although the calculation procedures of the widths to account for two-ship interaction in overtaking W_{ov} is basically the same as those described in (b), the values shown in **Fig. 2.3.29** and **2.3.30** by Lee et al.[23] can be used in place of those in **Fig. 2.3.28** (in **Fig. 2.3.29** and **2.3.30**, $S_{pov12} = S_{p12}$).

2) Standard calculation method and calculation equation

i. **Fig. 2.3.29** and **Fig. 2.3.30** show only the case when S_{p12}/L_1 = 0.2. Thus, in order to obtain the values of C_{M1} (= C_{Mov1}) corresponding to the cases of S_{p12}/L_i = 0.3, 0.4 and 0.5, their approximated values need to be calculated in a manner that obtains the ratios of the peak value of C_{M1} (= C_{Mov1}) when S_{p12}/L_1 = 0.2 corresponding to the respective conditions in **Fig. 2.3.29** and **Fig. 2.3.30** to the peak value of C_{M1} (= C_{Mov1}) when S_{p12}/L_1 = 0.2 in **Fig. 2.3.28**, and multiplies the peak values for S_{p12}/L_i by the ratios.

ii. For example, the C_{M1} (= C_{Mov1}) value when U_1 = 10 (kt) and U_2 = 15 (kt); that is, U_2/U_1 = 1.5, in **Fig. 2.3.29**, is –0.0769, and the C_{M1} (= C_{Mov1}) value when Sp_{12}/L_1 = 0.2 in **Fig. 2.3.28** is –0.050. Thus, the ratio of the two values is 1.56. Then, the C_{M1} (= C_{Mov1}) values when U_2/U_1 = 1.5 can be obtained by multiplying the C_{M1} (= C_{Mov1}) values when Sp_{12}/L_i = 0.3, 0.4 and 0.5 when U_2/U_1 = 1.2 by the ratio.

iii. When ships of different types overtake or are overtaken, C_{M1} (= C_{Mov1}) can be obtained in the same way as above using the values in **Fig. 2.3.30**.

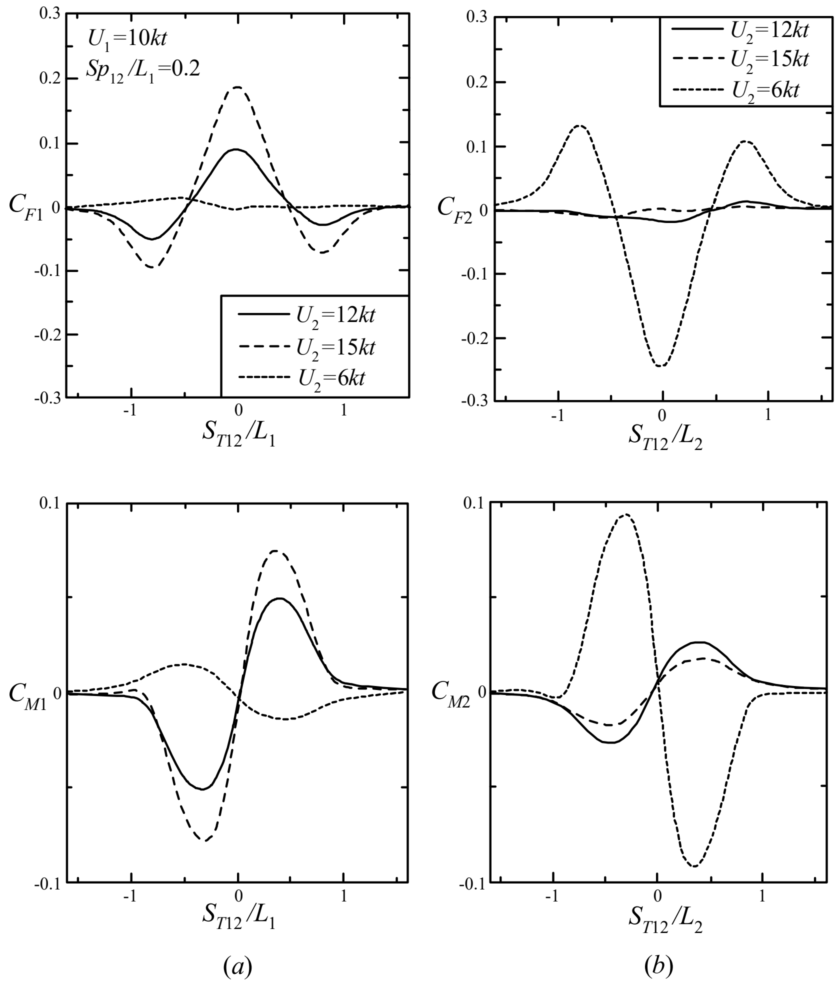


Fig. 2.3.29 Suction Force and Repulsive Moment Acting on Ships in Overtaking (h/d = 1.2, L_2/L_1 = 1.0)[18)]

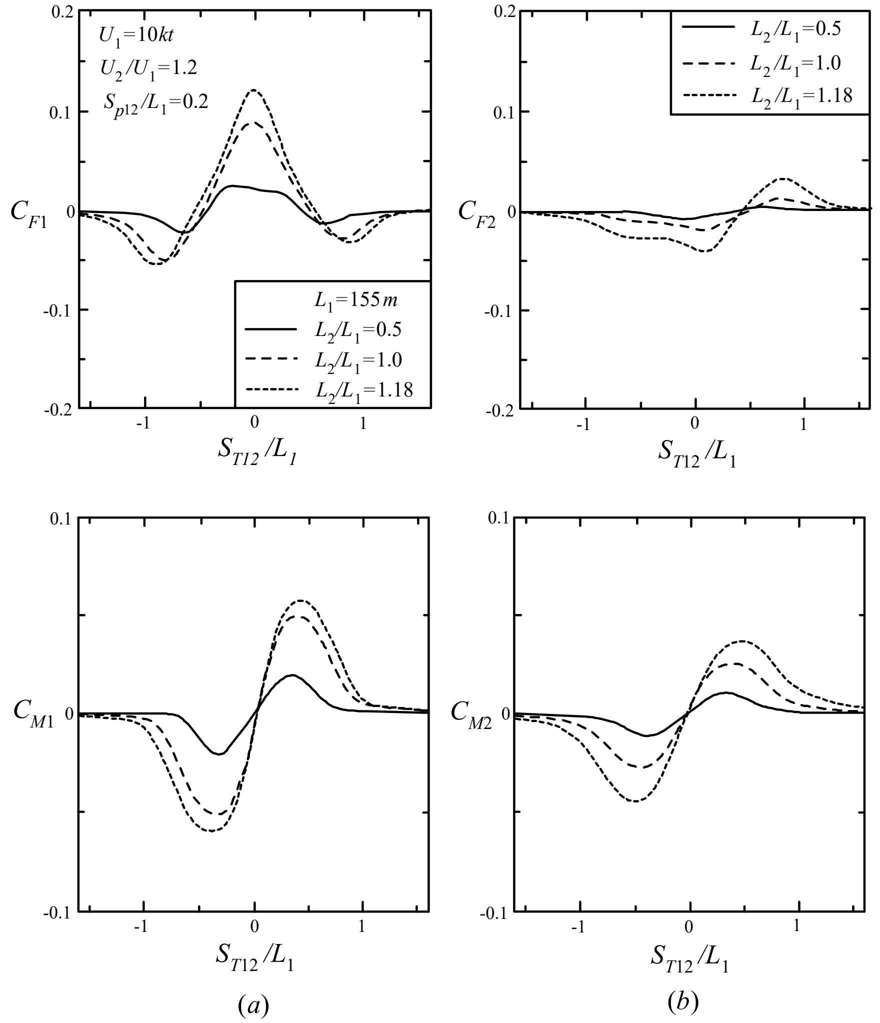


Fig. 2.3.30 Suction Force and Repulsive Moment Acting on Ships in Overtaking (h/d = 1.2)[18)]

(2) Convergence Calculation to Obtain the Widths of Newly Planned Navigation Channels

In calculating a basic ship maneuvering width W_{mi} which serves as an element to obtain the navigation channel width W using the required widths $W_m(a)$ and $W_m(R)$ to detect drift through visual and radar observation, respectively, the calculation procedure needs to start with W_{buoy} (the distance between anterior buoys); that is, the navigation channel width that will eventually be calculated, as an initial value. Thus, convergence calculation needs to be repeated until W_{buoy}, the initial value, becomes equal to W (the navigation channel width) that will eventually be calculated. The flow chart of this kind of convergence calculation is shown in **Fig. 2.3.31**.

In the figure, $W(i)$ represents the navigation channel width W obtained as a result of the ith calculation.

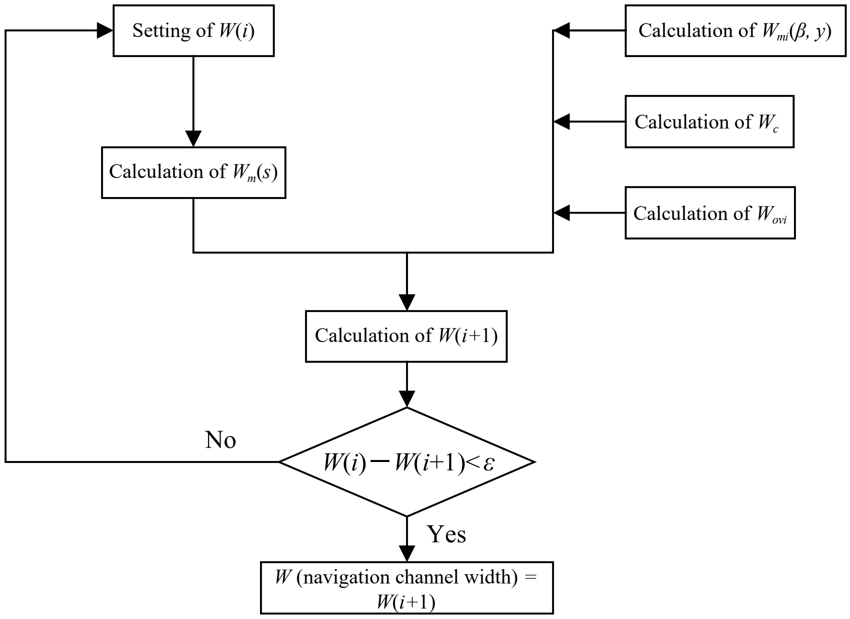


Fig. 2.3.31 Concept of Convergence Calculation

Convergence calculation is not required for the required widths $W_m(GPS)$, $W_m(D{\cdot}GPS)$ and $W_m(L)$ in cases where navigators are enabled to detect drift using GPS and guide marks (lights) because the calculation procedures of the required widths do not require W_{buoy} (the distance between anterior buoys) to be used as an initial entry.

(3) Application to the Design Changes of Existing Navigation Channels

When changing the design ships and navigation environment of existing navigation channels with the navigation channel widths W calculated by using the required widths $W_m(a)$ and $W_m(R)$ for enabling navigators to detect drift through visual and radar observation of navigation buoys on both sides of the navigation channels, W_{buoy} (the distance between anterior buoys) can be applied to the distance between buoys on both sides of the existing navigation channels. The navigation channel widths W calculated in this way can be evaluated by **equation (2.3.55)**.

$$W \text{ (width of the existing navigation channel)} \geq W \text{ (calculated navigation channel width)} \qquad \textbf{(2.3.55)}$$

In cases where the above equation cannot be satisfied, it is advisable to reassess the design changes or to expand the widths to a level equivalent to those obtained through convergence calculation as is the case with newly planned navigation channels.

2.4 Alignment of Navigation Channels (Bends)

2.4.1 Fundamentals of Performance Verification

(1) In Class 1 navigation channels, in cases where the bend angles exceed 30º and the design ships and features of the navigation environment such as rudder angles and ship speeds are not specified, it is advisable that the centerlines of the bends in the navigation channels be arcs with curvature radius roughly 4 times the lengths between perpendiculars of the design ships L_{pp} or greater, and that the widths of the navigation channels be equal to or greater than the necessary widths. When the angles of the intersection of the centerlines are 30º or greater in two-way navigation channels having widths W, it is advisable that the corner cuts be designed as shown in **Fig. 2.4.1**.

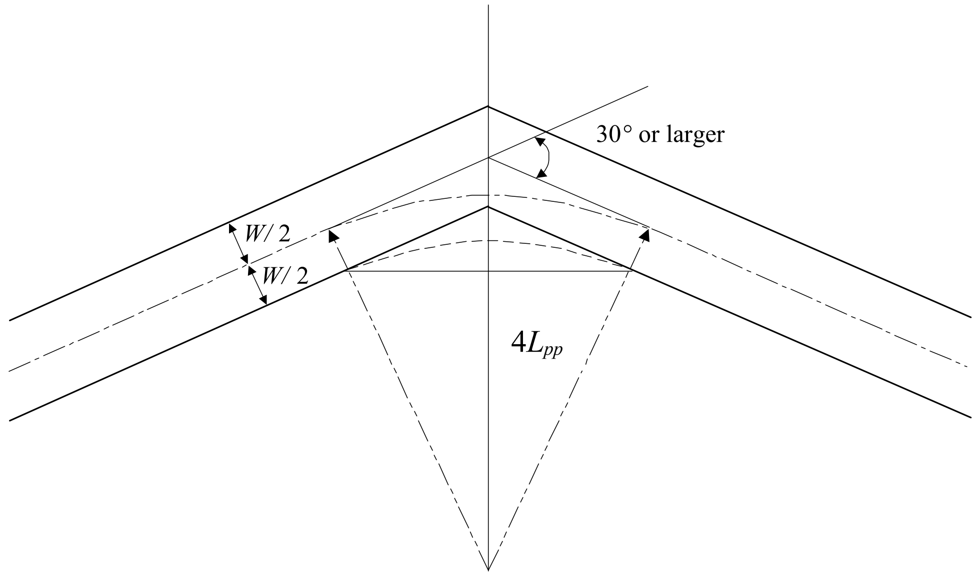


Fig. 2.4.1 Corner Cuts at the Bend Sections of Width *W* of Navigation Channels

(2) In Class 2 navigation channels, in cases where the bend angles exceed 30° and the design ships and features of the navigation environment such as rudder angles and ship speeds are specified, the curvature radius can be calculated based on the maneuverability indexes of turning, which show the turning performance factor of ships. It is advisable that the widths at the bends be expanded by corner cuts, etc., so as to be greater than the required widths.

The widths at bends may be expanded, not by corner cuts, but by curved corners, etc., considering the installation of buoys and other equipment based on adjustments with the parties concerned with maritime affairs. In particular, corner cuts are not always effective in cases where the angles of the intersection between the centerlines are large; therefore, it is advisable to consider the possibility of employing curved corners in such cases.

2.4.2 Performance Verification of Class 2 Navigation Channels

(1) The required ship turning radius of Class 2 navigation channels can be calculated by the following method.

① The required ship turning radius at the bends in navigation channels can be calculated by **equation (2.4.1)**.

$$R = L_{pp} / (K'\delta) = U / (K\delta) \quad \textbf{(2.4.1)}$$

where

R : ship turning radius of the centerline at a bend in the navigation channel (m);

K : turning performance factor;

K' : dimensionless value of the turning performance factor [$K' = K/(U/L_{pp})$];

L_{pp} : length between perpendiculars of the design ship (m);

δ : rudder angle of the design ship navigating a bend section (rad);

U : speed of the design ship navigating a bend section (m/s).

② The following table shows the reference values of *K'* in shallow water areas obtained as a result of simulation studies of the maneuverability of the types of ships listed in **Table 2.3.1** under calm conditions with a rudder angle fixed at 20°. For a method for calculating *K'* values, refer to the literature 1). The values in the table may be applicable to the rudder angles in the range of 15 to 20°.

For ships which are subjected to strong winds or which have particularly large superstructures, *K'* values need to be calculated separately.

Table 2.4.1 Dimensionless Values of Turning Performance Factor *K'* in Shallow Water Areas
[Shallow Water Areas: *D/d* = 1.2]

	Ship type	*K'*
1	Cargo ship	0.55
2	Cargo ship (small size)	0.46
3	Container ship (10,000 TEU)	0.60
4	Container ship (6,000 TEU OVER PANAMAX)	0.50
5	Container ship (4,000 TEU PANAMAX)	0.51
6	Bulk carrier (VLOC)	0.64
7	Bulk carrier (CAPESIZE)	0.51
8	Bulk carrier (NEW PANAMAX)	0.67
9	Bulk carrier	0.51
10	Bulk carrier (small size)	0.61
11	Tanker (VLCC)	0.62
12	Tanker (small size)	0.58
13	Pure car carrier (PCC) (large size)	0.63
14	Pure car carrier (PCC)	0.65
15	LNG ship	0.72
16	Refrigerated cargo carrier	0.58
17	Passenger ship (2 shafts, 2 propellers)	0.70
18	Ferry boat (2 shafts, 1 propeller)	0.56

Table 2.4.2 shows the ship turning radii of different types of ships. The values given in this table can be used as approximate values of *R*.

Table 2.4.2 Ship Turning Radii *R* in Shallow Water Areas [Shallow Water Areas: *D/d* = 1.2]

	Ship type	*R*
1	Cargo ship	4.9
2	Cargo ship (small size)	5.9
3	Container ship (10,000 TEU)	4.5
4	Container ship (6,000 TEU OVER PANAMAX)	5.5
5	Container ship (4,000 TEU PANAMAX)	5.3
6	Bulk carrier (VLOC)	4.4
7	Bulk carrier (CAPESIZE)	5.4
8	Bulk carrier (NEW PANAMAX)	4.2
9	Bulk carrier	5.4
10	Bulk carrier (small size)	4.4
11	Tanker (VLCC)	4.4
12	Tanker (small size)	4.5
13	Pure car carrier (PCC) (large size)	4.4
14	Pure car carrier (PCC)	4.2
15	LNG ship	3.8
16	Refrigerated cargo carrier	4.6
17	Passenger ship (2 shafts, 2 propellers)	3.4
18	Ferry boat (2 shafts, 1 propeller)	4.8

(2) For the determination of the curved geometry at the bend sections, reference can be made to the **Reference 24)** and the examples and guidelines at overseas ports.

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3 Basins

[Ministerial Ordinance] (Performance Requirements for Basins)

Article 10

The performance requirements for basins shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied in light of geotechnical characteristics, waves, water currents and wind conditions along with the usage conditions of the surrounding water areas for securing safe and smooth use by ships.

[Public Notice] (Performance Criteria for Basins)

Article 31

The performance criteria for basins shall be as specified in the subsequent items:

(1) The size of a basin shall satisfy the following standards, provided, however, that the standards shall not be applied to basins for design ships with a gross tonnage of less than 500 tons:

(a) Basins which are provided for use in the anchorage or mooring of ships excluding basins in front of quay walls, mooring piles, piers and floating piers shall have an area greater than a circle that has a radius obtained by adding an appropriate value to the length of the design ship in light of the conditions of geotechnical characteristics, waves, water currents and winds, as well as the usage conditions of the surrounding water areas. Provided, however, that in cases where the area specified above is not required, owing to the mode of anchorage or mooring, the basin size can be reduced to an area that shall not hinder the safe anchorage or mooring of ships.

(b) Basins which are provided for use in the anchorage or mooring of ships in front of quay walls, mooring piles, piers and floating piers shall have an appropriate area of which the length and width are greater than those of the design ship, respectively, in light of the conditions of geotechnical characteristics, waves, water currents and winds, the usage condition of the surrounding water areas and the mode of anchorage or mooring.

(c) Basins which are provided for use in turning the bow of a ship shall have an area greater than a circle that has a radius obtained by multiplying the length of the design ship by 1.5, provided, however, that in cases where the area specified above is not required depending on the method for turning the bow, the basin size can be reduced to an area that shall not hinder safe turning.

(2) The basin shall have an appropriate depth that is greater than the draft of a design ship, in light of the degree of the motions of the design ship due to waves, water currents, winds and other forces.

(3) Basins which are provided for use in the anchorage or mooring of ships in front of quay walls, mooring piles, piers and floating piers shall in principle secure harbor calmness, enabling the working rate of cargo handling operation at equal to or greater than 97.5% in terms of time throughout the year. Provided, however, that this rate shall not be applied to basins where the mode of utilization of mooring facilities or the water areas in front of them are regarded as special.

(4) In basins which are provided as safe harbor for refuge during stormy weather, the wave conditions during the storm shall remain below the level that is admissible for refuge of the design ship.

(5) In basins which are provided for the anchorage or mooring of ships for the main purpose of timber sorting, measures shall be taken to prevent the timber from drifting.

[Interpretation]

(2) Performance Criteria of Basins (Article 10 of the Ministerial Ordinance and the interpretation related to Article 31 of the Public Notice)

① The required performance of basins shall be serviceability. Here, serviceability means the performance of basins enabling ships to anchor, moor and shelter safely and smoothly.

② **The size of basins provided for use in the anchorage and mooring of ships**

Cases where the specified area is not required owing to the mode of anchorage or mooring are cases of buoy mooring. In such circumstances, the possible horizontal displacement of buoys due to the use conditions of basins and the effects of the tidal changes shall be appropriately examined.

③ **The size of basins provided for use in turning the bow of a ship**

Cases where the specified area is not required owing to the method of turning the bow mean ship turning with the use of tug boats, thrusters having sufficient thrust, or anchors.

④ **The depth of basins**

An appropriate depth that is greater than the draft of a design ship means the assumed maximum laden draft of the design ship, plus a keel clearance to be set according to the maximum draft.

⑤ **The harbor calmness of basins provided for use in the anchorage and mooring of ships**

Basins where the mode of utilization of mooring facilities or the water areas in front of them is regarded as special mean those which have a low annual usage frequency and marginal conditions for their use.

3.1 General

(1) Basins are preferably designed in consideration of the following factors: safe anchorage, facilitation of ship handling, efficiency of cargo handling, meteorological and hydrographic conditions, the effects of reflected waves and ship wake waves on harbor calmness, and consistency with related facilities.

(2) Because basins include not only anchorages and buoy mooring basins but also other water areas such as turning basins for ship handling, it is preferable that they have:

① calm and sufficiently wide water areas ;

② bottom sediment with good anchoring ;

③ buoys;

④ favorable meteorological and hydrographic conditions such as winds and tidal currents.

(3) The determination of the locations and areas of basins can be based on the results of ship maneuvering simulation systems using the data from an automatic identification system (AIS). Examples of ship maneuvering simulation based on AIS data are shown in **Fig. 3.1.1**. In addition, the literature 1) is one such case which studies the required area of a basin in front of a quay wall using AIS data.

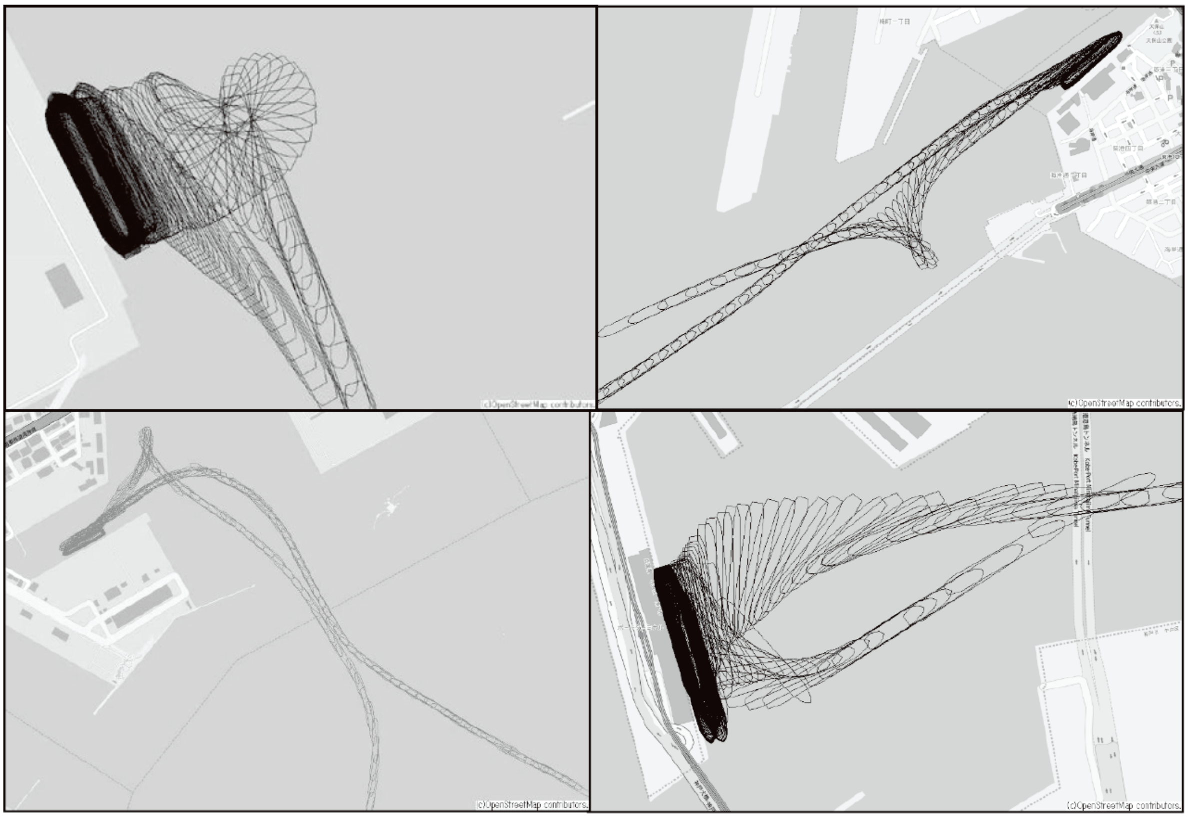


Fig. 3.1.1 Examples of Actual Ship Maneuvering Simulation Using AIS Data (Prepared by the Port Planning Division, the National Institute for Land and Infrastructure Management)

3.2 Performance Criteria

(1) Areas of Basins (Serviceability)

① **Basins provided for use in the anchorage or mooring of ships**

(a) Basins excluding those in front of quay walls or other facilities

Basins provided for use in the anchorage or mooring of ships, excluding those in front of quay walls, mooring piles, piers and floating piers, mean basins provided for use in anchoring and buoy mooring. In determining the areas of the basins for their performance verification, appropriate consideration shall be given to seabed properties, the effects of winds and water depths depending on the required functions and expected use conditions of the objective facilities. Cases where the specified area is not required owing to the mode of anchorage or mooring are cases of buoy mooring. In such circumstances, when determining the areas of the basins for their performance verification, the possible horizontal displacement of buoys due to the use conditions of basins and the effects of the tidal changes shall be appropriately examined.

(b) Basins in front of quay walls and other facilities

In the performance verification of basins in front of quay walls, mooring piles, piers and floating piers, the proper areas of the basins that shall be wider than the lengths overall and the widths of the design ships shall be determined with due consideration to: the additional lengths necessary for allowing design ships to come alongside mooring facilities and additional widths suitable for the methods to handle, moor and unmoor the design ships; the maneuverability of the design ships; the layouts of mooring facilities and navigation channels; the facilitation of ship handling; and safety when the design ships come alongside or leave mooring facilities.

(c) Other

In the performance verification of basins between piers, their widths shall be set with due consideration to the types of design ships, the number of berth and the presence or absence of the operation of tug boats.

Furthermore, in the performance verification of basins, their areas shall be determined with due consideration to: the expected use conditions, such as design ships coming alongside and leaving mooring facilities, as well as entering and leaving the basins concerned as needed; anchorage errors in the case of anchorage basins; and safety distances for basins used by ships loaded with hazardous cargo.

② **Basins provided for use in turning the bow of a ship**

(a) Basins provided for use in turning of the bow (hereinafter called “ship turning”) are also called turning basins. In the performance verification of the turning basins concerned, their scales shall be properly determined with due consideration to the design ships’ methods for ship turning, the positions and turning performance of the design ships, the arrangement of the mooring facilities and navigation channels, and the maneuverability of the design ships. Cases where the specified area is not required owing to the method of turning the bow mean ship turning with the use of tug boats, thrusters having sufficient thrust, or anchors.

(b) Basin areas not hindering safe ship turning

1) In the performance verification of turning basins, the areas of the turning basins that do not hinder safe ship turning can be the following values. Here, these values are determined for the sake of safety and are applicable to all cases regardless of the types of ships, performance of ship turning, wind speeds or topographical conditions.

For ship turning with thrusters having sufficient thrust, the value for the case of ship turning with use of a tug boat can be used.

- In the case of ship turning under a ship’s own power: a circle having a diameter 3 times the length overall of a design ship
- In the case of ship turning with the use of a tug boat: a circle having a diameter 2 times the length overall of a design ship

2) Special turning basins for small craft

For basins provided for use in the turning of small craft, in cases where it is of necessity to reduce the areas for the turning basins due to topographical constraints, the areas that do not hinder safe ship turning can be set by using the following values based on the utilization of mooring anchors, winds or tidal currents.

For ship turning with thrusters having sufficient thrust, the value in the case of ship turning with the use of tug boats can be used.

- In the case of ship turning under a ship’s own power: a circle having a diameter 2 times the length overall of the design ship
- In the case of ship turning with the use of a tug boat: a circle having a diameter 1.5 times the length overall of the design ship

3) Other special cases

- In cases where topographical constraints or other local conditions do not allow turning basins to have enough area for safe ship turning unless the water areas of neighboring navigation channels are temporarily available for ship turning in emergency situations, water areas smaller than those specified above can be set as turning basins that do not hinder safe ship turning, provided that the dimensions and navigation performance of the design ships are known and such temporary measures are determined to be implemented safely.
- In cases where the positional relationships between the mooring facilities and navigation channels are determined to allow ships to turn their bows with turning angles required for anchorage and mooring less than 90° without hindering safe navigation of other ships, the shapes of the turning basins may be appropriately set in accordance with the local situations of the turning basins concerned and the maneuvering methods of the design ships.

(c) Mooring and unmooring basins

In the performance verification of basins provided for use in mooring and unmooring, the scales of the basins shall be appropriately determined with due consideration to the design ships’ turning methods, the presence or absence of the use of thrusters, the effects of winds and tidal currents, and the maneuverability of the design ships.

(2) Water Depths of Basins (Serviceability)

① An appropriate depth that is greater than the draft of the design ship means an assumed maximum draft, such as the full laden draft of the design ship, plus a keel clearance to be set according to the maximum draft. In the performance verification of basins, water depths shall be appropriately determined so as to ensure depths greater than the drafts of the design ships below the datum levels for port management, provided, however, that this provision shall not apply to basins for use in hull fitting and other basins provided for use in the special anchorage or mooring of ships.

② **Ship turning with thrusters**

In the performance verification of basins expected to be used for special ship turning with thrusters, as is the case for ferries, keel clearances shall be appropriately set; for example, a value larger than approximately 10% of the general maximum draft, taking into consideration the effects of special ship turning.

(3) Harbor Calmness of Basins (Serviceability)

The harbor calmness of basins means the percentage of time when ships can use the basin safely and smoothly. In the performance verification of basins, when necessary, harbor calmness shall be appropriately verified by evaluating local conditions such as waves which may hinder the anchorage, mooring and cargo handling of ships in the basins. Although the harbor calmness of basins can be generally verified with wave heights as an index, due consideration shall be given to the directions and periods of waves which may cause the ship motion as well as mooring methods of the design ships as needed.

(4) Wave Conditions in Basins during Adverse Weather (Serviceability)

In the performance verification of basins, the allowable range of wave conditions during adverse weather shall be appropriately set giving due consideration to the heights, directions and periods of waves in the basins concerned, depending on the types and principal dimensions of the design ships and sheltering methods.

3.3 Performance Verification

[1] Locations and Areas

(1) Locations

① It is preferable to decide the locations of basins with due consideration to the positional relationships with breakwaters, wharves and navigation channels, as well as ensuring harbor calmness.

(2) Areas of Basins Provided for Use in Anchorage and Mooring

① Single anchoring (**Fig. 3.3.1 (a)**) and dual anchoring (**Fig. 3.3.1 (b)**) are the mooring methods most frequently employed. Other mooring methods include two-anchoring and bow-and-stern anchoring.

② It is necessary to determine the lengths of anchor chains in a manner that enables the anchor holding power of mooring anchors and chains lying on the seabed to resist possible actions acting on the hulls depending on the types of ships, mooring methods and meteorological, as well as hydrographic, conditions. In general, the stability of moored ships is improved with an increase in the lengths of anchor chains.

③ The swinging radius of anchorage can be determined from the sum of the ship's length and the horizontal distance between the bow and the center of rotation of the laying chain.

④ When the conditions required to calculate the lengths of the anchor chains are unknown, the values in **Table 3.3.1** may be used as references.

⑤ The conceptual diagrams of single buoy mooring and dual buoy mooring using two buoys at the bow and stern sides are shown in **Figs. 3.3.1 (c)** and **3.3.1 (d)**, respectively. For dual buoy mooring, the buoys shall be arranged in a manner that aligns the bow-and-stern directions of the ships parallel to the directions of the winds and tidal currents. For the sizes of water areas required for this kind of buoy mooring, refer to **Table 3.3.2**.

⑥ The widths of basins positioned between piers can be determined with reference to the following values.

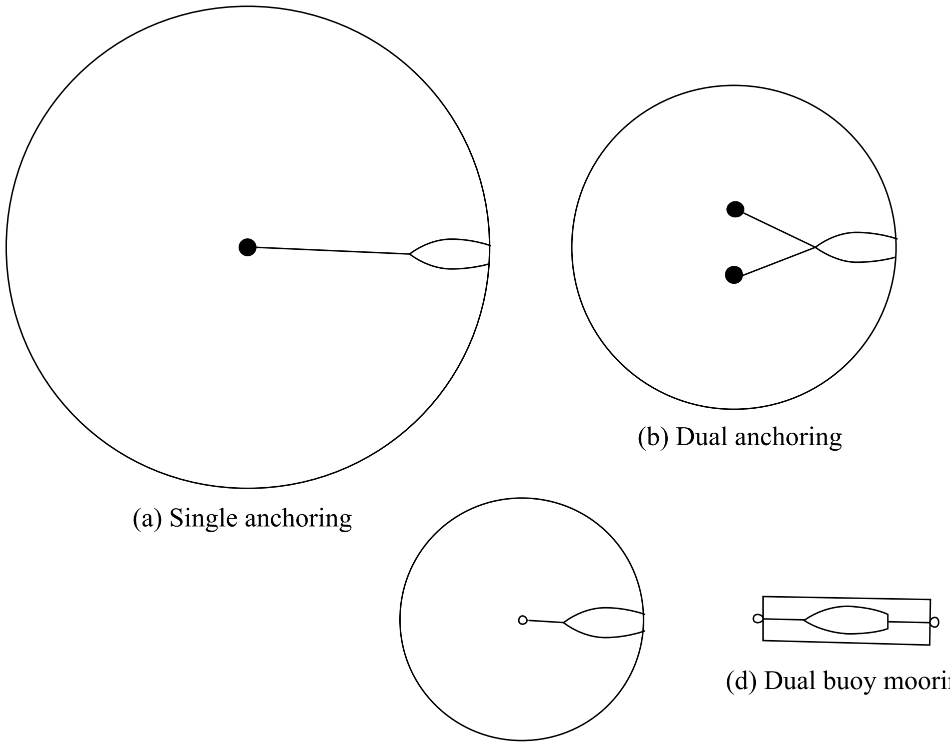
(L_{oa}: the length overall of the design ship)

(a) When the number of berths on one side of the pier is 3 or fewer: 1.0 L_{oa}

(b) When the number of berths on one side of the pier is 4 or more: 1.5 L_{oa}

When the innermost part of the water area between the piers is to be used as a small craft basin or reserved for the use of bunkering ships or barges, the widths of the basins between piers shall be determined in consideration of such use.

⑦ When examining anchoring methods and the scales of basins to cope with adverse weather, refer to the literature in 2) to 6).



(a) Single anchoring

(b) Dual anchoring

(c) Single buoy mooring

(d) Dual buoy mooring

Fig. 3.3.1 Concept of the Scale of Basins (Per Ship)

Table 3.3.1 Anchorage Basins

Usage purpose	Usage method	Sea bottom sediment	Radius (m)
Offshore waiting or cargo handling	Single anchoring	Good anchoring	$L_{oa} + 6D$
		Poor anchoring	$L_{oa} + 6D + 30$
	Dual anchoring	Good anchoring	$L_{oa} + 4.5D$
		Poor anchoring	$L_{oa} + 4.5D + 25$

Note: L_{oa}: length overall of the design ship (m), D: water depth (m)

Table 3.3.2 Sizes of Basins for Buoy Mooring

Usage method	Area
Single buoy mooring	A circle with a radius of (L_{oa} + 25) (m)
Dual buoy mooring	A rectangle with sides of (L_{oa} + 50) (m) and L_{oa}/2 (m)

Note: L_{oa}: length overall of the design ship (m)

(3) Areas of Mooring/Unmooring Basins Provided for Use in Maneuvering

Basin for mooring/unmooring

① In general, basins used for mooring/unmooring and navigation channels can be planned in the same water areas for the sake of efficient arrangement and for the use of port facilities, provided, however, that the basins and navigation channels are preferably separated in the case of dense marine traffic.

② When examining the sizes of basins for mooring/unmooring with the use of tug boats, refer to the literature in 7) and 8).

(4) Other

There may be cases where ships anchor in water areas inside and outside of ports in the event of a tsunami following an earthquake. The literature 9) shows the concept of determining the sizes of anchoring circles and the required water areas with Tokyo Bay given as an example.

[2] Water Depths

(1) The appropriate depths of basins shall be those that ensure the assumed maximum drafts, such as the full laden drafts of design ships, plus keel clearances to be set according to the maximum drafts below the datum levels for construction work.

(2) Basins provided for use in special anchorage or mooring mean those used by ships under outfitting or unloading at multiple ports in single voyages as part of normal operation, or design ships entering and leaving ports with drafts smaller than their standard full load drafts.

(3) When the drafts of design ships cannot be identified beforehand, refer to **Part III, Chapter 5, 2.1 Items Common to Quay Walls**.

(4) When water levels fall below the lowest astronomical tides due to seasonal variations in mean water levels larger than the variations in the astronomical tides, as is the case on the coast of the Sea of Japan, or when basins are subjected to significant propagation of a swell, it is necessary to determine the water depths of the basins with due consideration to the effects of the variation in water levels and the propagation of a swell.

[3] Harbor Calmness

(1) The performance verification of harbor calmness can be carried out with reference to **Part II, Chapter 2, 4.6 Concept of Harbor Calmness**.

(2) In the performance verification, the critical wave heights for cargo handling shall be appropriately set with due consideration to the types, dimensions and cargo handling performance of the design ships, as well as the directions and periods of object waves. When setting the critical wave heights for cargo handling, refer to the **Impact Evaluation Manual for Long-period Waves in Ports**.[10] Furthermore, in cases where ship motion due to a swell or long-period waves is not likely to cause problems with cargo handling, the critical wave heights for cargo handling can be set with reference to **Table 3.3.3**.

Table 3.3.3 Reference Values of Critical Wave Heights for Cargo Handling without the Influence of Swelling or Long-Period Waves

Ship type	Critical wave height for cargo handling ($H_{1/3}$)
Small craft	0.3 m
Medium/large ships	0.5 m
Very large ships	0.7 to 1.5 m

Note: Here, small craft mean ships of roughly 500 GT class or less, which mainly use small craft basins; very large ships mean ships of roughly 50,000 GT class or greater, which mainly use large-scale dolphins or offshore berths; and medium/large ships mean those other than small craft or very large ships.

[4] Timber Sorting Ponds

For structures and facilities used for timber sorting ponds, refer to Part III, Chapter 4, 8 Breakwaters for Timber Handling Facilities and Part III, Chapter 4, 3.2 Cargo Sorting Areas for Timber.

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4 Small Craft Basins

[Ministerial Ordinance] (Performance Requirements for Small Craft Basins)

Article 11

The performance requirements for small craft basins shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied in light of geotechnical characteristics, waves, water currents and wind conditions, as well as the usage conditions of the surrounding water areas so as to secure safe and smooth use by ships .

[Public Notice] (Performance Criteria for Small Craft Basins)

Article 32

1 The requirements specified in the preceding Article, item (ii) shall be applied mutatis mutandis to the performance criteria for small craft basins.

2 In addition to the provisions of the preceding paragraph, the performance criteria for small craft basins shall provide the shape, area and calmness necessary for the safe and smooth use of ships.

[Interpretation]

9. Waterways and Basins

(3) Performance Criteria of Small Craft Basins (Article 11 of the Ministerial Ordinance and the interpretation related to Article 32 of the Public Notice)

① With regard to the performance criteria and the interpretation of small craft basins, those of the depth of basins in Section 3.2(2) shall apply mutatis mutandis.

② In addition to the above, the performance verification of small craft basins shall be carried out with due consideration to the design ships and use conditions.

(1) Small craft basins are water areas where small crafts enable to be safely moored, landed and put to rest, as well as fulfill their functions in conjunction with breakwaters for harbors, mooring facilities and rest facilities. Small crafts include port service boats, public boats operated by government offices, work ships, tug boats, fishing boats, leisure fishing boats and pleasure boats etc.

(2) It is recommended that small craft basins shall designed with due consideration to safe mooring, facilitation of ship maneuvering and suitability for meteorological and hydrographic conditions, as well as consistency with the related facilities.

(3) Performance Criteria of Small Craft Basins

① **The performance criteria of basins shall apply mutatis mutandis. (serviceability)**

The provisions in item (2), Article 31 of the Standard Public Notice (Performance Criteria of the Depths of Basins) shall apply to small craft basins mutatis mutandis.

② **Shapes, areas and harbor calmness of small craft basins (serviceability)**

(a) Shapes

In the performance verification of small craft basins, their shapes shall be properly determined with due consideration to ensuring the required harbor calmness and for the prevention of accidental contacts between small crafts and the breakage of mooring lines.

(b) Areas

In the performance verification of small craft basins, their areas shall be properly determined with due consideration to the following items.

- **Design ships**

  The ship types and ship dimensions of the design ships shall consider in the design. However there are wide variability of those information, it is difficult to set a standard value. Therefore, the dimensions of the design ships are preferably set based on specifications of the ships, which actually use the small craft basins concerned, through hearings or other means. The Register of Ships in Japan[1)] can be used as a reference.

- **Mooring facilities**

  The scales of onshore facilities and water areas for ship mooring shall be determined after the determination of mooring methods and, preferably, based on the actual operational condition of the design ships. For example, literature 2) introduces the actual operational condition of tug boats.

- **Waterways and basins**

  It is necessary to properly determine water areas provided for use in ship navigation and turning and the dimensions of the navigation channels (widths, depths and the shapes of bends) connecting the inside and outside of basins as needed.

(c) Harbor calmness

Harbor calmness shall be appropriately determined in consideration of the wave conditions of the small craft basins concerned.

(4) For those small craft basins to be provided in marinas, refer to the descriptions in **Reference (Part III), Chapter 2, 4 Marina**.

[References]

1) Japan Shipping Exchange: Register of Ships (in Japanese)

2) Japan Association of Cargo-handling Machinery Systems: Cargo Handling, Vol.162, p233, 2017 (in Japanese)

Chapter 4 Protective Facilities for Harbors

1 General

[Ministerial Ordinance] (General Provisions)

Article 13

Protective facilities for harbors shall be installed at appropriate locations in light of geotechnical characteristics, meteorological characteristics, sea states, other environmental conditions, navigation channels, and other usage conditions of water areas around the facilities.

[Ministerial Ordinance] (Necessary Items concerning Protective Facilities for Harbors)

Article 24

The necessary matters for the enforcement of the requirements as specified in this Chapter by the Minister of Land, Infrastructure, Transport and Tourism and other performance requirements for protective facilities for harbor shall be provided by the Public Notice.

[Public Notice] (Protective Facilities for Harbors)

Article 33

The items to be specified by the Public Notice under Article 24 of the Ministerial Ordinance concerning the performance requirements of protective facilities for harbors shall be as prescribed in the following Article through Article 46.

1.1 Purposes of Protective Facilities for Harbors

The purpose of protective facilities for harbors includes ensuring harbor calmness, maintaining water depths, preventing beach erosion, controlling the rise of water levels in the areas using protective facilities during storm surges, diminishing invading waves by tsunamis and protecting harbor facilities and hinterland from waves, storm surges, and tsunamis.

In the deliberation of the measures against tsunamis and storm surges for harbors, it is necessary to appropriately set the targets of protecting harbors according to the magnitude and occurrence frequency of tsunamis and storm surges, after considering sufficiently their impacts on human lives, property and socioeconomic activities.

Recently, there has been demand for water intimate amenity functions enabling people to enjoy the proximity to marine environment and play with water. In general, many protective facilities for harbors are provided with additional facilities to fulfill some of these functions. Accordingly, the performance verification shall consider the usability enabling each protective facility for harbors to fulfill these purposes.

1.2 Points of Caution When Constructing Protective Facilities for Harbors

(1) When constructing protective facilities for harbors, their layouts and structural types shall be decided after carefully considering the influences that will be exerted on the nearby water areas, facilities, topography, and water currents. The influences caused by the protective facilities for harbors are as follows:

① When the protective facilities for harbors are constructed on sandy coasts, they may cause various topographic changes to the surrounding area such as beach accretion or erosion.

② Construction of breakwaters may increase the wave heights outside the protective facilities for harbors because of reflected waves.

③ The calmness of water areas inside of harbors may be disturbed because of the multiple wave reflection triggered by the construction of new protective facilities for harbors or the induction of harbor resonance due to the changes in harbor shapes.

④ Construction of the protective facilities for harbors may bring about changes in the surrounding tidal currents or flow conditions in rivers, thereby causing localized changes in water quality.

(2) The damage to protective facilities for harbors may affect the safety of ships in harbors, mooring facilities, and other facilities in hinterland. Thus, during the construction, improvement, and maintenance of protective facilities for harbors, sufficient deliberation is required for the protection of such damage according to the required performance of respective ships in harbors, mooring facilities, and other facilities in hinterland.

1.3 Role of Protective Facilities for Harbor in Multilevel Protection Concept

In the deliberation of the measures against tsunamis and storm surges in harbors, it is important to adopt the concept of providing protection at multiple levels (multilevel protection) by the entire harbor facilities including forefront breakwaters and seawalls while taking full functions of existing stocks.

Considering that most industrial and logistic facilities in harbors are located at seaward side of protection lines protecting urban areas at the back of harbor areas from disasters, it is effective to use harbor facilities such as breakwaters for protecting the functions of the industrial and logistic facilities from tsunamis and storm surges.

However, because there are cases where protective facilities for harbors such as forefront breakwaters and seawalls may not be able to completely protect the functions from tsunamis and storm surges, it is important to establish plans to evacuate people working in industrial and logistic facilities in harbors and harbor users.

1.4 Environmentally conscious protective facilities for harbors

Because the protective facilities for harbors also provide the habitat for marine organisms such as fish, marine plants, and plankton, it is preferable to determine the layouts and structural types of the facilities, with considering the habitat for these marine organisms as needed.

When installing protective facilities for harbors close to natural park districts or cultural facilities, it is preferable to consider not only the original functions of the protective facilities for harbors but also their external appearances such as shapes and colors. Additionally, in situations where water intimate amenity functions will be added to the protective facilities for harbors, convenience and safety of users must also be considered.

2 Common Items for Breakwaters

[Ministerial Ordinance] (Performance Requirements for Breakwaters)

Article 14

1 The performance requirements for breakwaters shall be as prescribed in the following items depending on the structure type for the purpose of securing safe navigation, anchorage and mooring of ships, ensuring smooth cargo handling, and preventing damage to buildings, structures, and other facilities in the port by maintaining the calmness in the harbor water area.

(1) Breakwaters shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable the reduction of the height of waves intruding into the harbor.

(2) Damage to breakwaters, etc. due to self-weight, variable waves, Level 1 earthquake ground motion, etc. shall not impair the functions of the breakwaters and shall not adversely affect the continuous use of the breakaters.

2 In addition to the provisions of the preceding paragraph, the performance requirements for the breakwaters in the following items shall be specified respectively in those items:

(1) "Performance requirements for breakwaters which are required to protect the hinterland of the breakwaters from storm surges or design tsunamis" shall be such that the breakwaters satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable the appropriate reduction of the rise in water level and flow velocity due to storm surges or design tsunamis in the harbor.

(2) "Performance requirements for breakwaters for the purpose of environmental conservation" shall be such that the breakwaters satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to contribute to conservation of the environment of ports without impairing the original functions of the breakwaters .

(3) "Performance requirements for breakwaters to be utilized by an unspecified large number of people" shall be such that the breakwaters satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to ensure the safety of the users of the breakwaters.

(4) "Performance requirements for breakwaters in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity if they are stricken by disaster" shall be such that damage to breakwaters, etc. due to design tsunamis, accidental waves, Level 2 earthquake ground motions, etc. does not have a serious impact on the structural stability of the breakwaters in consideration of the structure type even in cases where functions of the breakwaters are impaired. Provided, however, that in cases where performance requirements for the breakwaters which are required to protect the hinterland of the breakwaters from design tsunamis, the damage due to design tsunamis, Level 2 earthquake ground motion, etc. shall not adversely affect restoration through minor repair works of the functions of the breakwaters.

3 In addition to the provisions of the preceding two paragraphs, the performance requirements for breakwaters in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity, shall be such that a serious impact on the structual stability of the breakwaters caused by damage, etc. due to the actions of the tsunamis, etc. even in cases where tsunami with intensity exceeding the design tsunami occurs at the place where the breakwaters are located, shall be delayed as much as possible in consideration of the structure type.

[Public Notice] (Performance Criteria for Breakwaters)

Article 34

1 The performance criteria common to breakwaters shall be as prescribed respectively in the following items:

(1) Breakwaters shall be located appropriately so as to satisfy the harbor calmness provided in Article 31, item (iii), and shall have the dimensions which enable the transmitted wave height to be equal to or less than the allowable level.

(2) Breakwaters having wave-absorbing structures shall have the dimensions which enable full performance of the required wave-absorbing function.

2 In addition to the provisions of the preceding paragraph, the performance criteria for the breakwaters specified in the following items shall be as prescribed respectively in those items:

(1) "Performance criteria for breakwaters which are required to protect the hinterland from storm surges" shall be such that the breakwaters are located appropriately so as to reduce the rise of water level and flow velocity in the harbor due to storm surges and have the dimensions necessary for functions of breakwaters.

(2) "Performance criteria for breakwaters required to protect the hinterland from design tsunamis" shall be such that the breakwaters are located appropriately so as to reduce the rise of water level and flow velocity in the harbor due to design tsunamis and have the dimensions necessary for functions of breakwaters.

(3) "Performance criteria for breakwaters for the purpose of environmental conservation" shall be such that the breakwaters shall have the necessary dimensions so that they can contribute to conservation of the environment of ports without impairing their original functions in consideration of the environmental conditions, etc. to which the facilities are subjected.

(4) "Performance criteria for breakwaters utilized by an unspecified large number of people" shall be such that breakwaters have the dimensions necessary to secure the safety of users in consideration of the environmental conditions, usage conditions, etc. to which the facilities are subjected.

(5) "Performance criteria for breakwaters in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity by the damage to the breakwaters" shall be such that the degree of damage under the accidental situation, in which the dominating actions are design tsunamis, accidental waves, or Level 2 earthquake ground motions, is equal to or less than the threshold level in consideration of the performance requirements.

[Interpretation]

10. Protective Facilities for Harbors

(1) Performance Criteria Common to Breakwaters (Article 14, Paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 1 of the Public Notice)

① Breakwaters shall have serviceability as their common performance requirement. The term "serviceability" refers to that the harbor calmness in the ports is secured.

② The dimensions for securing harbor calmness shall indicate a structure including shape and crown height that affects the transmitted wave height or transmission ratio of waves. In setting the crown height in the performance verifications of breakwaters, appropriate consideration shall be given to the effect of ground settlement.

③ The allowable transmitted wave height is the limit value of the wave height of waves transmitted from outside the harbor to inside the harbor over the breakwaters. Provided, however, that the index of the limit value in the performance verifications is not limited to the transmitted wave height, but also includes cases in which the wave transmission ratio is used.

④ In the performance verifications of breakwaters, the allowable transmitted wave height or wave transmission ratio shall be set appropriately to secure harbor calmness. Furthermore, the allowable transmitted wave height or wave transmission ratio shall generally be calculated considering the type of structure and crown height of the breakwater.

(2) Performance Criteria for Specific Breakwaters

① Storm surge protection breakwaters (Article 14, Paragraph 2 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 1 of the Public Notice)

(a) Storm surge protection breakwaters refer to breakwaters that shall protect the hinterland from storm surges. In addition to the provisions common to breakwaters, the items listed below apply to them.

(b) Storm surge protection breakwaters shall have serviceability as their performance requirement. The term "serviceability" refers to the performance that demonstrates a peak cut effect in reducing the rise of the water level and flows of water due to storm surges.

(c) The dimensions of storm surge protection breakwaters shall indicate the crown height, opening width, and water depth at the opening. In setting the layout, crown height, opening width, and water depth at the opening in performance verifications of storm surge protection breakwaters, appropriate consideration shall be given to the effect of storm surge and tide levels such that the performance above is demonstrated.

② **Tsunami protection breakwaters** (Article 14, Paragraph 2 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 2 of the Public Notice)

(a) Tsunami protection breakwaters refer to breakwaters that shall protect the hinterland from design tsunamis.

(b) Tsunami protection breakwaters shall have serviceability as their performance requirement. The term "serviceability" refers to the performance that demonstrates a peak cut effect in reducing the rise of the water level and flows of water due to tsunamis.

(c) The dimensions of tsunami protection breakwaters shall indicate the crown height, opening width, and water depth at the opening. In setting the layout, crown height, opening width, and water depth at the opening in the performance verifications of tsunami protection breakwaters, appropriate consideration shall be given to the effect of tsunamis and tidal levels such that the performance above is demonstrated.

③ **Symbiosis breakwaters** (Article 14, Paragraph 2, Item 2 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 3 of the Public Notice)

(a) Symbiosis breakwaters refer to breakwaters that aim at preserving the environments. In addition to the provisions common to breakwaters, the items listed below apply to them.

(b) Symbiosis breakwaters shall have serviceability as their performance requirement. The term "serviceability" refers to the performance that contributes to the preservation of the port environments (e.g., living things and ecosystems) without impairing original functions of breakwaters.

(c) The dimensions of breakwaters that aim at preserving the environments refer to the structure, cross-sectional dimensions, and ancillary facilities. In setting the structure and cross-sectional dimensions in the performance verifications of breakwaters that aim at preserving the environments and in installing ancillary equipment, appropriate consideration shall be given to factors that affect the targets that contribute to the preservation of the port environments (e.g., living things and ecosystems) without impairing original functions of the breakwaters.

④ **Amenity-oriented breakwaters** (Article 14, Paragraph 2, Item 3 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 4 of the Public Notice)

(a) Amenity-oriented breakwaters refer to breakwaters that an unspecified large number of people use. In addition to the provisions common to breakwaters, the items listed below apply to them.

(b) Amenity-oriented breakwaters shall have serviceability as their performance requirement. The term "serviceability" refers to the performance that can secure the user safety in consideration of the environmental conditions to which the revetments concerned are subjected, their utilization conditions, and other conditions.

(c) The dimensions of amenity-oriented breakwaters shall indicate the structure, cross-sectional dimensions, and ancillary facilities. In setting the structure and cross-sectional dimensions in the performance verifications of amenity-oriented breakwaters and in installing ancillary equipment, consideration shall be given to the effects of wave overtopping and spray, prevention of slipping, overturning, and falling into the water of users, smooth execution of rescue activities for users who fall into the water, and ancillary equipment such as falling prevention fences shall be installed appropriately.

⑤ **Breakwaters of facilities prepared for accidental incidents** (Article 14, Paragraph 2, Item 4 and Paragraph 3 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 5 of the Public Notice)

(a) Breakwaters of facilities prepared for accidental incidents refer to breakwaters in the place where there is a risk of serious impact on human lives, properties, or socioeconomic activities by the damage.

(b) Breakwaters of facilities prepared for accidental incidents (excluding tsunami protection breakwaters) shall have safety as their performance requirement in accidental situations where the dominating actions are Level 2 earthquake ground motions, design tsunamis, and accidental waves.

Attached Table 10-1 shows performance verification items and standard indexes to determine limit values for such actions. Necessary performance verification items shall be appropriately selected depending on the type of structure of the breakwater concerned. The reason for indicating “damage” in the “Performance verification item” column of **Attached Table 10-1** is that it is necessary to use a comprehensive term considering that the performance verification items will vary depending on the type of structure. Indexes to determine limit values shall be appropriately determined in the performance verifications.

Attached Table 10-1 Performance Verification Items and Standard Indexes to Determine Limit Values for Accidental Actions on Breakwaters of Facilities Prepared for Accidental Incidents (excluding Tsunami Protection Breakwaters)

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Performance verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
14	2	4	34	2	5	Safety	Accidental	L2 earthquake ground motion [Accidental wave]	Self-weight, water-pressure	Damage	-
								Design tsunami	Self-weight, water pressure, water flows		

* [ ] indicates an alternative dominant action to be studied as design situations.

(c) Tsunami protection breakwaters shall have restorability as their performance requirement against accidental situations where the dominating actions are Level 2 earthquake ground motion and design tsunamis. Also, Tsunami protection breakwaters shall have safety as their performance requirement in accidental situations where the dominating actions are accidental waves. **Table 10-2** shows performance verification items and standard indexes to determine limit values for such actions. Necessary performance verification items shall be appropriately selected depending on the type of structure of the breakwater concerned. Indexes to determine limit values shall be appropriately determined in the performance verifications.

Attached Table 10-2 Performance Verification Items and Standard Indexes to Determine Limit Values for Accidental Actions on Tsunami Protection Breakwaters among Breakwaters of Facilities Prepared for Accidental Incidents

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
14	2	4	34	2	5	Restorability	Accidental	Level 2 earthquake ground motion	Self-weight, water pressure	Deformation of breakwater body	Residual deformation
								Design tsunami	Self-weight, water pressure, water flows	Sliding and overturning of breakwater body, bearing capacity of foundation ground	Action-resistance ratio of sliding Action-resistance ratio of overturning Action-resistance ratio of bearing capacity

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
						Safety		Accidental waves	Self weight, water pressure	Sliding and overturning of breakwater body, bearing capacity of foundation ground	Action-resistance ratio of sliding Action-resistance ratio of overturning Action-resistance ratio of bearing capacity

(d) It may be noted that, as the performance criteria in connection with the accidental situations that are common to breakwaters of facilities prepared for accidental incidents, in addition to these provisions, the settings in connection with the Public Notice, Article 22 Performance Criteria Common to Members Comprising Facilities subject to the Technical Standards shall be applied as necessary.

(e) The structure of breakwaters of facilities prepared for accidental incidents shall be well configured to allow them to be as much stable as possible even when they are subjected to actions (e.g., tsunami with intensity exceeding the design tsunami) at places where they are installed so that the disaster mitigation effects can be demonstrated and the harbor calmness is secured immediately after the disaster.

2.1 Matters relating to Breakwaters with Basic Functions

2.1.1 General

(1) Breakwaters are generally classified as shown in **Fig. 2.1.1** by the type of structure and functions or purposes. The characteristics of each structural type are described in the applicable section for each type.

- Breakwater
  - Ordinary breakwater (breakwater having basic functions)
  - Breakwater with combination of added functions
    - Symbiosis breakwaters
    - Amenity-oriented breakwater
    - Tsunami protection breakwater
    - Storm surge protection breakwater
    - Breakwater for timber handling facilities

(a) Classification by function

- Breakwater
  - Composite breakwater
  - Upright breakwater
  - Sloping breakwater
  - Breakwater covered with wave-dissipating blocks
  - Gravity type special breakwater
    - Upright wave-dissipating block type breakwater
    - Wave-dissipating caisson type breakwater
    - Sloping top caisson breakwater
  - Pile type breakwater
  - Floating breakwater
  - Breakwater with wide footing on soft ground

Types of structures enclosed in broke line are gravity type breakwaters.

(b) Classification by type of structure

Fig. 2.1.1 Classification of Breakwaters

(2) In design and performance verifications of breakwaters, it is preferable to consider their layout, effects on the topographical features of the surrounding areas, harmonization with the surrounding environments, design conditions, structural type, whether they are used for multiple purposes, verification procedures, construction methods, and economy.

(3) Maintenance of harbor calmness shall be examined from the following two viewpoints: the enabling of cargo handling in the basin and the condition of waves enabling refuge during rough weather. For harbor calmness in the basin and the condition of waves during rough weather, **Part II, Chapter 2, 4.6 Concept of Harbor Calmness** and **Part III, Chapter 3, 3 Basins** can be used as references.

(4) Reflected waves from a breakwater may become large depending on its structure, which may hinder ships from sailing outside the port. Such large waves affect small-sized ships significantly, in particular, so it is preferable to adopt structure with lesser reflected waves depending on the sailing pattern.

2.1.2 Layout

(1) Breakwaters shall be appropriately arranged to keep the inside of the ports, such as waterways and basins, calm.

(2) Breakwaters are constructed to maintain the harbor calmness, facilitate smooth cargo handling, ensure the safety of ships during navigation or anchorage, and protect facilities in ports. To fulfill these requirements, the following objectives must be met:

① Breakwaters should be located such that the harbor entrance is at the location not facing the direction of the most frequent waves and the direction of the highest waves to reduce waves entering into the harbor.

② Breakwater alignment should be arranged to protect the harbor from the most frequent waves and the highest waves.

③ The harbor entrance should have a sufficient effective width so that it will not present an obstacle to ship navigation, and it should orient the navigation channel in a direction that makes navigation easy.

④ Breakwaters should be located at the place where the speed of tidal currents is as slow as possible. In cases in which the speed of tidal currents is high, it is necessary to take appropriate countermeasures.

⑤ The influences of reflected waves, Mach-stem waves, and wave concentration on the waterways and basins should be minimized.

⑥ Breakwaters should enclose a sufficiently large water area that is needed for ship berthing, cargo handling, and ship anchorage.

These objectives are also mutually contradictory goals, however. A narrow harbor entrance width, for example, is best to achieve calmness in a harbor, but is inconvenient for navigation. The direction of the most frequent waves and the direction of the highest waves are not necessarily the same. In such a situation, the breakwater layout should be determined through a comprehensive investigation of all the factors such as conditions of ship use, construction cost, construction works, and ease or difficulty of maintenance.

(3) In situations in which concerns for deterioration of water quality exist, consideration is preferably given to the exchangeability of seawater with the outside sea so that seawater within the harbor does not stagnate.

(4) In the construction of breakwaters, economy should also be examined considering the natural conditions and construction conditions. In particular, it is preferable to consider the following.

① Layouts that cause wave concentrations should be avoided.

② Locations where the ground is extremely poor should be avoided, considering constructability and economy.

③ The layout should consider the effects of topographical features such as capes and islands.

④ On sandy beaches, the layout should consider invasion of littoral drift into the harbor.

⑤ Adequate consideration should be given to the effect on adjacent areas after the construction of the breakwater.

For wave concentration, **Part II, Chapter 2, 4.4.4 Wave Reflection, [3] Transformation of Waves at Concave Corners near the Heads of Breakwaters and around Detached Breakwaters** can be used as a reference; for breakwaters to be constructed on sandy beaches, **Part II, Chapter 2, 7.4 Littoral Drift** can be used as a reference.

(5) Breakwaters should be located such that they do not form an obstacle to the future development of the harbor.

(6) The “effective harbor entrance width” means the width of the waterway at the specified water depth, not merely the width of the harbor entrance. The speed of the tidal currents cutting across the harbor entrance is preferably 2 to 3 knots or less.

(7) In the areas surrounding shoals, the wave height often increases because of wave refraction. In some cases, impact wave forces will act on the breakwater constructed on a seabed with steep slope. It should be noted that a significantly large structure may be required when a breakwater is placed on or directly behind a shoal.

(8) For detached breakwaters that are to be constructed in isolation offshore, if the length of the breakwater is equal to or less than several times that of the incident waves, the distribution of the wave heights behind the breakwater will fluctuate greatly because of the effect of diffracted waves from the two ends of the breakwater, which will affect the stability of the breakwater body; therefore, exercising caution is necessary. For the effects of diffracted waves, **Part II, Chapter 2, 4.4.2 Wave Diffraction** and **Part II, Chapter 2, 4.4.4 Wave Reflection, [3] Transformation of Waves at Concave Corners near the Heads of Breakwaters and around Detached Breakwaters** can be used as references.

2.1.3 Selection of Structural Type and Setting of Cross Section

(1) In setting the cross sections of breakwaters, the type of structure shall be selected based on a comparative examination of the layout conditions, natural conditions, use conditions, importance, construction conditions, economy, term of construction work, ease of obtaining materials, and ease of maintenance, considering the features of respective types of structures.

(2) In determining the cross-sectional dimensions of the wave-dissipating work in the wave-dissipating function of a breakwater, it is necessary to provide adequate consideration to hydraulic characteristics so that the specified wave-dissipating function is demonstrated. In particular, it is preferable that the crown height of the wave-dissipating section be approximately the same as that of the breakwater body so that impulsive breaking wave pressure will not act on the breakwater body.

(3) In cases in which the layout of a breakwater includes a corner, the wave height around the corner will increase. Therefore, it is preferable to adopt a low reflective structure around corners.

(4) The crown section of a submerged breakwater installed at an opening may be damaged by the dragging power of tsunamis and waves and the foundation mound and the ground under it may be scoured by formed flow of seawater, etc. passing through the foundation mound, etc. Therefore, appropriate scour prevention measures are required, when needed.

(5) It shall be noted that when the seawater's permeability becomes higher due to the structure of the foundation, etc., the effect for reducing storm surges becomes smaller. In addition, the difference in the tide levels between the inside and outside of the port due to storm surges may form the flow of seawater, etc. that pass through the inside of the foundation mound, etc. Such flow may scour the ground at the lower section of the foundation mound, etc. Therefore, appropriate scour prevention measures are required, when needed.

(6) Selection of a permeable-type breakwater structure is advantageous for promoting circulation of seawater in the harbor. However, because this also invites inflow of littoral drift and an increase in the height of transmitted waves, adequate consideration of the merits and demerits is necessary.

(7) Breakwaters become important bases for the life of living beings inside and outside the ports and bases to which they are attached in some cases. Therefore, in setting the structure and cross-sectional dimensions of breakwaters, they may be designed considering that the port environment is well preserved.[1), 2), 3), 4), 5), 6), 7), 8), 9), 10)] (**Reference (Part I), Chapter 3, 2 Symbiosis Port Structures**)

2.1.4 Matters to be Considered to Maintain the Harbor Calmness in Ports

(1) In the installation of breakwaters, the crown height of the breakwater, relationship between the position of the breakwater and waterways and basins, and position and direction of the harbor entrance should be examined so as to maintain the harbor calmness necessary for cargo handling and refuge. In the performance verifications of the harbor calmness of basins, **Part II, Chapter 2, 4.6 Concept of Harbor Calmness** can be used as a reference. Furthermore, it is preferable that conditions be set to enable protection of the port facilities behind the breakwater, including during typhoons and other rough weather.

(2) The crown height of a breakwater necessary in securing harbor calmness can generally be set to an appropriate height at least 0.6 times the significant wave height ($H_{1/3}$) used in examination of the safety of the breakwater above the mean monthly-highest water level. In this case, the appropriate height is set considering harbor calmness in the basin behind the breakwater, preservation of facilities in the harbor behind the breakwater, and other factors. In the existing breakwaters, there are many examples in which the crown height is determined as follows.

① In a harbor where large ships call, where the water area behind the breakwater is so wide that wave overtopping is allowed to some extent, the crown height is set at $0.6H_{1/3}$ above the mean monthly-highest water level in situations in which it is not necessary to consider the influence of storm surges.

② In a harbor where the water area behind the breakwater is small and is used for small ships, overtopping waves should be prevented as much as possible. Hence, the crown height is set at $1.25H_{1/3}$ above the mean monthly-highest water level.

③ The crown height values above are often seen in design examples in the past. Documents 11) and 12) show examination results of actual breakwater crown height determined based on such height values, volume of wave overtopping, and transmission ratio and can be referred to.

(3) For ports for which effects of storm surges shall be considered, the tide level that is calculated by adding appropriate deviation to the mean monthly-highest water level based on the past records shall preferably be used as a reference surface to calculate the crown height.

(4) Even in case of a harbor where large ships call which has a wide water area behind the breakwaters at the harbor where large storm waves close to the design waves attack frequently with long duration, the activities of harbor may be limited by the influence of waves overtopping the breakwaters, if the crown height is set at of $0.6H_{1/3}$ above

the mean monthly-highest water level. Accordingly, in such a harbor, the crown height is preferably set higher than $0.6H_{1/3}$ above the mean monthly-highest water level.

(5) For the effects of reflected waves on the harbor calmness in ports, **Part II, Chapter 2, 4.4.4 Wave Reflection** can be used as a reference.

2.2 Matters relating to Breakwaters of Facilities Prepared for Accidental Incidents

The descriptions provided below shall be referred to for breakwaters of facilities prepared for accidental incidents.

(1) Gravity-type Breakwaters of Facilities Prepared for Accidental Incidents

① Accidental situations where the dominating actions are Level 2 earthquake ground motions

(a) Deformation volume

When the limit value of the degree of damage in accidental situations in which the dominating actions are Level 2 earthquake ground motions is used as the breakwater body's deformation volume, the breakwater body's allowable residual deformation volume shall be appropriately determined. In setting the allowable residual deformation volume, the degree of allowable damage can be a level at which the breakwater body does not fall down, it does not slide down from the foundation mound, and settlement more than the allowable value does not occur.

② Accidental states where the dominating actions are design tsunamis

(a) Consideration of the effects of earthquake ground motions

In performance verifications for design tsunamis, when supposed design tsunamis are caused by an earthquake for which the hypocenter is near the facility concerned, it shall be appropriately considered that the facility concerned is subject to actions by the earthquake ground motions before receiving actions by the design tsunamis. In this case, performance verifications for the design tsunamis shall be performed considering the effects of the actions of the earthquake ground motions that come before the design tsunamis. It shall be noted that the assumed earthquake ground motions coming before design tsunamis in such a case are not always equal to Level 2 earthquake ground motions.

(b) Points to note in performance verifications of breakwaters

In setting limit values of the degrees of damage in accidental situations where the dominating actions are design tsunamis, how protective facilities for the harbors (e.g., revetments on the back and floodgates) and other facilities in the vicinity have been maintained, software measures for disaster prevention and mitigation in the areas, and other factors shall be comprehensively considered, in addition to the functions of the breakwaters.

(c) Stability against design tsunamis and tsunamis with the intensity exceeding the design tsunamis

For tsunami-resistant design in which the stability against design tsunamis and tsunamis with the intensity exceeding the design tsunamis is considered, one can refer to the **Guideline for Tsunami-Resistant Design of Breakwaters**.[13] However, the guideline handles composite breakwaters and breakwaters covered with wave-dissipating blocks as the target structural types. Therefore, appropriate consideration is necessary for other structural types.

③ Accidental states where the dominating actions are accidental waves

(a) Consideration of the effects of storm surges

In performance verifications for accidental waves, storm surges that are caused at the same time with supposing waves shall be appropriately considered. In setting accidental wave conditions, **Part II, Chapter 2, 4.1.2 Setting of Wave Conditions for Verification of Serviceability of the Structural Members** and **Part II, Chapter 2, 3.2 Storm Surges** can be referred to.

(b) Points to note in performance verifications of breakwaters

In setting limit values of the degrees of damage in accidental situations where the dominating actions are accidental waves, how protective facilities for the harbors (e.g., revetments on the back and floodgates) and other facilities in the vicinity have been maintained, software measures for disaster prevention and mitigation in the areas, and other factors shall be comprehensively considered, in addition to the functions of the breakwaters.

(2) Floating Breakwaters of Facilities Prepared for Accidental Incidents

① **Points to note in performance verifications of breakwaters**

In verifications of the stability of mooring anchors and other equipment against accidental situations where the dominating actions are design tsunamis and accidental waves, consideration is necessary to prevent the floating structures from drifting because of design tsunamis and accidental waves and thereby significantly affecting the sounding areas.

[References]

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3 Ordinary Breakwaters

3.1 Gravity-type Breakwaters (Composite Breakwaters)

[Public Notice] (Performance Criteria for Gravity-type Breakwaters)

Article 35

The performance criteria for gravity-type breakwaters are prescribed respectively in the following items:

(1) Under the permanent state, in which the dominating action is self-weight, the risk of slip failure of ground shall be equal to or less than the threshold level.

(2) Under the variable situation , in which the dominating actions are variable waves and Level 1 earthquake ground motion, the risk of failures due to the sliding and overturning of breakwater body and the insufficient bearing capacity of the foundation ground shall be equal to or less than the threshold level.

[Interpretation]

10. Protective Facilities for Harbors

(3) Performance Criteria of Gravity-type Breakwaters (Article 14, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 35 of the Public Notice)

① The required performance of gravity-type breakwaters under the permanent action situation in which the dominant action is self-weight and the variable action situation in which the dominant actions are variable waves and Level 1 earthquake ground motions shall focus on usability. The performance verification items and standard indexes to determine the limit values with respect to the actions shall be those shown in **Attached Table 10-3**, except those for sloping breakwaters, which are separately shown in **Attached Table 10-4**.

Attached Table 10-3 Performance Verification Items and Standard Indexes to Determine the Limit Values of Gravity-type Breakwaters (Except Sloping Breakwaters)

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
14	1	2	35	-	1	Serviceability	Permanent	Self-weight	Water pressure	Circular slip failure of ground	Action–resistance ratio with respect to circular slip failure
					2		Variable	Variable waves (L1 earthquake ground motion)	Self-weight, water pressure	Sliding/overturning of breakwater body, bearing capacity of foundation ground	Action–resistance ratios with respect to sliding, overturning and bearing capacity

* [ ] means the alternative dominant action to be studied as design situations.

Attached Table 10-4 Performance Verification Items and Standard Indexes to Determine the Limit Values of Sloping Breakwaters

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
14	1	2	35	-	1	Serviceability	Permanent	Self-weight	Water pressure	Circular slip failure of ground	Action–resistance ratio with respect to circular slip failure
					2		Variable	Variable waves	Self-weight, water pressure	Sliding and overturning of superstructure	Action–resistance ratios with respect to sliding and overturning
										Bearing capacity of foundation ground	Action–resistance ratio with respect to bearing capacity
								L1 earthquake ground motion	Self-weight, water pressure	Bearing capacity of foundation ground	Action–resistance ratio with respect to bearing capacity

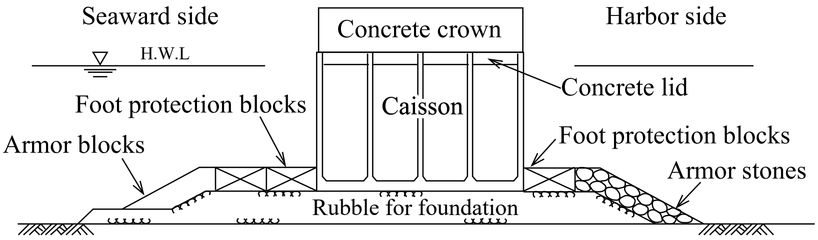
② In addition to the above, gravity-type breakwaters shall be subjected to the following: the requirements and commentaries in Paragraph 3, Article 22 of the Public Notice (Scouring and Outflow) and Article 28 of the Public Notice (Performance Criteria of Armor Stones and Blocks) as needed; the requirements and commentaries in Articles 23 to 27 of the Public Notice depending on the types of members constituting breakwaters.

③ In addition to the above, breakwaters with wave-dissipating structures (breakwaters covered with wave-dissipating blocks, upright wave-absorbing block breakwaters, wave-absorbing caisson breakwaters, etc.) shall be subjected to the requirements in Item 2, Paragraph 1, Article 34 of the Public Notice (Serviceability with Respect to Wave-Dissipating Function).

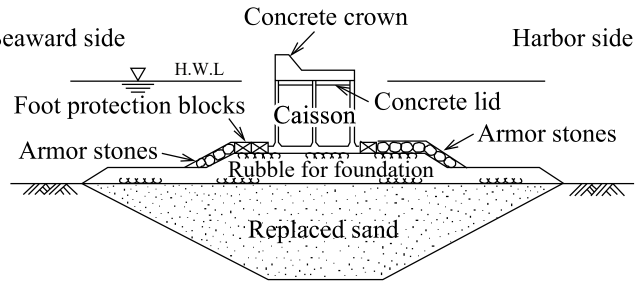
3.1.1 General

(1) Composite breakwaters with upright breakwater bodies placed on rubble mound foundations are the most typical structure of gravity-type breakwaters in Japan. Therefore, the general descriptions of gravity-type breakwaters in this section are those for composite breakwaters.

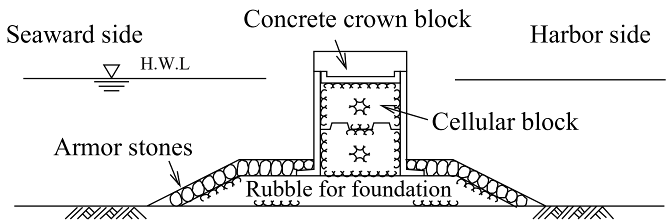
(2) **Fig. 3.1.1** shows examples of cross sections of composite breakwaters.



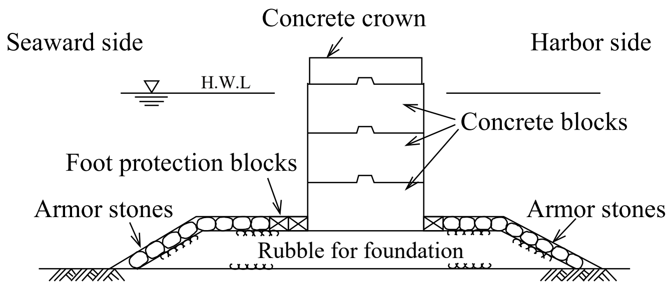
(a) Caisson type composite breakwater (sandy ground)



(b) Caisson type composite breakwater (soft ground)



(c) Cellular block type composite breakwater

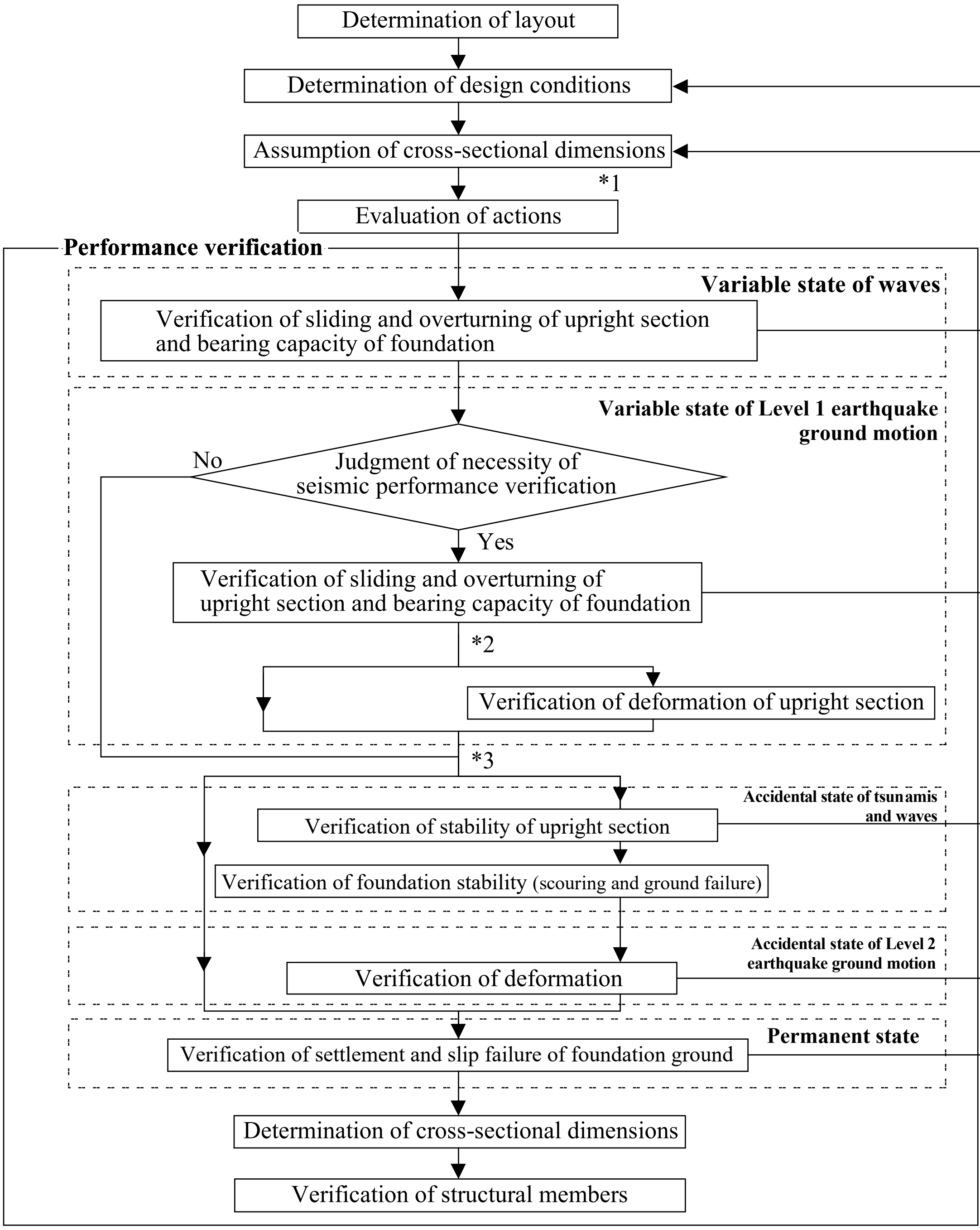


(d) Concrete block type composite breakwater

Fig. 3.1.1 Examples of Cross Sections of Composite Breakwaters

(3) Given that structures with upright breakwaters are placed on rubble mound foundations, composite breakwaters have characteristics that are closer to sloping breakwaters and upright breakwaters as the ratio of the depths of the crown levels of rubble mounds to wave heights becomes smaller and larger, respectively.

(4) **Fig. 3.1.2** shows an example of the performance verification procedure for composite breakwaters.



*1: The evaluation of the effects of liquefaction and settlement are not shown; therefore, this must be separately considered.

*2: The analysis of deformation due to Level 1 earthquake ground motions may be carried out by dynamic analysis when necessary. For facilities where damage to the objective facilities is assumed to have a serious impact on life, property, and social activity, it is preferable to conduct an examination of deformation by dynamic analysis.

*3: For facilities where damage to the objective facilities is assumed to have a serious impact on life, property, and social activity, it is preferable to conduct a verification for the accidental situations when necessary. For the verification of the accidental situation, reference can be made to **Part III, Chapter 4, 2.2 Items concerning Breakwaters as Facilities Prepared for Accidental Incidents**. Verification for accidental situations associated with waves shall be conducted in cases where facilities handling hazardous cargoes are located directly behind the breakwater and damage to the objective facilities would have a catastrophic impact.

Fig. 3.1.2 Example of the Performance Verification Procedure for Composite Breakwaters

3.1.2 Setting of Basic Cross Sections

(1) The basic cross sections of breakwaters shall be set after clarifying their relations with the lowest astronomical tides, mean monthly-highest (lowest) tide levels, mean tide levels, highest high (lowest low) tide levels, high tide levels during storm surges, Tokyo Bay's mean sea level, etc. The relation between the lowest astronomical tides and datum levels for construction work shall also be clarified if they are different from each other. Furthermore, it is preferable to study the durations of storm surges and the probabilities of their occurrence as needed. For further details on tides, refer to **Part II, Chapter 2, 3 Tide Levels**.

(2) The design tide levels for calculating wave force are generally set in a manner that places breakwaters into the most unstable states, e.g., mean monthly-highest or lowest tides in cases of breakwaters for ports with no necessity of considering the effects of storm surges and tide levels obtained by applying appropriate deviations to mean monthly-highest or lowest tides in cases of breakwaters for ports with the necessity of considering the effects of storm surges.

(3) In cases wherein the foundation ground is soft and settlement can be expected, the crown heights of breakwaters shall be set with preliminarily allowances for settlement or the structures of breakwaters shall facilitate the future leveling of crown heights.

(4) The following factors cause settlement of breakwaters:

① Consolidation settlement of foundation ground
② Washing out of foundation ground
③ Lateral flow of foundation ground
④ Sinking of rubbles and blocks into foundation ground
⑤ Contraction of rubble mounds due to the reduction in porosity

For the settlement allowance for factor ① above, refer to **Part II, Chapter 5, 1 Ground Settlement**. Given that the effects of factors ② to ⑤ above vary depending on the mass of upright sections and the thicknesses of rubble mounds, the allowances for these factors cannot be generalized but can be roughly set on the basis of the actual cases of past construction. For the consolidation settlement of foundation ground after the installation of breakwater bodies, the settlement allowances can be set for either rubble mounds or superstructures, and settlement allowances shall be appropriately set in either case in consideration of construction conditions and the like.

(5) In cases wherein the foundation ground is soft and remarkable settlement or extensive rubble sinking is conceivable, countermeasures shall be taken in a manner that improves soft ground or disperses actions on breakwater bodies via mattresses laid under rubble mounds.

(6) The crown heights of breakwaters in shallow sea areas, particularly in shallow sandy beaches, shall be determined with consideration to the prevention of possible siltation inside ports due to sand carried by overtopping waves.

(7) The crown heights of breakwaters to be used for the protection of swimming beaches, for water intake, and other special purposes shall be determined after fully understanding the purposes of constructing the breakwaters.

(8) In terms of disaster prevention, the thicknesses of the superstructures of upright breakwater bodies are preferably 1 m or more in cases wherein significant wave heights in the front of breakwaters are 2 m or more or at least 50 cm or more even in cases wherein significant wave heights are less than 2 m. For breakwaters with breakwater bodies made of layers of blocks, it is preferable that superstructure concrete has sufficiently large mass as weights to enable the breakwaters to resist against sliding failures. **Fig. 3.1.3** shows the relationship between the thicknesses of superstructures and design wave heights by using practical examples.

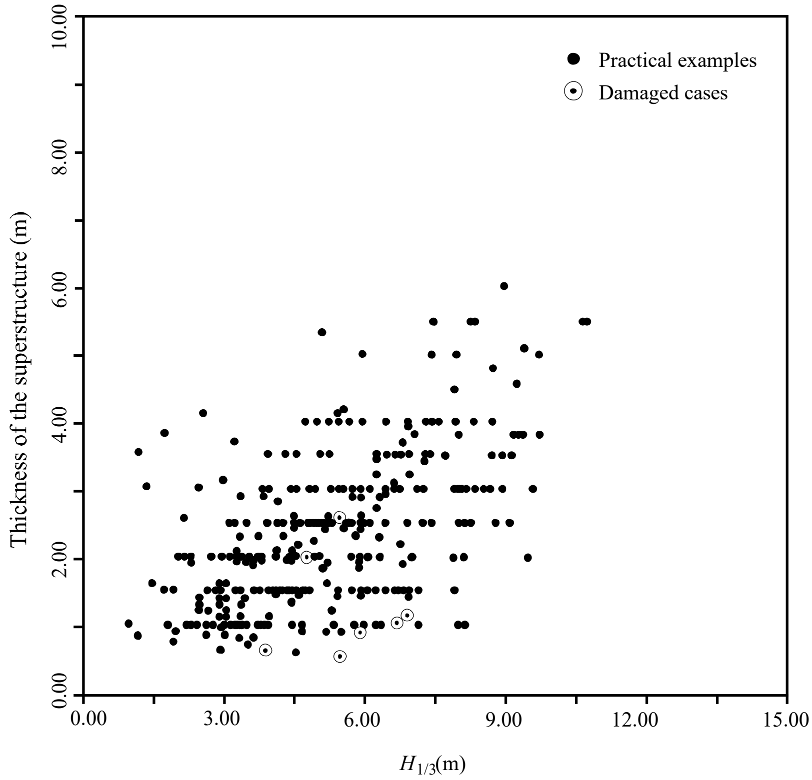


Fig. 3.1.3 Relationship between the Thicknesses of Superstructures and Design Wave Heights (Practical Examples)

(9) Considering that caisson designs with low top surface levels impose constraints on the work to install caissons, fill sand, and cast lid and superstructure concrete, the top surface levels are generally set higher than the mean monthly-high tide levels. In the case of block-type breakwaters, it is preferable that the top surface levels of the uppermost blocks or cellular blocks are set at least higher than the mean tide level or higher than the mean monthly-highest tide level to facilitate the construction of superstructures.

(10) The crown levels of rubble mounds should be as deep as possible to protect the rubble mounds from impulsive breaking waves, except in the case of using caissons as upright sections, for which the crown levels shall be set to make caissons installable. Furthermore, the berm widths at the seaward side of the rubble mounds (excluding footing sections) shall be sufficiently wide depending on the wave height to reduce the unfavorable effects of the action of impulsive breaking wave force as much as possible with reference to **Part II, Chapter 2, 6.2.4 Impulsive Breaking Wave Force**.

(11) The required mass of rubble below armor units and blocks shall be appropriately set in accordance with site conditions to prevent materials from being washed out. It is preferable that the weights of rubble below armor units and blocks are approximately 1/20 or more of the mass of armor units, and the mass of the materials to be laid under the rubble below armor units and blocks are approximately 1/20 or more of that of the rubble and blocks.[1)]

(12) The berm widths of the rubble mounds shall be set to secure the specified stability against the slip failure of ground and eccentric and inclined loads (refer to **Part III, Chapter 2, 3.2.5 Bearing Force against Eccentric and Inclined Actions**). Furthermore, it is preferable that the berm widths at the seaward side are set at not less than 5 m, excluding the footing sections, to reduce the effects of the action of impulsive breaking wave force to the greatest extent possible. However, this shall not apply in the case of hybrid caissons and other special structural types. The berm widths of the rubble mounds at the harbor side can be approximately 2/3 of those at the seaward side.

(13) There may be a case of high rubble backing for reinforcing the sliding resistance of upright sections. However, caution is necessary in such a case because rubble is easily scattered by overtopping waves. It is preferable that the rubble is provided with the armor of cubic blocks or deformed blocks as needed. The performance verification of cross sections shall be appropriately performed by referring to the following provisions in **Part III, Chapter 4, 3.1.6 Performance Verification and Cautions When the Harbor Side of Upright Sections is Reinforced**.

(14) Rubble mound foundations should have thicknesses of 1.5 m or more to enable them to produce the effects of broadly distributing the loads transferred through the weights of upright sections, providing the upright sections with flat installation ground, and preventing waves from causing scouring.

(15) Although the slope gradients of rubble mound foundations shall be determined on the basis of stability calculations, they can be generally set at 1:2 to 1:3 and 1:1.5 to 1:2 for the seaward side and harbor side of breakwaters, respectively.

3.1.3 Actions

(1) Types of Actions to be Considered in Respective Design Situations

In the stability verification of composite breakwaters, the following actions shall be considered in respective design situations provided that the performance verification of accidental action situation can be omitted in cases wherein design object breakwaters are not categorized as facilities prepared for accidental incidents.

① **Permanent action situation**

The dominant action to be considered shall be the self-weight of breakwater bodies. For the setting of self-weight, refer to **Part II, Chapter 10, 2 Self-Weight**.

② **Variable action situation**

(a) Variable waves and Level 1 earthquake ground motions shall be the dominant actions to be considered. For the setting of variable waves and Level 1 earthquake ground motions, refer to **Part II, Chapter 2, 4.1 Setting of Wave Conditions** and **Part II, Chapter 6, 1.2 Level 1 Earthquake Ground Motions Used in Performance Verification of Facilities**, respectively.

(b) The necessity of the performance verification of earthquake resistance in terms of sliding and overturning due to Level 1 earthquake ground motions can be determined on the basis of the relationship between the cross-sectional dimensions of breakwater bodies and Level 1 earthquake ground motions under a variable action situation with respect to variable waves.[2), 3)]

(c) The necessity of the performance verification of earthquake resistance can be determined using the relationship between the B_w/h ratios of the widths of breakwater bodies B_w, excluding footings to their installation depths h and the maximum engineering acceleration at bed rock (**Fig. 3.1.4**). The performance verification of earthquake resistance can be omitted when the concerned breakwaters are plotted below the curve in the figure. The figure is established on the basis of 30 cm as an allowable residual deformation of the upright sections of breakwaters subjected to Level 1 earthquake ground motions. Therefore, when adopting other values for the allowable residual deformation, the specific verification of their appropriateness should be implemented.

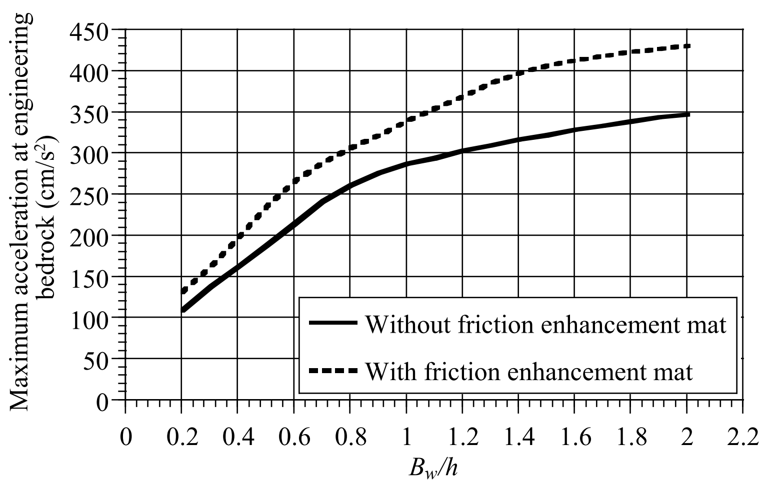


Fig. 3.1.4 Figure for Determining the Necessity of the Performance Verification of Earthquake Resistance

③ **Accidental action situation**

(a) For the setting of actions under the accidental action situation in which the dominant action is Level 2 earthquake ground motions, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Prepared for Accidental Incidents**.

(b) For the setting of actions under the accidental action situation in which the dominant action is design tsunamis, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Prepared for Accidental Incidents**.

(c) For the setting of actions under the accidental action situation in which the dominant action is accidental waves, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Prepared for Accidental Incidents**.

(2) Points of Caution When Setting Actions

① In the performance verification, there are cases in which the most dangerous tide levels differ depending on the verification items and objects of verification.

② The wave parameters necessary in the performance verifications are wave heights, wave directions, wavelengths, periods, etc. In determining these parameters, refer to **Part II, Chapter 2, 4 Waves** and **3.6 Design Tide Level Conditions**. For the data on wind used in wave hindcasting, refer to **Part II, Chapter 2, 2.3 Wind Pressure**.

③ The duration of waves is also considered an element that influences the stability of breakwaters. However, at present, such influences have not been fully understood. Therefore, it shall be noted that there are cases wherein repeated waves over an extended period of time are considered to have caused damage to breakwaters, particularly breakwater mounds, facing the open sea. Furthermore, because there are cases of damage to facilities during construction, it is necessary to decide the parameters for waves during construction in consideration of the construction plans and processes.

④ Rubble mounds with high crown heights and moderately wide berm widths may induce impulsive breaking wave force. Therefore, due consideration shall be given to the possible occurrence of impulsive breaking wave force by referring to **Part II, Chapter 2, 6.2 Wave Force on Upright Walls**. It shall also be noted that there may be a case that the intensity of wave pressure on breakwaters is increased as their crown heights increase.

⑤ In the performance verification, it shall be noted that the most dangerous waves to the stability of upright sections may be different from those in the calculation of the required mass of armor units.

⑥ In cases of the differences in still tide levels inside and outside breakwaters, it is preferable to consider the hydrostatic pressure equivalent to the differences in tide levels.

⑦ It is necessary to consider the buoyancy of the breakwater bodies below the still tide levels. In cases wherein differences exist in still tide levels inside and outside breakwaters, buoyancy can be considered for the portions of breakwater bodies below the lines connecting the still tide levels inside and outside breakwaters.

⑧ The influences of wind pressure, earth pressure, impulsive force of ships and floating objects, and currents shall be considered as needed.

⑨ In cases wherein erosion, sedimentation, and changes in the gradients of sea bottoms can be expected after the construction of breakwaters, the influences of those phenomena shall also be considered.

⑩ For dynamic water pressure during earthquakes, refer to **Part II, Chapter 4, 3.2 Dynamic Water Pressure**.

3.1.4 Performance Verification of the Overall Stability of Breakwater Bodies

(1) Performance Verification Items for the Overall Stability of Breakwater Bodies

For the performance verification items when conducting the performance verification of the overall stability of breakwater bodies under respective design situations on the basis of the static equation of equilibrium, refer to **Part III, Chapter 4, 3.1 [interpretation], Attached Table 10-3**. The performance verification of the variable action situation in which the dominant action is Level 1 earthquake ground motions can be performed on the basis of **Fig. 3.1.4 Figure for Determining the Necessity of the Performance Verification of Earthquake Resistance**. The performance verification of an accidental action situation can be omitted in cases wherein design object breakwaters are not categorized as facilities prepared for accidental incidents.

(2) Performance Verification of the Overall Stability of Breakwater Bodies under Permanent Action Situation

The performance verification of the overall stability of breakwater bodies under a permanent action situation in which the dominant action is self-weight shall be performed for the circular slip failures of foundation ground in general.

① The verification of the circular slip failures of foundation ground under a permanent action situation with respect to the self-weights of breakwater bodies can be performed using **equation (3.1.1)**. In this equation, subscripts *k* and *d* indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.1**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

② The partial factors shown in **Table 3.1.1** were set with reference to the safety levels in past standards.[4] Furthermore, the coefficients of the variation *CV* of cohesive soil in the table can be determined using the coefficients of variation *CV* corresponding to the correction factor b_1 from the process of calculating the characteristic values of adhesion in **Part II, Chapter 3, 2.1 Estimation of the Physical Property of Ground**. In such a case, among the soil layers (excluding thin ones) where circles can pass through, the soil layer that has the largest coefficient of variation *CV* can be the representative soil layer.

$$
\begin{aligned}
& m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\
& R_k = \sum \left[\left\{ c'_k\, s + (w'_k + q_k)\cos^2\theta \tan\phi'_k \right\} \sec\theta \right] \\
& S_k = \sum \left[(w_k + q_k)\sin\theta \right]
\end{aligned}
\qquad \textbf{(3.1.1)}
$$

where

c' : undrained shear strength for cohesive soil ground or apparent adhesion under a drained condition for sandy soil ground (kN/m^2);

s : width of a segment (m);

w' : effective weight of a segment (kN/m) (atmospheric weight when above water surface or underwater weight when below water surface);

q : surcharge acting on a segment (kN/m);

ϕ' : apparent shear resistance angle on the basis of effective stress (°);

θ : angle between the bottom face of a segment and a horizontal plane (°);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.1.1 Partial Factors Used for the Performance Verification of the Circular Slip Failure of Foundation Ground

Verification object	Coefficient of variation of cohesive soil in the representative soil layer CV	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Circular slip failure of foundation ground (Permanent state)	Case of no cohesive soil in the layer where a circle passes through	0.83	1.01	— (1.00)
	Less than 0.10	0.86	1.05	— (1.00)
	Not less than 0.10 and less than 0.15	0.85	1.04	— (1.00)
	Not less than 0.15 and less than 0.25	0.80	1.02	— (1.00)

Verification object	Coefficient of variation of cohesive soil in the representative soil layer CV	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
	Not less than 0.25	— (1.00)	— (1.00)	1.30

③ In the performance verification of the sliding of ground, the most dangerous tide levels should be used for the stability of breakwaters. For the setting of tide levels, refer to **Part II, Chapter 2, 3 Tide Levels**.

④ When improving foundation ground, the verification of circular slip failure can be performed by referring to **Part II, Chapter 2, 5 Ground Improvement Method**.

(3) Performance Verification of the Overall Stability of Breakwater Bodies under a Variable Action Situation (Variable Waves)

① **General**

The performance verification of the overall stability of breakwater bodies under a variable action situation in which the dominant action is variable waves shall be performed for the sliding and overturning of breakwater bodies and the bearing capacity of foundation ground.

② **Examination of the sliding of breakwater bodies**

(a) The verification of the sliding of breakwater bodies with respect to variable waves can be performed using **equation (3.1.2)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.2**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$R_k = \{f_k (W_k - P_{B_k} - P_{U_K})\} \tag{3.1.2}$$
$$S_k = P_{Hk}$$

where

f : friction coefficient between the bottom face of a wall body and a foundation;

W : weight of a breakwater body (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift (kN/m);

P_H : horizontal wave force (kN/m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.1.2 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of a breakwater body (Variable state of waves)	0.83	1.08	– (1.00)

(b) The partial factors shown in **Table 3.1.2** were set with reference to the safety levels in past standards.[5)] Furthermore, the partial factors above are set under the conditions that the topographies of seabed where breakwaters are installed have gradients of less than 1/30. In cases wherein the topographies of seabed have gradients larger than 1/30, partial factors shall be appropriately set with reference to the descriptions in **Reference 5)**.

(c) For the verification of the sliding failures of composite breakwaters with the harbor side of upright sections reinforced, refer to **Part III, Chapter 4, 3.1.6 Performance Verification and Cautions When the Harbor Side of Upright Sections is Reinforced**.

(d) The tide levels to be used for the verification of the sliding and overturning of wall bodies, as well as bearing capacity, are generally either mean monthly-lowest water levels (L.W.L) or mean monthly-highest water levels (H.W.L).

(e) In cases wherein caissons with footings have rectangular cross sections at both seaward and landward sides, buoyancy P_B can be calculated using the following equation. In this equation, subscript k indicates the characteristic value. For footings with other shapes and hunched sections, buoyancy shall be appropriately set.

$$P_{B_k} = \rho_w g \{(wl_k + h)B_c + 2h_f B_f\} \tag{3.1.3}$$

where

$\rho_w g$: unit weight of sea water (kN/m^3);

wl : tide level (m);

h : installation depth (m);

B_C : width of breakwater body (m);

h_f : height of footing (m);

B_f : width of footing (m).

(f) For the calculation of wave force, refer to **Part II, Chapter 2, 6.2 Wave Force Acting on Vertical Walls**.

(g) For the unit weights and friction coefficients to be used in the performance verification, refer to **Part II, Chapter 10 Self-weight and Surcharges** and **Part II, Chapter 11, 9 Friction Coefficients**. There are cases wherein friction enhancement mats are laid under the bottom faces of upright sections to increase the friction coefficients between the upright sections and foundation mounds. For the friction enhancement mats, refer to **Part II, Chapter 11, 9 Friction Coefficients**.

③ **Examination of the overturning of breakwater bodies**

(a) The verification of the overturning of breakwater bodies due to variable waves can be performed using **equation (3.1.4)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.3**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

$$\begin{aligned} & m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\ & R_k = (a_1 W_k - a_2 P_{B_k} - a_3 P_{U_K}) \\ & S_k = a_4 P_{H_k} \end{aligned} \tag{3.1.4}$$

where

W : weight of a breakwater body (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift force (kN/m);

P_H : horizontal wave force (kN/m);

a_1–a_4 : arm lengths of actions (m) (refer to **Fig. 3.1.5**);

R_k : resistance term (kN·m/m);

S_k : load term (kN·m/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

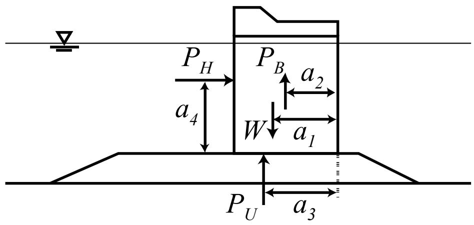


Fig. 3.1.5 Arm Lengths When Calculating Moment

Table 3.1.3 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	0.95	1.14	— (1.00)

(b) The partial factors shown in **Table 3.1.3** were set with reference to the safety levels in past standards.[5] Furthermore, the partial factors above are set under the conditions that the topographies of seabed where breakwaters are installed have gradients of less than 1/30. In cases wherein topographies of seabed have gradients larger than 1/30, partial factors shall be appropriately set with reference to the descriptions in **Reference 5)**.

(c) In cases wherein caissons with footings have rectangular cross sections at both seaward and landward sides, buoyancy can be calculated using **equation (3.1.3)**. For footings with other shapes and hunched sections, buoyancy shall be appropriately set.

④ **Examination of the bearing capacity of foundation ground**

(a) The verification of the stability of the bearing capacity of foundation ground at the bottom faces of the upright section against variable waves can be conducted in accordance with the simplified Bishop method (refer to **Part III, Chapter 2, 4 Slope Stability**), which is one of the circular slip calculation methods based on the slicing method. The simplified Bishop method was adopted in the verification because it has been proven by experiments in centrifugal fields to be a model that can best explain the stability of bearing capacity compared with the modified Fellenius' method and the friction circle method.[6]

(b) The performance verification of the bearing capacity of foundation ground can be performed using **equation (3.1.5)**, which was obtained using the simplified Bishop method. In the equation, the partial factors in the equation can be selected from the values in **Table 3.1.4**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification of convenience. Furthermore, in the equation, subscripts k and d indicate the characteristic value and design value, respectively. When using **equation (3.1.5)**, first an auxiliary parameter F_f needs to be determined via repeated calculation so that F_f satisfies $R_k = F_f \times S_k$ (with attention to the fact that R_k is a function of F_f), and the stability verification of bearing capacity can be performed using R_k and S_k obtained as a result of the repeated calculation.

$$
\begin{aligned}
& m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\
& F_f = \frac{R_k(F_f)}{S_k} \\
& R_k = \sum \left[\frac{\{c'_k s + (w'_k + q_k)\tan\phi'_k\}\sec\theta}{1 + \tan\theta \tan\phi'_k / F_f} \right] \\
& S_k = \sum \{(w'_k + q_k)\sin\theta\} + \frac{dP_{H_k}}{r}
\end{aligned}
\qquad \textbf{(3.1.5)}
$$

where

P_H : horizontal wave force (kN/m);

c' : undrained shear strength for cohesive soil ground or apparent adhesion under a drained condition for sandy soil ground (kN/m^2);

s : width of a segment (m);

w' : effective weight of a segment (kN/m) (atmospheric weight when above the water surface or underwater weight when below the water surface);

q : surcharge acting on a segment (kN/m);

ϕ' : apparent shear resistance angle on the basis of effective stress (°);

θ : angle between the bottom face of a segment and a horizontal plane (°);

F_f : auxiliary parameter representing a ratio of a resistance term to a load term;

d : arm length of horizontal wave force P_H (a length of a vertical line from the center of a circle to an acting force vector);

r : radius of a slip failure circle (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : an adjustment factor.

Table 3.1.4 Partial Factors Used for the Performance Verification of the Bearing Capacity of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Bearing capacity of foundation ground (Variable state of waves)	— (1.00)	— (1.00)	1.00

(4) Performance Verification of the Overall Stability of Breakwater Bodies under a Variable Action Situation (Level 1 Earthquake Ground Motions)

① **Examination of the sliding failures of breakwater bodies**

(a) Although the verification of the stability of breakwater bodies against Level 1 earthquake ground motions is often omitted, in cases wherein breakwaters have deep installation depths and small design wave heights, there may be the cases wherein Level 1 earthquake ground motions can be a dominant action. In such cases, the performance verification shall be performed for the earthquake resistance.

(b) For the determination of the necessity of the performance verification of earthquake resistance and the method for calculating the seismic coefficients when performing the performance verification of earthquake resistance, refer to **Fig. 3.1.4** and **Reference (Part III), Chapter 1, 1 Details of Seismic Coefficients for Verification**.

(c) The verification of the sliding failures of breakwater bodies due to Level 1 earthquake ground motions on the basis of the equation of equilibrium can be performed using **Equation (3.1.6)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.5**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification of convenience. The seismic coefficients for verification in **equation (3.1.6)** can be calculated by the method described in **Reference (Part III), Chapter 1, 1 Details of Seismic Coefficients for Verification**.

$$
\begin{aligned}
& m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\
& R_k = \mu W'_k \\
& S_k = k_{h_k} W_k + 2P_{dw_k}
\end{aligned}
\tag{3.1.6}
$$

where

k_h : seismic coefficient for verification;

W : weight of a breakwater body (kN/m);

P_{d_w} : resultant force of dynamic water pressure (kN/m), which can be calculated by **equation (3.1.7)**.

$$
P_{d_w} = \frac{7}{12} k_h \rho_w g H^2 \tag{3.1.7}
$$

where

$\rho_w g$: unit weight of sea water (kN/m^3);

H : installation depth of a breakwater body (m);

W' : effective weight of a breakwater body in water ($= W - P_B$) (kN/m);

P_B : buoyancy (kN/m);

μ : friction coefficient between a breakwater body and a rubble mound;

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.1.5 Partial Factors Used for the Performance Verification of the Sliding Failure of Breakwater Bodies Due to Level 1 Earthquake Ground Motions

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding failure of breakwater body (Variable state of Level 1 earthquake ground motions)	— (1.00)	— (1.00)	1.20

② **Examination of overturning of breakwater bodies**

(a) The verification of the overturning of breakwater bodies due to Level 1 earthquake ground motions on the basis of the equation of equilibrium can be performed using **equation (3.1.8)**. In the equation, subscripts k and d indicate the characteristic value and design value, respectively, and the buoyancy acting on breakwater bodies can be calculated by **equation (3.1.3)**. The partial factors in the equation can be selected from the values in **Table 3.1.6**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$R_k = a_3 W'_k \qquad \textbf{(3.1.8)}$$
$$S_k = a_1 k_{hk} W_k + 2a_2 P_{dwk} W_k$$

where

k_h : seismic coefficient for verification;

W : weight of a breakwater body (kN/m);

P_{d_w} : resultant force of dynamic water pressure (kN/m), which can be calculated by **equation (3.1.7)**.

Table 3.1.6 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies Due to Level 1 Earthquake Ground Motions

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of Level 1 earthquake ground motions)	— (1.00)	— (1.00)	1.10

③ **Examination of the bearing capacity of foundation ground**

(a) The verification of the bearing capacity of foundation ground against Level 1 earthquake ground motions can be performed with due consideration to the actions of earthquake ground motions and with reference to **Part III, Chapter 2, 3.2 Shallow Foundations**. However, for breakwaters that are expected to have major problems with the bearing capacity of foundation ground and the stability against settlement, it is preferable to perform detailed examinations, including dynamic analysis.

(5) Performance Verification of Breakwater Bodies under Accidental Situation

① For the performance verification of breakwater bodies under an accidental situation in which the dominant action is Level 2 earthquake ground motions, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters**

Categorized as the Facilities Prepared for Accidental Incidents and **Part III, Chapter 4, 7.4.1 Setting and Impact Assessment of Earthquake Ground Motions Preceding Tsunamis**.

② For the performance verification of breakwater bodies under an accidental situation in which the dominant action is design tsunamis, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Categorized as the Facilities Prepared for Accidental Incidents** and **Part III, Chapter 4, 7 Tsunami Protection Breakwaters**.

③ For the performance verification of breakwater bodies under an accidental situation in which the dominant action is accidental waves, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Categorized as the Facilities Prepared for Accidental Incidents** and **Part III, Chapter 4, 6 Storm Surge Prevention Breakwaters**.

3.1.5 Performance Verification and Points of Cautions for Other Items about the Overall Stability of Breakwater Bodies

(1) Performance Verification of the Stability of Sloped Sections

① For the performance verification of the stability of rubble sections, refer to **Part III, Chapter 2, 3.2.5 Bearing Capacity against Eccentric Inclined Actions**.

② The armor units of rubble sections shall have mass to achieve sufficient stability against wave force and thicknesses to prevent the inner materials from being washed out.

③ For the calculation of the required mass of armor units, refer to **Part II, Chapter 2, 6.6.2 Required Mass of Armor Stones and Blocks of Composite Breakwater Mounds against Waves.**

④ For the performance verification of the sloped sections covered by sand mastic, refer to past cases and existing study results.[7)]

(2) Points of Caution When Performing the Performance Verification of Head and Corner Sections

① Unlike the trunk sections, the head sections of composite breakwaters have factors that have not been fully elucidated, such as the washing out of and actions on foundations. Therefore, it is preferable that the mass of armor stones and the blocks of head sections are larger than that of the trunk sections. For the calculation of the mass of armor materials, refer to **Part II, Chapter 2, 6.6.2 Required Mass of Armor Stones and Blocks of Composite Breakwater Mounds against Waves**.

② In cases of soft ground, the performance verification shall be performed for the sliding failures of breakwaters in their face line directions. In such cases, the performance verification can take into consideration the friction resistance at foundation sides.

③ For the sliding failures of breakwaters in their face line directions, refer to **Part III, Chapter 2, 4 Slope Stability**.

④ The performance verification of corner sections shall take into consideration the increases in wave heights.

⑤ When there are corner sections on the face lines of composite breakwaters, the corner sections not only allow waves to converge on them but also cause the increase in wave heights around them owing to the superposition of reflection waves from respective sections along the face lines. There have been cases of damage to breakwaters due to such wave actions at corner sections. Therefore, the determination of face lines and the stability calculations of composite breakwaters shall be made with reference to **Part II, Chapter 2, 4.4 Wave Deformation** and **Part II, Chapter 2, 6.2.8 Calculation of Wave Force in Consideration of the Effects of the Shapes of Face Lines**.

⑥ Head sections provided with beacons shall ensure their structural safety with the beacons, including the ancillary facilities necessary to maintain the functions of the beacons. For the wind pressure acting on beacons, refer to **Part II, Chapter 2, 2 Wind**.

⑦ It shall be noted that there have been cases of damage to the base sections of breakwaters extended from beaches because the structures of the base sections were simplified.

(3) Examination of Settlement

The performance verification of the settlement due to consolidation or other reasons shall be performed with due consideration to the characteristics of the ground and structures. For the settlement, refer to **Part III, Chapter 2, 3.5 Settlement of Foundations**.

(4) Other Performance Verification

① The stability verification of breakwater bodies against variable waves can be performed with reference to design methods[8) to 13)], which take into consideration the expected sliding distances.

② The performance verification of the deformation and stresses of breakwater bodies or ground with respect to earthquake ground motions can be performed using the seismic response analysis based on the finite element method. For the points of caution when using the seismic response analysis, refer to the contents in **Reference (Part III), Chapter 1, 2 Basic Items for Seismic Response Analysis**.

3.1.6 Performance Verification and Cautions When the Harbor Side of Upright Sections is Reinforced

(1) General

① One of the typical methods for reinforcing the harbor side of upright sections is embankment widening work, which installs rubbles and blocks at the back of upright sections. Properly arranged rubbles and blocks enable upright sections to increase resistance against the sliding and bearing capacity of foundations. It shall be noted that embankment widening work needs to be implemented to avoid the disturbance of navigation, sheltering, and mooring of ships inside harbors. The items described in this section are based on the research outcomes of Takahashi et al.[14)] and Sato et al.[15)] In addition to the embankment widening work using rubble and blocks, there are other alternative methods. For the details of these alternative methods, refer to **Reference 16)**.

② In the verification of the stability of breakwater bodies against sliding failure due to wave force on upright sections without considering the embankment widening work at the back of breakwater bodies, the action–resistance ratios calculated with a partial factor of 1.0 shall be less than 1.0 in essence because large action–resistance ratios have risks of causing upright sections to slide until the embankment widening work deforms to a level that produces sufficient resistance to stabilize the upright sections or to slide or be overturned seaward owing to backwash.

③ Embankment widening work shall have enough protection from damage due to overtopping and longshore waves, as well as overflowing tsunamis. Furthermore, full-scale embankment widening work causes an increase in the uplift force to be applied to shielding work. Therefore, it is necessary to examine the stability of breakwater bodies in consideration of the uplift force or to provide adequate openings on shielding work so that water pressure can be released.

④ In cases wherein there are water level differences between the seaward and landward sides of breakwater bodies due to tsunamis, the embankment widening work made of foundation mounds and rubble receives seepage force and that made of blocks and shielding work receive uplift force. The seepage force that acts on the embankment widening work made of foundation mounds and rubble reduces the maximum resistance force of the embankment widening work in the performance verification for slide failures and reduces the stability of the entire ground in the performance verification for bearing capacity. Likewise, the uplift force that acts on the embankment widening work made of blocks reduces the maximum resistance force of the embankment widening work in the performance verification for slide failures and reduces the stability of the entire ground in the performance verification for bearing capacity. Furthermore, the stability of shielding work shall be examined with due consideration to the effects of uplift force by preferably referring to **Part III, Chapter 4, 7 Tsunami Protection Breakwaters**.

⑤ When rubble is used for embankment widening work, height a and width b of embankment widening work are basically not less than 1/3 of the heights of upright sections (including superstructures) (refer to **Fig. 3.1.6**). In cases wherein the embankment widening work is smaller than the above, it is necessary to conduct performance verification by using centrifugal model tests or finite element analyses, which can appropriately assess the behavior of ground in addition to the verification described below. In actual construction, it is also necessary to avoid the use of rubble stones that are rounded and have small diameters because the foundation ground made of rounded rubble stones has small shear strength and rubble stones with small diameters are likely to cause piping.

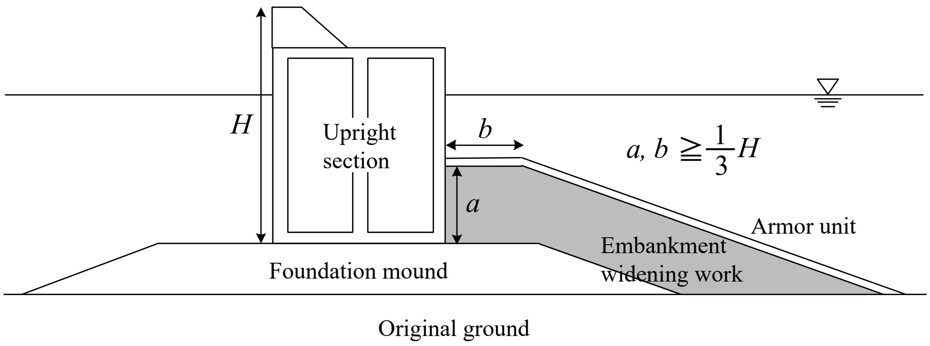


Fig. 3.1.6 Breakwater Reinforced with the Harbor Side Reinforced with Rubble Stones

⑥ When reinforcing breakwaters with embankment widening work made of blocks such as concrete blocks, it is necessary to perform embankment widening work without leaving gaps between the upright sections and the blocks. These blocks shall have sufficiently long durability. Furthermore, a study[14)] reported that blocks installed at the bottom levels that are different from those of upright sections can increase the resistance against sliding. Therefore, there is a possibility of providing breakwater bodies with larger resistance against sliding by devising the methods for installing blocks.

⑦ Basically, the performance verification of bottom slabs shall not consider the reaction force from embankment widening work because the characteristics of the reaction force from embankment widening work have not been clarified. However, such reaction force may be incorporated into performance verification in cases wherein the characteristics of the reaction force can be appropriately examined through model tests.

(2) Verification of Sliding Failures

① The verification of sliding failures can be performed using **equation (3.1.9)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.7**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$R_k = \left\{ f_k (W_k - P_{Bk} - P_{Uk} - P_{Vk}) + P_{H2\max k} \right\} \tag{3.1.9}$$

$$S_k = P_{Hk}$$

where

f : friction coefficient between the bottom face of an upright section and a foundation mound;

W : atmospheric weight of an upright section (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift force (kN/m);

P_H : horizontal wave force (kN/m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

$P_{H2\max}$: maximum resistance from reinforcing rubble or block (kN/m);

P_V : friction force between the bottom face of an upright section and rubble stones (kN/m);

m : adjustment factor.

Table 3.1.7 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies When the Harbor Side of Upright Sections Is Reinforced

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

② The maximum resistance $P_{H2\,\max}$ when reinforcing breakwaters with embankment widening work made of rubble stones can be obtained by **equation (3.1.10)**. This equation is the simplified Bishop method (expressed in the form of effective stress) with a partial factor of 1.0 to obtain $P_{H2\,\max}$ in a manner that assumes a shallow circular slip surface started from a rear toe of an upright section (**Fig. 3.1.7**). It is necessary to change the positions of the circular slip surface to identify the position that achieves the least $P_{H2\,\max}$. Here parameters such as shear strength (c' and ϕ') shall be set in accordance with the values of foundation mounds.

$$\sum\left[\frac{\left\{c'_k s+\left(w'_k+q_k\right)\tan\phi'_k\right\}\sec\theta}{1+\tan\theta\tan\phi'_k}\right]=\sum\left\{\left(w'_k+q_k\right)\sin\theta\right\}+\frac{a_2 P_{H2\max k}}{r} \qquad \textbf{(3.1.10)}$$

where

c' : undrained shear strength for cohesive soil ground or apparent adhesion of the ground made of stone materials (kN/m^2);

ϕ' : shear resistance angle under a drained condition for sandy soil or the ground made of stone materials (°);

s : width of a segment (m);

w' : effective weight of a segment (kN/m) (atmospheric weight when above water surface or underwater weight when below water surface);

q : vertical load acting on a segment (including q_V and P_V) (kN/m) (where $P_V = \tan 15°\cdot P_{H2\,\max}$);

θ : angle between the bottom face of a segment and a horizontal plane (°);

$P_{H2\,\max}$: maximum resistance from reinforcing rubble stones (kN/m) (with the working height set at 1/3 of the height a of embankment widening work);

a_2 : arm length of $P_{H2\,\max}$ (a length of a vertical line from the center of a circle to an acting force vector) (m);

r : radius of a slip circle (m).

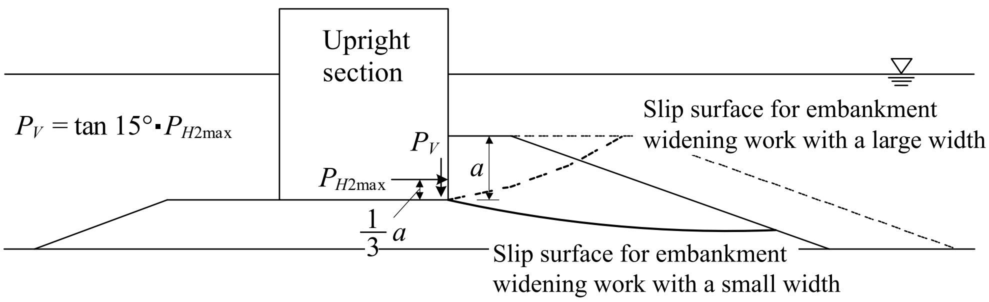


Fig. 3.1.7 Concept of a Shallow Slip Surface When the Harbor Side of Upright Sections Is Reinforced with Rubble Stones

③ When reinforcing breakwaters with embankment widening work made of blocks, the maximum resistance $P_{H2\,max}$ can be obtained by **equation (3.1.11),** in which the friction force P_v is omitted. It is necessary to use a friction coefficient f that is obtained via friction tests.

$$P_{H2\max k} = f_k\, W_{bk} \tag{3.1.11}$$

where

f : friction coefficient between a block and a foundation mound;

W_b : effective weight of a block (kN/m) (atmospheric weight when above water surface or underwater weight when below water surface).

(3) Performance Verification of Overturning

Owing to a low working point, the resistance force from embankment widening work with a height that is 1/3 the height of the upright section does not contribute significantly to the stability of the upright section against overturning. Furthermore, the stone materials used for foundation mounds and embankment widening work exert large shear resistance force after they are subjected to a certain level of shear strain. Therefore, there is a risk that upright sections may have already lost their stability against overturning by the time the resistance force from embankment widening work is exerted. With all these factors, the performance verification of overturning shall be performed without considering the effects of embankment widening work by using **equation (3.1.8**), with partial factors selected from **Table 3.1.8**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification of convenience. However, the resistance force from embankment widening work can be considered only in cases wherein the effectiveness of the resistance force is appropriately verified by model tests.

Table 3.1.8 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies When the Harbor Side of Upright Sections is Reinforced

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

(4) Performance Verification of Bearing Capacity

① The performance verification of bearing capacity when the harbor side of upright sections is reinforced with rubble stones can be performed using **equation (3.1.12)**. In the equation, subscripts k and d indicate the characteristic value and design value, respectively. **Equation (3.1.12)** is the simplified Bishop method, which is expressed in the form of effective stress and assumes deep circular slip surfaces starting from the lower points of upright sections (**Figs. 3.1.8** and **3.1.9**).

When using **equation (3.1.12)**, an auxiliary parameter F_f first needs to be determined via repeated calculations so that F_f satisfies $R_k = F_f \times S_k$ (with attention to the fact that R_k is a function of F_f). Thereafter, the performance verification of bearing capacity can be performed using the R_k and S_k obtained as a result of the repeated calculation. The partial factors and adjustment factors in the equation can be selected from the values in **Table 3.1.9**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience. In the case of embankment widening work made of rubble stones, parameters such shear strength (c' and ϕ') of embankment widening work shall be set in accordance with the values of foundation mounds.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$F_f = \frac{R_k(F_f)}{S_k}$$

$$R_k = \sum \left[\frac{\{c'_k s + (w'_k + q_k) \tan \phi'_k\} \sec \theta}{1 + \tan \theta \tan \phi'_k / F_f} \right] \tag{3.1.12}$$

$$S_k = \sum \{(w_k' + q_k) \sin \theta\} + \frac{a_1 P_{H1_k} + a_2 P_{H2_k}}{r}$$

where

c' : undrained shear strength for cohesive soil ground or apparent adhesion of the ground made of stone materials (kN/m^2);

ϕ' : shear resistance angle under a drained condition for sandy soil or the ground made of stone materials (°);

s : width of a segment (m);

w' : effective weight of a segment (kN/m) (atmospheric weight when above water surface or underwater weight when below water surface);

q : vertical load acting on a segment (including q_v, P_V and q_b) (kN/m);

θ : angle between the bottom face of a segment and a horizontal plane (°);

P_{H1} : friction resistance force on the bottom face of an upright section (kN/m);

P_{H2} : resistance force from reinforcing rubble stones (kN/m) (with the working height set at 1/3 of the height a of embankment widening work. This resistance force cannot be considered in the case of embankment widening work made of blocks);

a_1 : arm length of P_{H1} (a length of a vertical line from the center of a circle to an acting force vector) (m);

a_2 : arm length of P_{H2} (a length of a vertical line from the center of a circle to an acting force vector) (m);

F_f : auxiliary parameter representing a ratio of a resistance term to a load term;

r : radius of a slip failure circle (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.1.9 Partial Factors Used for the Performance Verification of Bearing Capacity When the Harbor Side of Upright Sections is Reinforced

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Bearing capacity of foundation ground (Variable state of waves)	— (1.00)	— (1.00)	1.00

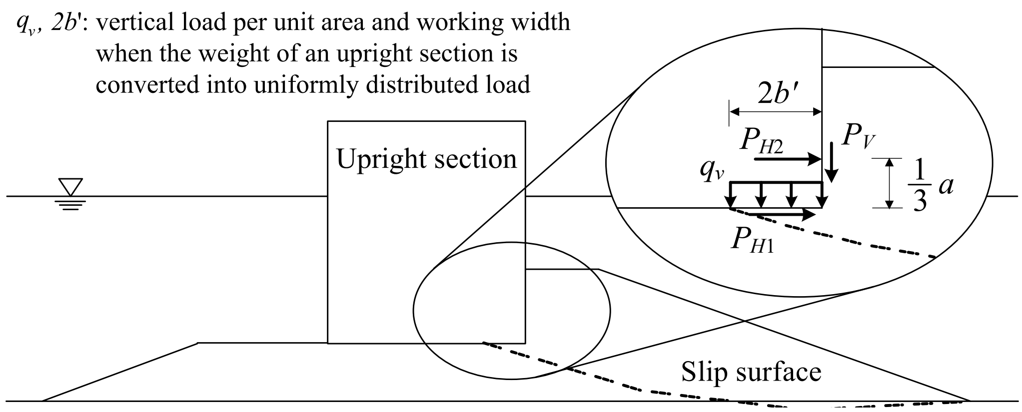


Fig. 3.1.8 Concept of a Deep Slip Surface When the Harbor Side of Upright Sections is Reinforced with Rubble Stones

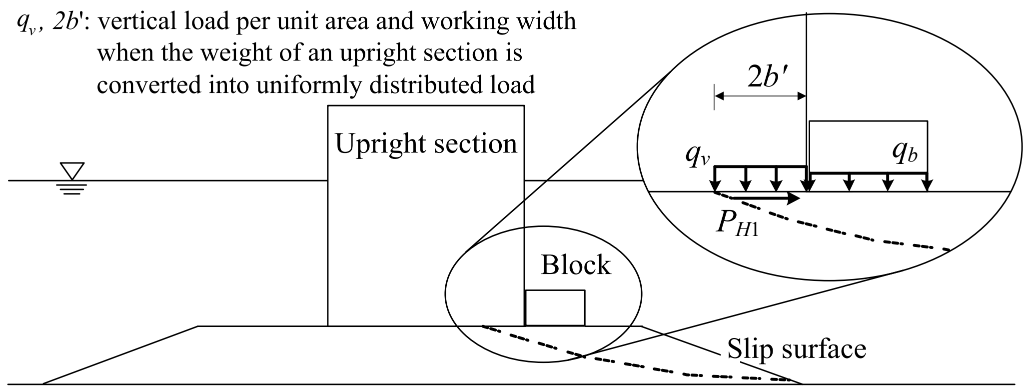


Fig. 3.1.9 Concept of a Deep Slip Surface When the Harbor Side of Upright Sections is Reinforced with Blocks

② When reinforcing breakwaters with embankment widening work made of rubble stones, upright sections are subjected to horizontal wave force P_H, buoyancy P_B, uplift force P_U, reaction force q_v and friction force P_{H1} from foundation mounds, and resistance force P_{H2} and friction force P_V from embankment widening work. The loads acting on a foundation mound and embankment widening work can be calculated using the equation of equilibrium at an upright section and hypothetical conditional equations (i.e., **Equations (3.1.13)** and **(3.1.14])**. Thereafter, the performance verification of bearing capacity can be performed by substituting the calculated loads into **equation (3.1.12)**.

$$P_{Vk} = \tan 15° \cdot P_{H2k} \tag{3.1.13}$$

$$P_{H1_k} = r^* \cdot P_{Hk}\ ,\ P_{H2_k} = \left(1 - r^*\right) P_{Hk} \tag{3.1.14}$$

where

r^* : load sharing ratio (a ratio of the portion of horizontal wave force to be resisted by the friction force on the bottom face of an upright section to total horizontal wave force)

③ The load sharing ratios r^* are subjected to cross-sectional shapes, ground materials, and displacement states of upright sections; therefore, it is difficult to set it to a fixed value. By contrast, it has been known that the load sharing ratios r^* have small effects on the stability assessment results.[15)] Therefore, the load sharing ratios r^* can be set at 0.5 in the performance verification. However, when P_{H1} and P_{H2} calculated with r^* set at 0.5 exceed the maximum values of $f\,(W\text{-}P_B\text{-}P_U\text{-}P_V)$ and $P_{H2\,\max}$, respectively, the load sharing ratios r^* shall be adjusted to make P_{H1} and P_{H2} less than their maximum values.

④ When reinforcing breakwaters with embankment widening work made of blocks, upright sections are subjected to horizontal wave force P_H, buoyancy P_B, uplift force P_U, and reaction force q_v and friction force P_{H1} from foundation mounds. The load acting on a foundation mound and a working point can be calculated using the equation of equilibrium at an upright section. Furthermore, the effective weight of a block is considered to act on the foundation mound. Thereafter, by using all these loading conditions, the performance verification of bearing capacity can be performed by **equation (3.1.12)**.

3.1.7 Foot Protection Blocks

(1) Breakwaters are preferably provided with foot protection blocks to protect rubble sections from being washed out, except for breakwaters that have very deep rubble sections or are installed in water areas where wave heights are low and rubble mass is sufficient for achieving theoretical stability. It is also preferable that foot protection blocks are brought into tight contact with upright sections.

(2) Armor units for mounds are installed at the front side of foot protection blocks. In terms of ensuring the stability of foot protection blocks, it is preferable to reduce the differences in the levels between armor units and blocks to the greatest extent possible.

(3) Those foot protection blocks provided with holes can reduce the uplift forces acting on them, thereby significantly improving their stability against wave actions.

(4) According to the research by Tanimoto et al.[17)], the aperture ratios of the holes on foot protection blocks are preferably set at approximately 10% because excessively large holes decrease the effect of foot protection blocks to prevent scouring and washing out.

(5) When installing foot protection blocks, it is preferable to arrange them in at least two rows at the seaward side and at least one row at the landward side of upright sections.

(6) The required thicknesses of foot protection blocks can be obtained by **equation (3.1.15)**.[18)]

$$t / H_{1/3} = d_f \left(h'/ h\right)^{-0.787} \qquad \textbf{(3.1.15)}$$

where

t : required thickness of a foot protection block (m);

d_f : 0.18 for a trunk section and 0.21 for a head section of a breakwater (m);

h : design water depth (m);

h' : water depth at the top of rubble mound (excluding a block) (m) with the scope of application in the range of h'/h = 0.4 to 1.0.

(7) For the dimensions of foot protection blocks, the required thicknesses can be calculated using **equation (3.1.15)**, and other dimensions can be selected from the values in **Table 3.1.10**. **Fig. 3.1.10** shows examples of the shapes and dimensions of foot protection blocks. In addition to these examples, the shapes of foot protection blocks can be determined with reference to **Reference 19)** and via hydraulic model tests.

Table 3.1.10 Required Thickness and Dimensions of Foot Protection Block (Example)

Required thickness of foot protection blocks t (m)	Dimensions l (m) × b (m) × t (m)	Mass (t/unit)	
		Block with openings	Block without openings
0.8 or less	2.5 × 1.5 × 0.8	6.23	6.90
1.0 or less	3.0 × 2.5 × 1.0	15.64	17.25
1.2 or less	4.0 × 2.5 × 1.2	24.84	27.60
1.4 or less	5.0 × 2.5 × 1.4	37.03	40.25
1.6 or less	5.0 × 2.5 × 1.6	42.32	46.00
1.8 or less	5.0 × 2.5 × 1.8	47.61	51.75
2.0 or less	5.0 × 2.5 × 2.0	52.90	57.50
2.2 or less	5.0 × 2.5 × 2.2	58.19	63.25

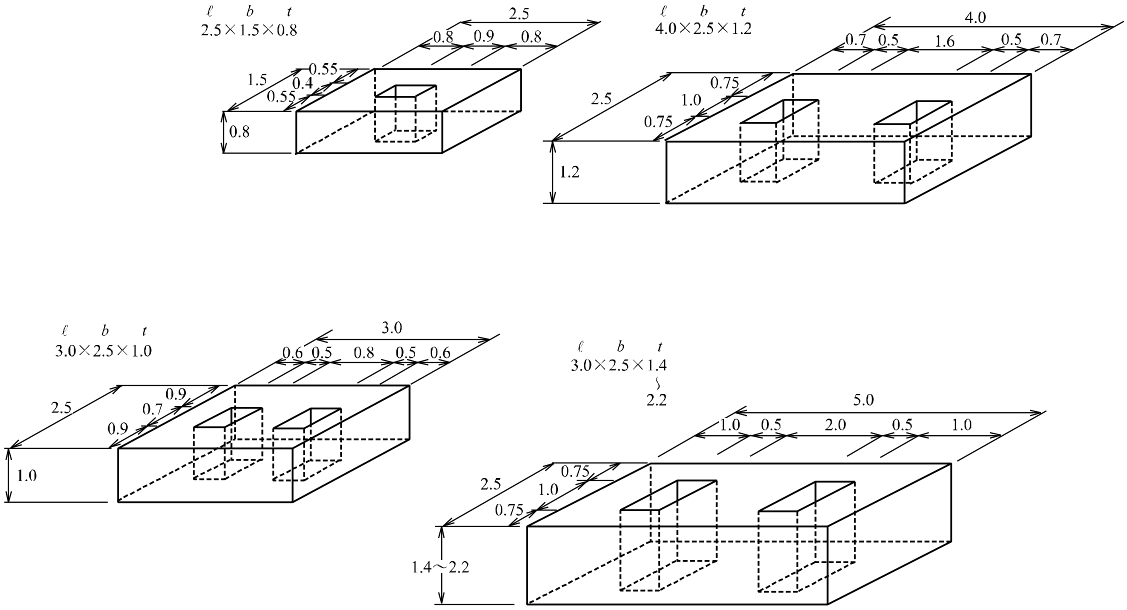


Fig. 3.1.10 Shapes of Foot Protection Blocks (unit in m)

(8) The performance verification of foot protection blocks installed at a harbor side shall be performed by taking into consideration the effects of the waves inside harbors, waves during construction, and overtopping waves as needed.

(9) Given that the occurrences of damage to foot protection blocks at a harbor side are rare, the mass of the foot protection blocks at a harbor side can be smaller than the conventional 1/2 mass at a seaward side provided that the mass shall not be smaller than the required mass determined with consideration to the waves inside harbors and during construction. Particular attention is required in the use of foot protection blocks at the offshore ends of breakwaters as temporary head sections during construction.

3.1.8 Performance Verification of Structural Members

For the performance verification of structural members for caisson type, cellular block type, and hybrid caisson-type breakwaters, refer to **Part III, Chapter 2, 2 Structural Members**.

3.1.9 Structural Details

(1) Items Common to Composite Breakwaters

① The concrete of superstructures shall be constructed with consideration to the integrity of the superstructures with breakwater bodies and shall be provided with joints at proper intervals in the face line directions. The joints are generally installed between caissons for caisson-type breakwaters and at intervals of 10 to 20 m for other types of breakwaters.

② It is preferable to study the necessity of taking measures to curb cracks due to the hydration heat of cement with consideration to the construction conditions of superstructure concrete as needed.

③ Rubble sections preferably undergo heavy weather seasons for the purpose of enhancing their compaction, thereby curbing settlement after the construction of upright sections.

④ To correctly install upright sections, the top surfaces of rubble sections shall be leveled with blinding to ensure flatness with stones that are sufficiently interlocked with one another. Severely uneven rubble surfaces may cause adverse effects on caissons, such as torsional force on caissons and concentrated loads on bottom slabs. The areas of rubble surfaces to be leveled shall consider allowances at both sides of upright sections and include those for foot protection blocks and armor stones.

⑤ Breakwaters with possible risks of scouring and washing out shall be provided with scour and washing-out prevention measures with reference to the descriptions in **Part II, Chapter 2, 7.5 Scouring and Washing Out**. The scour and washing-out prevention measures include small-stepped rubble mounds at slope toes or the protection of slope toes with submerged floor mats, asphalt mats,[20), 21)] or synthetic resin mats. The measures to prevent rubble mounds from settlement due to washing out include the installation of submerged floor mats or the laying of canvas sheets.[22)]

(2) Items for Caisson-type Composite Breakwaters

① The thicknesses of concrete lids of caisson-type composite breakwaters shall be carefully determined by taking into consideration wave and construction conditions.

② The types of materials used as the infill of caissons are concrete, concrete blocks, stones, gravel, sand, slag, etc. It is preferable to determine the types of materials in consideration of construction costs and construction/natural conditions. Sand is the typical material used as the infill of caissons. When using sand and gravel as infill, it is necessary to protect the infill in a manner that completely covers the infill with concrete lids or blocks.

③ Given that some types of slag expand when absorbing water, the quality of the types of slag to be used as infill needs to be carefully studied, including the pretreatment of slag before filling it into caissons.

④ The thicknesses of concrete lids shall be not less than 30 cm for caissons subjected to normal sea conditions or not less than 50 cm for caissons subjected to rough sea conditions. There are even cases wherein concrete lids have thicknesses of not less than 1.0 m for caissons subjected to rough sea conditions for a long period of time with infill protected only by lids without superstructures (refer to **Fig. 3.1.11**). When using precast concrete lids for caissons subjected to rough waves, a rubble layer of 30 to 50 cm thick can be laid beneath each precast concrete lid to prevent possible occurrence of the washing out of filling sand through the gaps between the precast concrete lids and caissons with in situ concrete filled in the gaps washed away by waves.

⑤ There are other cases of caissons that have canvas sheets laid between concrete lids and filling sand as the measure to prevent the filling sand from being washed out through possible cracks in the concrete lids created when the lids are severely hit by rough waves.

⑥ Regarding the wave force acting on superstructure concrete, there are many factors that have not been fully elucidated. Therefore, superstructure concrete shall be constructed so that it can be integrated with breakwater bodies. For the construction joints of superstructure concrete, refer to Standard Specifications for Concrete Structures[23)] by the Japan Society of Civil Engineers. The methods used for enhancing the integration between the superstructure concrete and breakwater bodies include the casting of superstructure concrete with the top sections of caissons embedded into it, provision of concrete lids with indented surfaces (mostly for precast concrete lids), and installation of reinforcing bars or shaped steel between superstructure concrete and caissons (refer to **Fig. 3.1.12**). It is also preferable to provide construction joints between parapets and superstructure concrete with tenons, reinforcing bars, or shaped steel so that they can be integrated.

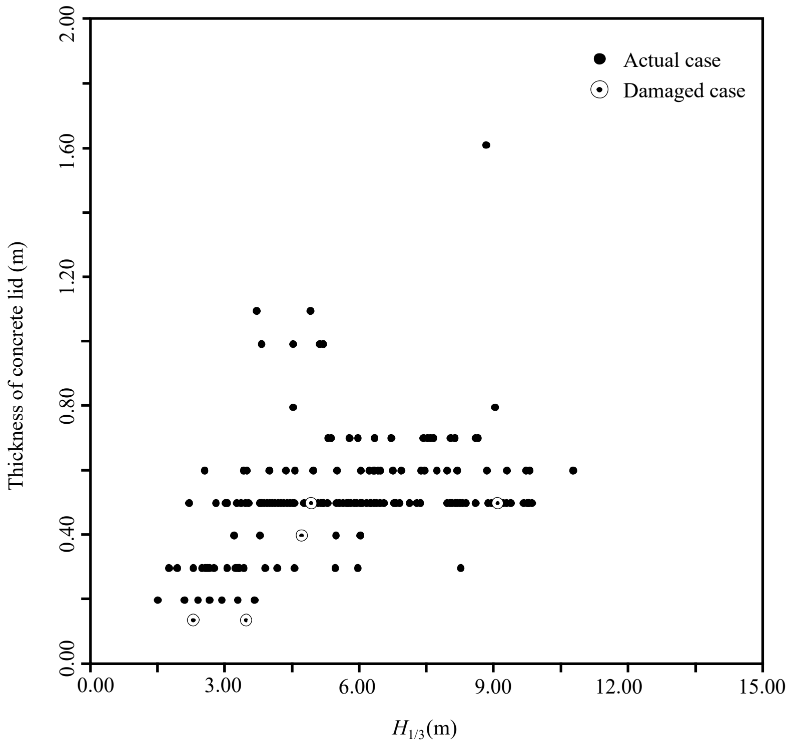


Fig. 3.1.11 Examples of the Construction of Concrete Lids

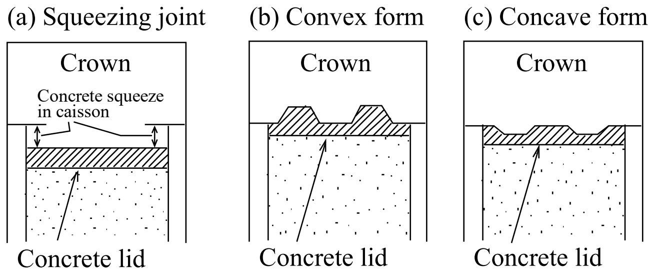


Fig. 3.1.12 Methods for Casting Superstructure Concrete

(3) Items for Block-type Composite Breakwaters

① Block-type composite breakwaters shall have the largest blocks possible. Particularly, it is preferable that the lowermost blocks shall be cast monolithically without joints.

② There are two methods for stacking blocks: one is horizontal stacking, and the other is inclined stacking. Generally, the former can be easily implemented and has been used often. In the case of horizontal stacking, it is preferable that vertical construction joints are not aligned from top to bottom in the cross sections perpendicular to the face lines of breakwaters but are arranged alternately to ensure the integration of breakwater bodies.

③ The inclined stacking method has also been implemented in cases of block-type breakwaters constructed in places with severe erosion or settlement and relatively shallow water depths. In such cases, superstructure concrete is constructed after blocks are sufficiently settled down. It is preferable that each cross section of breakwaters for inclined stacking comprises a single block. The inclination angles are generally 50° to 80° with respect to horizontal planes.

④ It is also preferable that the vertical construction joints are not aligned in the cross sections parallel to the face lines of breakwaters (longitudinal cross sections).

⑤ Blocks are generally provided with tenons and mortises (**Fig. 3.1.13**) to enable them to interlock with each other, thereby preventing them from sliding. Generally, the widths *a* and heights *b* of tenons are approximately

50 and 20 cm, respectively, and the widths *a'* and heights *b'* of mortises are larger than those of tenons by approximately 5 cm.

To prevent blocks from sliding, there is an alternative method in which blocks with prefabricated through-holes are stacked and integrated one above the other by filling the through-holes with concrete or by inserting steel materials into the through-holes with the gaps in them filled with mortar. In such a method, excessively small through-holes reduce the effect of preventing blocks from sliding and excessively large through-holes may cause the destruction of blocks. There are other alternative methods that use interlocking blocks, including deformed blocks with hexagonal or drum shapes. However, in the performance verification, the effects of these deformed interlocking blocks are generally ignored.

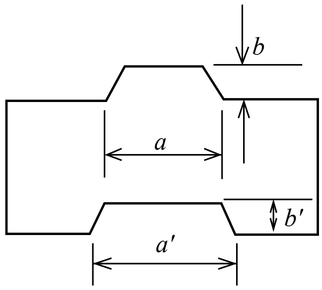


Fig. 3.1.13 Tenon and Mortise of Concrete Blocks

(4) Items for Cellular Block-type Composite Breakwaters

① The thicknesses of concrete lids of cellular block-type composite breakwaters shall be carefully determined by taking into consideration the wave and construction conditions.

② The lowermost cellular blocks should have footings to enhance stability.

③ Concrete or stones can be used as the infill of cellular blocks.

④ When stacking cellular blocks, the integration of cellular blocks shall be enhanced by interlocking effects with the tenons and mortises provided at the top and bottom portions of cellular block walls (**Fig. 3.1.1 [c]**).

⑤ Cellular blocks that are filled with stones may have bottom slabs to prevent the stones from being extruded.

(5) Items for In Situ Concrete Block-type Composite Breakwaters

① In many cases, each concrete block constituting the upright sections of in situ concrete block-type composite breakwaters has a size of 5 to 10 m to prevent cracks due to contraction or uneven settlement.

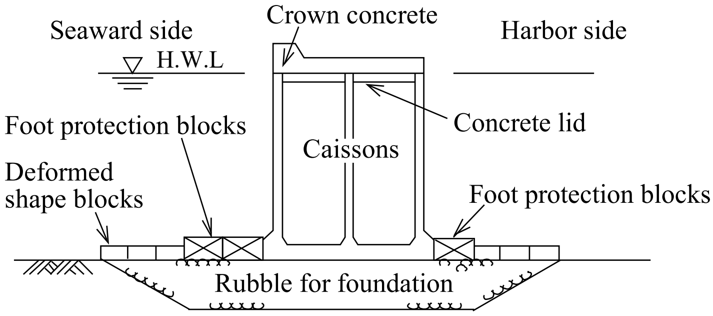
② In situ concrete can be cast by the underwater concrete, prepacked concrete, or dry work method.

3.2 Gravity-type Breakwaters (Upright Breakwaters)

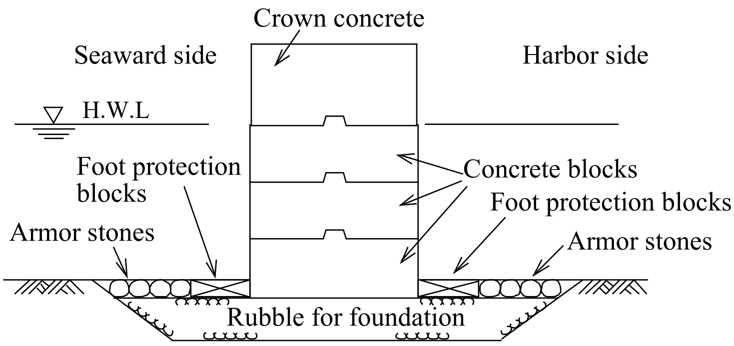
3.2.1 General

(1) Upright breakwaters have structures in which wall bodies with vertical front walls are installed on the seabed to reflect wave energy.

(2) **Fig. 3.2.1** shows examples of the cross sections of upright breakwaters.



1) Caisson type upright breakwater



2) Concrete block type upright breakwater

Fig. 3.2.1 Examples of Upright Breakwaters

3.2.2 Setting of Basic Cross Sections

The cross sections of upright breakwaters shall be appropriately set in conformity with **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

3.2.3 Actions

The actions for upright breakwaters shall be appropriately set in conformity with **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

3.2.4 Performance Verification

The performance verification of upright breakwaters can be performed with reference to **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

3.2.5 Structural Details

(1) The structural details of upright breakwaters shall be appropriately examined with reference to **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

(2) When constructing breakwater bodies with in situ concrete, unevenness on foundation surfaces is acceptable, but such surfaces shall be free from sand, rock fractions, or seaweeds to ensure the adhesiveness between in situ concrete and foundations. Furthermore, the portions of foundation surfaces that are brought into contact with

formworks shall be leveled to ensure the adhesiveness. In cases wherein there are difficulties in leveling remarkably hard and uneven seabed, it is preferable to ensure the adhesiveness between breakwater bodies and the seabed by flexibly adjusting the shapes of formworks in accordance with the topography of the uneven seabed.

(3) Considering that the lower sections of upright breakwaters are susceptible to scouring, upright breakwaters constructed on the seabed with no bedrock exposed shall be fully provided with foot protection work.

3.3 Gravity-Type Breakwaters (Sloping Breakwaters)

3.3.1 General

(1) Sloping breakwaters have structures where stones or concrete blocks are stacked in trapezoidal shapes in cross sections primarily for dissipating wave force by allowing waves to break on the slopes in front of breakwater bodies.

(2) **Fig. 3.3.1** shows examples of the cross sections of sloping breakwaters.

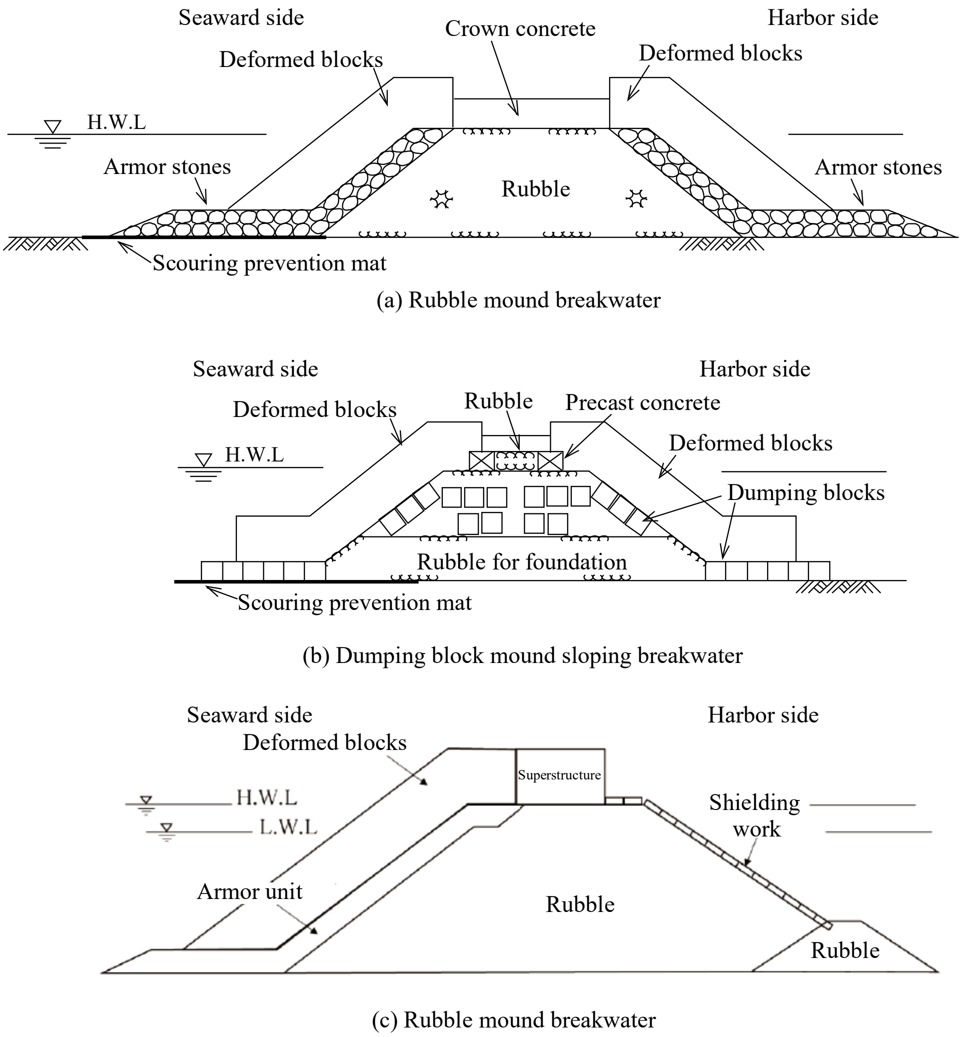


Fig. 3.3.1 Examples of the Cross Sections of Sloping Breakwaters

3.3.2 Setting of Basic Cross Sections

(1) The crown heights of sloping breakwaters can be set in conformity with composite breakwaters as described in **Chapter 4, 3.1.2 Setting of Basic Cross Sections**. Furthermore, the crown heights can be set in accordance with the intended use of the crowns.

(2) It shall be noted that because sloping breakwaters transmit waves, they may cause wave heights inside harbors to be higher than the cases of upright breakwaters even though the crown heights are identical. For overtopping and

transmitted waves, refer to **Part II**, **Chapter 2, 4.4.7 Wave Run-Up Heights, Overtopping Waves, and Transmitted Waves**.

(3) The crown widths can be set on the basis of the results of appropriate model tests.

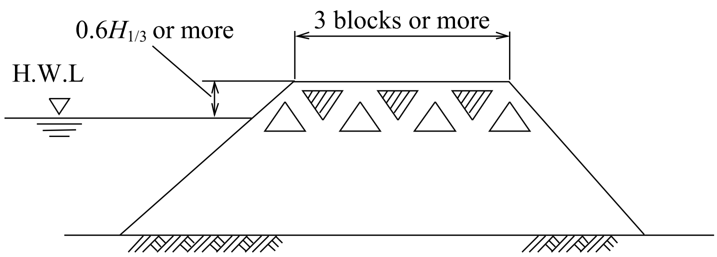
(4) Sloping breakwaters subjected to significant overtopping waves shall have sufficiently wide crown widths to prevent armor units at the top sections from being placed into an unstable state.

(5) The crown widths of rubble mound breakwaters to be constructed in a manner that extends rubble mounds from beaches out to the sea are preferably determined to satisfy the performance verification and to facilitate construction work.

(6) The crown heights and construction methods for sloping breakwaters to be constructed on soft ground can be set with reference to those for composite breakwaters as described in **Chapter 4, 3.1.2 Setting of Basic Cross Sections**.

(7) When deformed blocks are used for sloping breakwaters with crown heights of approximately $0.6H_{1/3}$ above the mean-monthly highest water levels, the crown widths can be equivalent to three rows of the deformed blocks or more (**Fig. 3.3.2**). However, it is preferable that the crown widths are determined via an appropriate model tests because the stability of the top sections of sloping breakwaters varies depending on the characteristics of armor units and wave conditions.

(8) According to the actual cases, the slope gradients are generally set at approximately 1:2 and 1:1.5 for the seaward and landward sides of rubble mound breakwaters, respectively, and approximately 1:1.3 to 1:1.5 for the rubble mound breakwaters covered by concrete blocks. In cases of seaward side slopes with different gradients and mass of armor units between upper and lower sections, the boundaries of the different gradients and mass shall be positioned at least $1.5H_{1//3}$ deeper than still water levels in general.



The number of pieces listed above are the number of hatched blocks in the upper layer of the crown.

Fig. 3.3.2 Crown Width of Sloping Breakwater

(9) The sloping breakwaters in the Europe and the U.S. tend to have high crown heights as a result of using the 2% exceedance run-up heights $R_{2\%}$ of random waves as target crown heights.

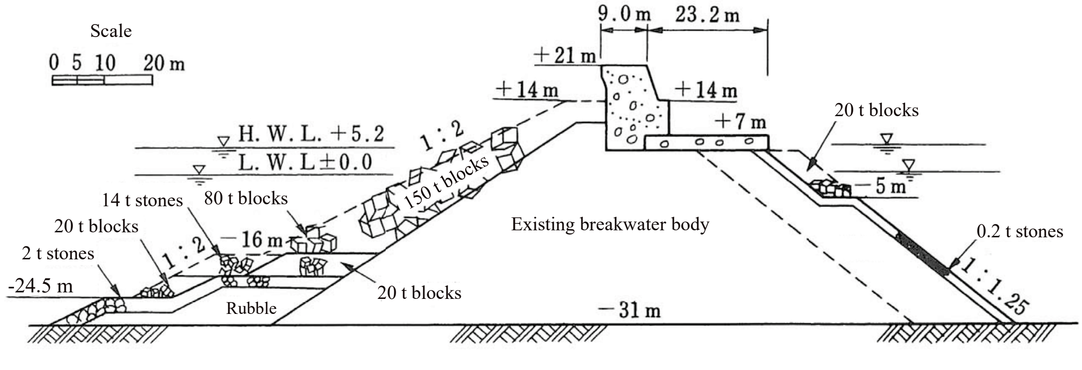


Fig. 3.3.3 Rubble Mound Breakwater (Typical Cross Section of the Head Section of Outer Breakwater at the Port of Bilbao[24)])

3.3.3 Performance Verification

(1) Performance verification items for sloping breakwaters

① Sloping breakwaters have problems with overtopping and transmitted waves and are subjected to the following failure modes: scouring and breakages of armor units; breakages, sliding, and overturning of superstructures; slip failures of front slopes; scouring of mounds below armor units; settlement of core materials; scouring of sandy ground at slope toes; washing-out of fine particle components due to internal instability of filtering materials; and ground settlement (**Fig. 3.3.4**). Therefore, the performance verification of sloping breakwaters shall be performed to prevent these failure modes.

② The performance verification items for sloping breakwaters include the stability of superstructures; the stability of armor units (rubble stones, concrete blocks, and deformed concrete blocks) at sloped sections, the required mass of rubble stones and blocks below the armor units at sloped sections and their internal stability as filtering layers, and the bearing capacity of sloped sections and ground.

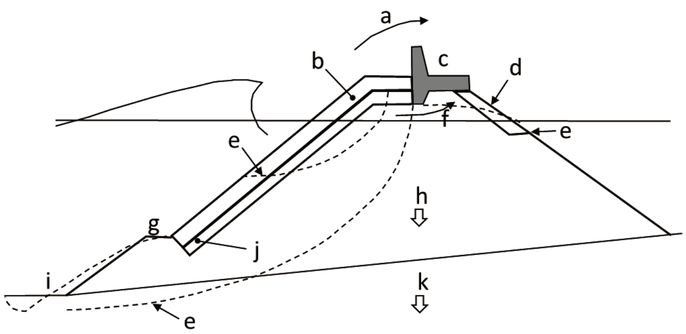


Fig. 3.3.4 Failure Modes of Sloping Breakwater (Refer to ISO 21650)

a: Overtopping waves b: Scouring and breakages of armor units
c: Breakages, sliding, and overturning of superstructures
d: Scouring of armor units e: Slip failures of front slopes
f: Transmitted waves g: Scouring of mounds below armor units;
h: Settlement of core materials i: Scouring of sandy ground at a slope toe
j: Internal instability of filtering materials
k: Ground settlement

(2) Performance verification of the stability of superstructures

① The verification of the stability of superstructures under the variable situation with respect to waves shall be performed for the sliding and overturning of superstructures.

② The verification of the stability of superstructures under the variable situation with respect to waves shall be performed using **Equations** (**3.3.1**) and (**3.3.2**). In these equations, the symbol γ is the partial factor for each subscript. Furthermore, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Tables 3.3.1** and **3.3.2**, in which the "—" symbol in a column indicates that the value in parentheses in the column can be used for the performance verification of convenience.

(a) Verification of sliding

$$m \cdot \frac{S_d}{R_d} \le 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$R_k = \{f_k (W_k - P_{B_k} - P_{U_K})\} \tag{3.3.1}$$

$$S_k = P_{Hk}$$

where

f : a friction coefficient between a superstructure and rubble stones;

W : weight of a superstructure (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift (kN/m);

P_H : horizontal wave force (kN/m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.3.1 Partial Factors Used for the Performance Verification of Sliding of Superstructures

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of superstructure (Variable state of waves)	– (1.00)	– (1.00)	1.20

(b) Verification of overturning

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$R_k = a_1 W_k - a_2 P_{B_k} - a_3 P_{U_k} \tag{3.3.2}$$

$$S_k = a_4 P_{H_k}$$

where

W : weight of a superstructure (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift (kN/m);

P_H : horizontal wave force (kN/m);

a_1 to a_4 : arm lengths of respective actions (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.3.2 Partial Factors Used for the Performance Verification of the Overturning of Superstructures

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of superstructure (Variable state of waves)	− (1.00)	− (1.00)	1.20

③ It is necessary to appropriately calculate the characteristic values for the weight W_k and the buoyancy of a superstructure in **equations** (**3.3.1**) and (**3.3.2**).

(3) Performance verification of armor units for sloped sections

① One of the methods for covering sloped sections is to use rubble stones or deformed concrete blocks as armor units, and another method is to cover sloped surfaces with sand mastic.

② The armor units for rubble sections shall have sufficient mass to ensure stability against waves and sufficient thicknesses to prevent infill from being washed out.

③ When calculating the required mass of armor units, refer to **Part II, Chapter 2, 6.6 Stability of Armor Rocks and Blocks against Waves** or **ISO 21650**.

④ The required mass of armor units shall be appropriately set when constructing armor layers not randomly but by orderly arranging armor units or by laying armor stones. The number of layers is generally set at two when constructing armor layers by randomly arranging armor units.

⑤ For the use of sand mastic to cover sloped surfaces, refer to past use cases and the research outcome[7)].

(4) Required mass of rubble stones and blocks below armor units at sloped sections and their internal stability as filter layers

① The required mass of filter layers (rubble stones and blocks) below armor units at sloping breakwaters are preferably approximately 1/10 to 1/15 or more of the mass of armor units. The mass of the stones (core materials) below the filter layers are preferably approximately 1/20 or more of the mass of filter layers.[1)]

② The verification of the stability of the mass of the stones (core materials) below the filter layers can be performed with reference to the following equation (ISO 21650):

$$\frac{d_{15,filter}}{d_{85,core}} < 4 \ to \ 5$$
$$\frac{W_{50,filter}}{W_{50,core}} < 15 \ to \ 20 \qquad \textbf{(3.3.3)}$$

where d and W represent the particle diameter and the mass of a stone or a concrete block, respectively; $d_{15,\mathrm{filter}}$ is the sieve size for 15% passing by mass; $d_{85,\mathrm{core}}$ is the sieve size 85% passing by mass; $W_{50,\mathrm{filter}}$ is the mass of a filter material with a median diameter; and $W_{50,\mathrm{core}}$ is the mass of a core material having a median diameter.

Furthermore, the verification of the internal stability of filter materials can be performed with reference to the following condition.

$$\frac{d_{60}}{d_{10}} < 10 \qquad \textbf{(3.3.4)}$$

(5) Bearing capacity of sloped sections and ground

① The stability of the sloped section of sloping breakwaters can be examined via the verification of the circular slip failures of rubble layers and their sliding failures due to eccentric and inclined loads.

② For the verification of the circular slip failures of rubble layers and the sliding failures due to eccentric and inclined loads, refer to **Part III, Chapter 2, 4.2.1 Stability Analysis by Circular Slip Surfaces** and **Part III, Chapter 2, 3.2.5 Bearing Capacity of Eccentric and Inclined Actions**, respectively.

(6) Performance verification of the stability of head sections

The head sections of sloping breakwaters are preferably constructed in a semicircular form by using armor units with a 1.5 times larger or more mass than those used for trunk sections. When calculating the mass of armor stones and wave dissipating blocks, refer to **Part II, Chapter 2, 6.6 Stability of Armor Stones and Blocks against Waves**. Generally, it is preferable that the stability of head sections be verified via hydraulic model tests.

3.3.4 Performance Verification of Structural Members

For the performance verification of structural members, refer to **Part III, Chapter 2, 2 Structural Members**.

3.3.5 Structural Details

(1) The foundations of sloping breakwaters shall be provided with scouring and washing-out prevention measures as needed.

(2) The scour prevention measures include small stepped rubble mounds at slope toes or the protection of slope toes with rubble blocks, submerged floor mats, asphalt mats, or synthetic resin mats (refer to **Fig. 3.3.1**).

(3) The measures to prevent rubble mounds from settlement due to washing-out include the installation of submerged floor mats or the laying of canvas sheets.

(4) Generally, when constructing superstructures on rubble block and rubble mound breakwaters, the rubble foundations of superstructures shall be blinded with small rubble blocks.

(5) The surface finish work of sloping breakwaters shall be implemented in a manner that ensures the adequate interlocking effects of surface armor unit materials with careful attention to the finishing of crown sections.

(6) In coastal areas affected by littoral drifts, sloping breakwaters are preferably provided with sediment infiltration prevention work to prevent harbors from possible siltation owing to sand passing through sloping breakwaters together with waves.

(7) Sediment infiltration prevention work is normally implemented in a manner that constructs walls with sheet piles or blocks inside breakwaters or dumps stone materials with a wide particle size distribution inside the sloping breakwaters or on the slopes at a harbor side.

(8) It shall be noted that sloping breakwaters are susceptible to wave actions that scatter stones.

(9) For the mixture of materials to be used when covering sloping breakwaters by sand mastic method, refer to **Part II, Chapter 11, 4 Asphalt Materials**.

(10) When constructing sloping breakwaters on soft ground, the settlement and subduction of breakwater bodies generally cause the quantities of rubble stones or blocks required in actual construction to be significantly larger than those based on the cross sections obtained by performance verification. Even in cases of favorable ground conditions, additional quantities of stones are preferably procured in actual construction in anticipation of the scattering and consolidation of stones due to waves.

3.4 Gravity-Type Breakwaters (Breakwaters Covered with Wave-Dissipating Blocks)

3.4.1 General

(1) Breakwaters covered with wave-dissipating blocks have structures wherein wave-dissipating blocks are installed in front of composite or upright type breakwaters so that wave energy can be dissipated by the blocks and transmitted waves can be blocked by upright sections.

(2) **Fig. 3.4.1** shows examples of the cross sections of breakwaters covered with wave-dissipating blocks.

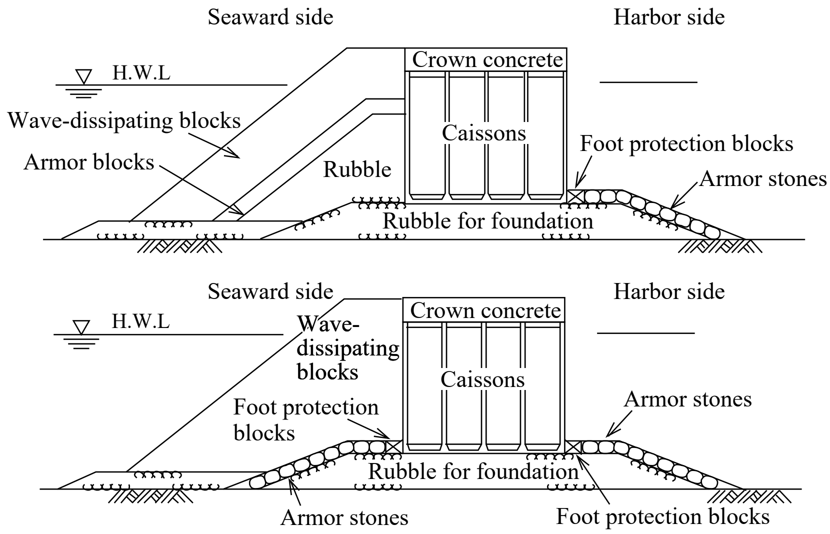


Fig. 3.4.1 Examples of Cross Sections of Breakwaters Covered with Wave-Dissipating Blocks

3.4.2 Setting of Basic Cross Sections

(1) The crown height of breakwaters covered with wave-dissipating blocks can be set in conformity with composite breakwater as described in **Chapter 4, 3.1.2 Setting of Basic Cross Sections**.

(2) Generally, wave-dissipating blocks with excessively lower crown heights than upright sections may allow impulsive breaking wave force to act on the upright sections, and wave-dissipating blocks with excessively higher crown heights than upright sections may destabilize the uppermost wave-dissipating blocks.

(3) Wave-dissipating blocks shall have crown widths equivalent to two rows of wave-dissipating blocks or more to enable them to exert the full wave-dissipating effect.[25), 26)]

(4) The thicknesses of superstructures and the crown heights of installed caissons can be set in compliance with upright breakwaters, and the thicknesses of rubble sections can be set in compliance with composite breakwaters.

(5) If crown heights are identical, breakwaters covered with wave-dissipating blocks can attenuate the degrees of overtopping and transmitted waves compared with upright and composite breakwaters. For overtopping and transmitted waves, refer to **Part II, Chapter 2, 6 Waves**.

(6) Wave-dissipating blocks can reduce wave pressure, attenuate the overtopping and transmitted waves, and curb reflected waves. Model tests should be conducted to accurately assess these effects.

(7) It is necessary to pay attention to the possible impulsive breaking wave force applied to the portions of upright sections where wave-dissipating blocks have not been fully installed during construction.

3.4.3 Actions

(1) For the actions on breakwaters covered with wave-dissipating blocks, refer to **Chapter 4, 3.1.3 Actions.**

(2) Generally, the self-weight of wave-dissipating blocks leaning on caissons is not considered an action on the caissons when breakwaters are subjected to waves. When considering the self-weight of wave-dissipating blocks as the action on caissons, refer to **Part II, Chapter 2, 6.2.5 Wave Force Acting on Vertical Walls Covered by Wave-Dissipating Blocks**.

3.4.4 Performance Verification of the Overall Stability of Breakwater Bodies

(1) For the verification of the stability of breakwaters covered with wave-dissipating blocks, refer to **Chapter 4, 3.1 Gravity-Type Breakwaters (Composite Breakwaters)**.

(2) The performance verification of the sliding and overturning failures of breakwaters covered with wave-dissipating blocks with respect to variable waves shall be performed using **equations (3.1.2)** and **(3.1.4)**. The partial factors in the equation can be selected from the values in **Tables 3.4.1** and **3.4.2**, in which the "—" symbol in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

Table 3.4.1 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of breakwater body (Variable state of waves)	0.79	0.90	– (1.00)

Table 3.4.2 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	0.98	0.99	– (1.00)

(3) The partial factors above were set with reference to the safety levels in past standards.[5)] Furthermore, the partial factors are set under the conditions that the topographies of seabed where breakwaters are installed have gradients of less than 1/30. In cases wherein the topographies of seabed have gradients larger than 1/30, partial factors shall be appropriately set with reference to the descriptions in **Reference 5)**.

(4) In **Reference 5)**, the partial factors were set under the condition that the crown heights of wave-dissipating blocks are not affected by settlement. Given that the crown settlement of wave-dissipating blocks poses a risk of the immediate destabilization of breakwater bodies when the wave force acting on them increases, it is necessary to pay attention to providing wave-dissipation blocks with settlement prevention measures.

3.4.5 Performance Verification of Other Items Related to the Overall Stability of Breakwater Bodies

(1) Performance verification of the stability of sloped sections

① For calculating the required mass of armor units for breakwaters covered with wave-dissipating blocks, refer to **Part II**, **Chapter 2, 6.6.2 Required Mass of Armor Rocks and Blocks of Composite Breakwater Mound against Waves**.

② The required mass of armor units shall be appropriately set when constructing armor layers not randomly but by orderly arranging armor units or by laying armor stones. The number of layers is generally set at two when constructing armor layers by randomly arranging armor units.

(2) Performance verification of the stability of head sections

The head sections of breakwaters covered with wave-dissipating blocks are preferably constructed in a semicircular form by using armor units with a 1.5 times larger or more mass than those used for trunk sections. When calculating the mass of wave-dissipating blocks, refer to **Part II, Chapter 2, 6.6.2 Required Mass of Armor Rocks and Blocks of Composite Breakwater Mound against Waves**.

(3) Performance verification of the stability of wave-dissipating works

For the performance verification of the stability of wave-dissipation works, refer to Part II, Chapter 2, 6.6.2 Required Mass of Armor Rocks and Blocks of Composite Breakwater Mound against Waves.

(4) In the performance verification of the accidental situation with respect to Level 2 earthquake ground motions, refer to Reference **(Part III), Chapter 1, 2 Basic Items Concerning Earthquake Response Analysis**.

3.4.6 Performance Verification of Structural Members

The performance verification of structural members can be performed with reference to **Part III, Chapter 2, 2 Structural Members**.

3.4.7 Structural Details

(1) The dimensions of foot protection blocks at the seaward side of breakwaters covered with wave-dissipating blocks can be calculated by **equation (3.1.15)** in **Chapter 4, 3.1 Gravity-Type Breakwaters (Composite Breakwaters)** for the waves during construction by taking into consideration the periods that breakwaters are left without wave-dissipating blocks during construction.

(2) Generally, because wave-dissipating blocks are susceptible to the scouring and washing out of foundation ground due to waves, they are provided with scouring and washing-out prevention work at the slope toes to prevent settlement, as needed.[27), 28), 29)] For the souring and washing-out prevention work, refer to **Chapter 4, 3.3.5 Structural Details**.

3.5 Gravity-Type Breakwaters (Upright Wave-Absorbing Block-Type Breakwaters)

3.5.1 General

(1) Upright wave-absorbing block-type breakwaters are mass concrete block-type upright breakwaters or composite breakwaters that are constructed by directly stacking special blocks with a wave-absorbing function (upright wave-absorbing blocks).

(2) Upright wave-absorbing block-type breakwaters, except those with large-scale monolithic structures, are normally used as breakwaters in inner bays or inner harbors where wave heights are relatively low.

(3) **Fig. 3.5.1** shows an example of a cross section of an upright wave-absorbing block-type breakwater.

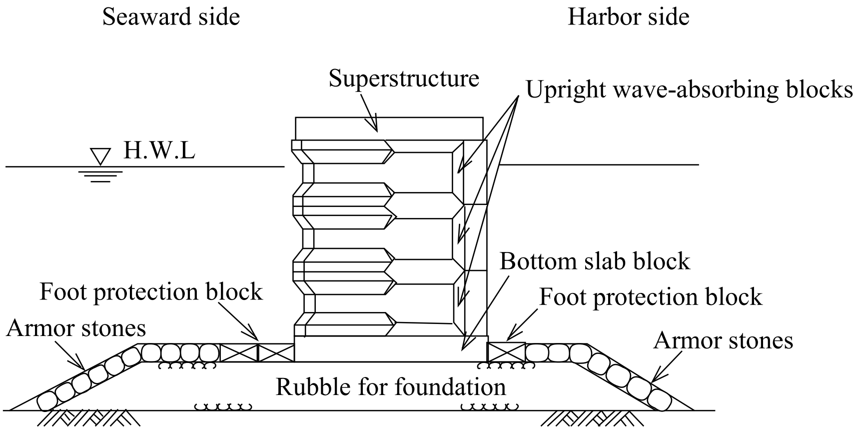


Fig. 3.5.1 Example of a Cross Section of an Upright Wave-Absorbing Block-Type Breakwater

3.5.2 Setting of Basic Cross Sections

(1) It is necessary that the structural dimensions of upright wave-absorbing block-type breakwaters are determined to enable them to deliver the required wave-absorbing performance.

(2) The crown heights of the upright wave-absorbing block-type breakwaters can be decided by considering the heights that satisfy the performance requirements and the heights of the wave-absorbing sections and by referring to **Chapter 4, 3.1.4 Performance Verification of Overall Stability of Breakwater Bodies**. The crown heights of the wave-absorbing sections shall be determined by considering the wave-absorbing performance. In cases of structures with permeability, the dimensions of the opening sections should be determined by considering the transmission characteristics.

(3) The wave-absorbing performance of the upright wave-absorbing block-type breakwaters vary depending on the crown heights and bottom elevations of the wave-absorbing block sections.

(4) Several types of wave-absorbing blocks have been developed. Appropriate types of blocks are preferably selected after sufficiently studying their wave-absorbing performance.

(5) In the upright wave-absorbing block-type breakwaters, wave over-topping and transmitted waves are small in comparison with those with composite breakwaters but tend to be larger than those with breakwaters covered with

wave-absorbing blocks. Accordingly, it is preferable that the crown heights be determined by giving adequate consideration to the conditions of use behind breakwaters. Furthermore, in determining the crown heights, the thickness required for constructing the crown concrete should be secured.

(6) It is preferable that the crown heights h_c' be at least 0.5 times or higher the significant wave heights used in the stability examination of the facilities above the mean monthly high-water levels. The bottom heights h_u should be at least two times or higher the significant wave height used in the stability examination of the facilities below the mean monthly high-water levels (see **Fig. 3.5.2**).

(7) For the run-up heights of the upright wave-absorbing block-type breakwaters and the characteristics of several types of wave-absorbing blocks, refer to **References 30)** and **31)**, respectively.

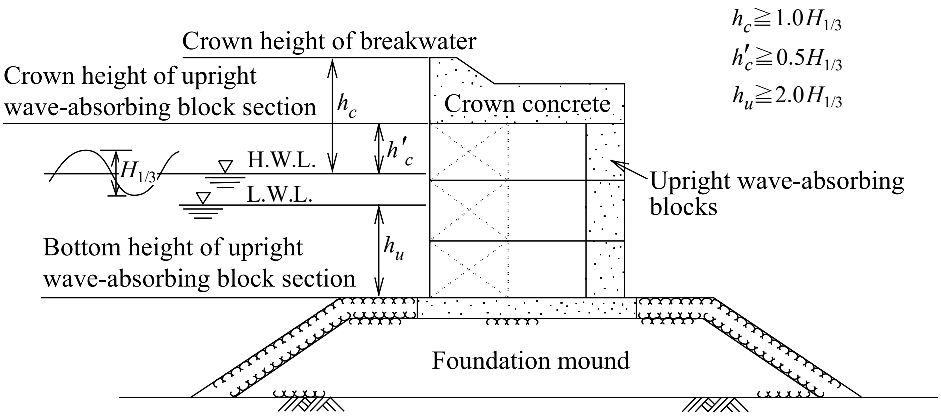


Fig. 3.5.2 Explanatory Diagram for the Crown Height of Upright Wave-Absorbing Block-Type Breakwater

3.5.3 Actions

(1) Depending on the purpose of absorbing waves and wave conditions, the characteristics of waves used for the verification of wave-absorbing performance can be determined separately from those used for the performance verification of the stability of facilities and structural members.

(2) Wave force shall be determined by the equation appropriate for upright wave-absorbing block-type breakwaters or via hydraulic model tests simulating actual wave conditions. For breakwaters with complex structures, the wave force acting on structural members should be studied in addition to the wave force used for verifying the stability of entire upright sections. For the wave force acting on upright wave-absorbing block-type breakwaters, refer to **Part II, Chapter 2, 6.2.7 Wave Force Acting on Upright Wave-Absorbing Caissons**.

(3) Considering that the reflection coefficients of upright wave-absorbing block-type breakwaters significantly vary depending on wave periods, the reflection coefficients shall be determined with due consideration to the influences of reflection waves. The reflection coefficients are preferably determined by hydraulic model tests that simulate actual conditions or may be determined by referring to existing test results.

3.5.4 Performance Verification of Overall Stability of Breakwater Bodies

(1) Given that the hydraulic characteristics of upright wave-absorbing block-type breakwaters such as the transmittance of waves and water permeability have not been fully elucidated, the performance verification with respect to wave actions are preferably performed on the basis of hydraulic model tests simulating actual conditions.

(2) The verification of the stability of upright wave-absorbing block-type breakwaters can be performed in conformity with that of composite breakwaters. The standard partial factors to be used for the verification of sliding and overturning failures are shown in **Tables 3.5.1 and 3.5.2**, in which the "—" symbol in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

Table 3.5.1 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

Table 3.5.2 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

3.5.5 Performance Verification of Structural Members

The performance verification of structural members can be performed with reference to **Part III, Chapter 2, 2 Structural Members**.

3.6 Gravity-Type Breakwaters (Wave-Absorbing Caisson-Type Breakwaters)

3.6.1 General

(1) Wave-absorbing caisson-type breakwaters are classified as deformed caisson breakwaters that use caissons with special shapes. They are provided with porous walls and wave chambers at their front sections to deliver the wave-absorbing effect.[32)]

(2) Compared with composite breakwaters, wave-absorbing caisson-type breakwaters have the following features:

① They can curb reflected waves.

② They can attenuate wave overtopping and transmitted waves.

③ They can reduce wave force. In particular, they can curb significant increases in wave force even under severe wave conditions, whereas conventional caissons on high foundation mounds undergo strong impulsive breaking wave force.

④ They possess a seawater aeration function with porous walls and wave chambers that enhance the mixing of air bubbles with seawater. Furthermore, wave chambers are effective as fishing banks.[33), 34)]

(3) **Fig. 3.6.1** shows an example of the cross section of a wave-absorbing caisson-type breakwater. Depending on the shapes of the respective elements and the combination of elements, various types of structures are conceivable, including vertical slit-wall caissons, horizontal slit-wall caissons, curved-slit caissons, perforated-wall caissons, and others. Regarding the structural type for wave-absorbing caisson-type breakwaters, an appropriate structure should be selected by considering the design conditions, use conditions, economy, etc., on the basis of a careful investigation of the wave-absorbing performance and wave resistance of each structure.

(4) For the structures and the features of various types of wave-absorbing caisson-type breakwaters, the **Technical Manual of New Type Breakwaters**[35)] can be used as a reference.

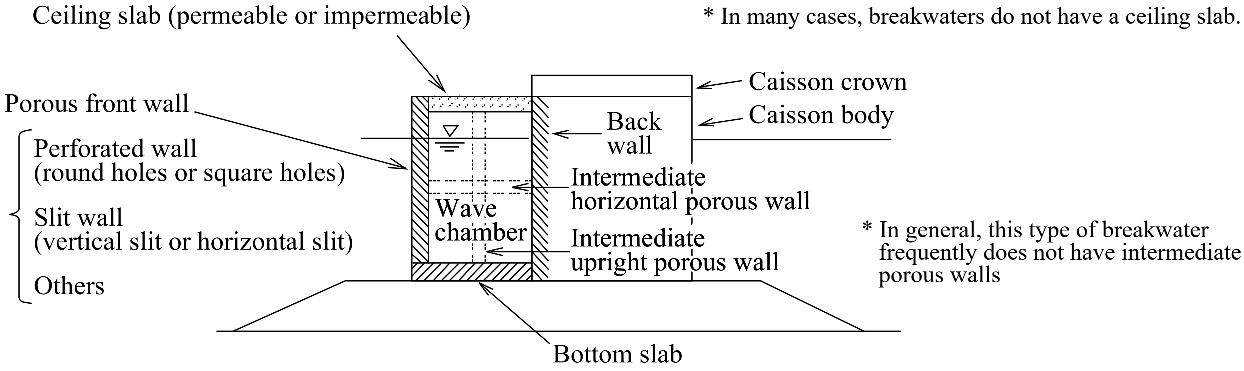


Fig. 3.6.1 Example of a Cross Section of Wave-Absorbing Caisson-Type Breakwater

3.6.2 Setting of Basic Cross Sections

(1) The structures of wave-absorbing caisson-type breakwaters shall be appropriately selected with due consideration to their wave-absorbing performance.

(2) In wave-absorbing caisson-type breakwaters, the required dimensions should be determined appropriately by considering the shapes of the structures. Given that the transmission coefficients differ depending on the structures or wave conditions, it is preferable that the crown heights that correspond to the transmission characteristics of the objective structures should be determined appropriately. In cases wherein the structures have permeability, it is preferable that the dimensions of the opening sections should be determined appropriately.

(3) In addition to wave-absorbing performance, the structure and dimensions of the wave-absorbing section are also related to wave overtopping, transmitted waves, and wave force. Therefore, it is preferable to determine the dimensions and structure by considering these characteristics.

(4) The reflection coefficients of wave-absorbing caisson-type breakwaters vary depending on the factors, including wave characteristics, water depths, structures of frontal porous walls, widths of wave chambers, presence or absence of ceiling slabs and their heights, heights of mounds, and others. Therefore, the structural dimensions of wave-absorbing sections shall be appropriately determined so that the reflection coefficients of object waves do not exceed the target reflection coefficients with due consideration to the effects of the above factors. In terms of enhancing the wave-absorbing performance, it is preferable that wave chambers have sufficiently high crown heights or have no ceiling slabs (permeable).

(5) For the reflection characteristics of vertical slit-wall caissons without ceiling slabs, the research by Tanimoto and Yoshimoto[36)] can be used as a reference.

3.6.3 Actions

(1) Depending on the purpose of absorbing waves and wave conditions, the conditions of waves used for the verification of wave-absorbing performance can be determined separately from those used for the performance verification of the stability of facilities and structural members.

(2) In many cases, wave-absorbing caissons are generally adopted to reduce reflected waves. Consequently, it is preferable to determine the conditions of the waves to be objects of wave absorption and target reflection coefficients corresponding to the required wave-absorbing performance. In particular, because the reflection coefficients of wave-absorbing caissons differ remarkably depending on the wave periods, the conditions of waves as objects of wave absorption shall be determined on the basis of the investigations of the characteristics of wave heights and wave periods.

(3) The wave force shall be determined by the equation appropriate for wave-absorbing caisson-type breakwaters or through hydraulic model tests simulating actual wave conditions. For breakwaters with complex structures, it is preferable to sufficiently study the wave force acting on structural members, in addition to the wave force used for verifying the stability of entire upright sections. For the wave force acting on wave-absorbing caisson-type breakwaters, refer to **Part II, Chapter 2, 6.2.7 Wave Force Acting on Upright Wave-Absorbing Caissons**.

(4) The performance verification of structural members shall be performed with the most severe wave force for respective members. For the wave force acting on the structural members of wave-absorbing caisson-type

breakwaters, refer to **Part II, Chapter 2, 6.2.7 Wave Force Acting on Upright Wave-Absorbing Caissons** and **[Facilities], Chapter 4, 3.5.3 Actions**.

3.6.4 Performance Verification

(1) Given that the hydraulic characteristics of wave-absorbing caisson-type breakwaters such as the transmittance of waves, reflection rates and water permeability have not been fully elucidated, the performance verification with respect to wave actions are preferably performed on the basis of hydraulic model tests as needed.

(2) The performance verification of the stability of wave-absorbing caisson-type breakwaters can be performed in conformity with that of composite breakwaters. The standard partial factors to be used for the verification of sliding and overturning failures are shown in **Tables 3.6.1** and **3.6.2**, in which the "—" symbol in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

Table 3.6.1 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

Table 3.6.2 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

3.6.5 Performance Verification of Structural Members

The performance verification of structural members can be performed with reference to **Part III, Chapter 2, 2 Structural Members**.

3.7 Gravity-Type Breakwaters (Sloping-Top Caisson Breakwaters)

3.7.1 General

(1) Sloping-top caisson breakwaters are classified as deformed caisson breakwaters that use caissons with special shapes. They are configured to reduce horizontal wave force while simultaneously using part of the wave force acting on the sloping walls of the superstructures to stabilize breakwater bodies. They are generally called sloping-top breakwaters.

(2) In sloping-top caisson breakwaters, the sloping walls of the superstructures are normally positioned above still water levels; however, semi-submerged sloping walls with their lower ends positioned below still water levels can further reduce wave force.[37)]

(3) **Fig. 3.7.1** shows an example of the cross section of a sloping-top caisson breakwater.

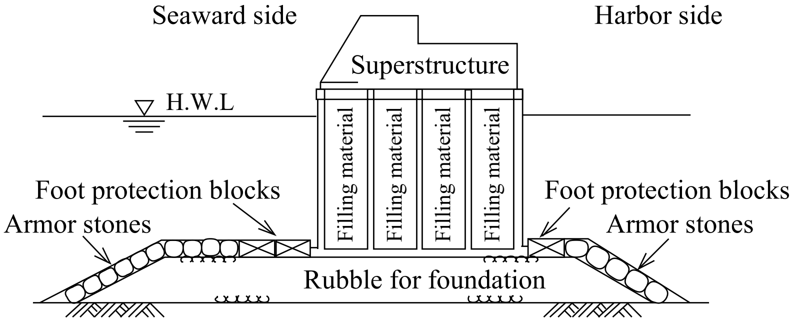


Fig. 3.7.1 Example of a Cross Section of Sloping-Top Caisson Breakwater

3.7.2 Setting of Basic Cross Sections

(1) The required dimensions of sloping-top caisson breakwaters shall be appropriately determined with consideration to their shapes. In particular, crown heights shall be appropriately determined in accordance with the transmission characteristics of structures because the transmission coefficients vary depending on structural types and wave conditions.

(2) The crown heights of sloping-top caisson breakwaters are preferably determined in consideration of harbor calmness because they increase the transmitted wave heights compared with conventional upright breakwaters.

(3) When crown heights are identical, sloping-top caisson breakwaters have wave height transmission coefficients that are approximately two times those of upright breakwaters[38)] (**Fig. 3.7.2**). Therefore, sloping-top caisson breakwaters with crown heights set to the similar level of significant wave heights $H_{1/3}$ can reduce transmitted wave heights to the same level, similar to the case for those upright breakwaters with crown heights set to 0.6 times the significant wave heights.

(4) With the increase in the gradients of sloping walls, sloping-top caisson breakwaters can be more effective in curbing the waves transmitted into harbors but receive a larger wave pressure, thereby reducing the effects of sloping breakwaters. According to the hydraulic model tests that observed the transmission coefficients by changing the gradients of sloping walls, no remarkable differences were identified in the transmission coefficients measured with the gradients of 30°, 45°, and 60°. Therefore, the gradient of sloping walls is preferably set at 45° by taking into consideration the wave pressure reduction effects and facilitation of construction work.

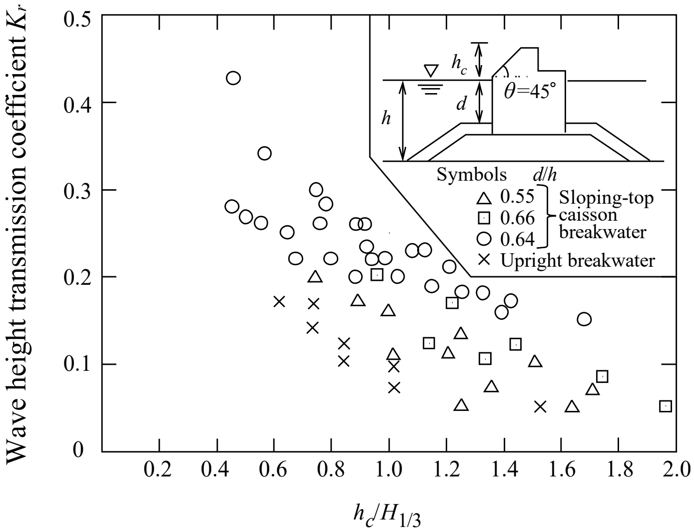


Fig. 3.7.2 Wave Height Transmission Coefficients and Relative Crown Heights

(5) When covering the upright front sections of caissons with wave-dissipating blocks, sloping-top caisson breakwaters may be subjected to impulsive breaking wave pressure depending on the crown heights of wave-dissipating blocks. Furthermore, in such a case, it is necessary to pay attention to the stability of wave-dissipating blocks because they are stacked up to still water levels.[39)]

3.7.3 Actions

(1) Although the wave force acting on sloping-top caisson breakwaters is preferably determined by hydraulic model tests, the provisions in **Part II, Chapter 2, 6.2.6 Wave Force Acting on Sloping-Top Caisson Breakwaters** can be used as a reference when the implementation of hydraulic model tests is difficult.

(2) Depending on the purpose of dissipating waves and wave conditions, the characteristics of waves used for the verification of wave-absorbing performance can be determined separately from those used for the performance verification of the stability of facilities and structural members.

(3) For the wave force acting on sloping-top caisson breakwaters covered with wave-dissipating blocks, the study result by Sato et al.[39)] can be used as a reference.

3.7.4 Performance Verification

(1) The performance verification of sloping-top caisson breakwaters should be performed by hydraulic model tests as needed with appropriate breakwater shapes selected on the basis of sufficient research on wave transmission characteristics.

(2) The verification of the stability of sloping-top caisson breakwaters can be performed in conformity with that of composite breakwaters. **Tables 3.7.1** and **3.7.2** show the standard partial factors to be used for the verification of sliding and overturning failures. For cases wherein the front faces of sloping-top caisson breakwaters are covered with wave-dissipating blocks, the standard partial factors can be selected from the values in the lower columns of the respective tables.

Table 3.7.1 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of sloping-top caisson breakwater (Variable state of waves)	0.84	1.11	— (1.00)
Sliding of sloping-top caisson breakwater covered with wave-dissipating blocks (Variable state of waves)	0.76	0.95	— (1.00)

Table 3.7.2 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of sloping-top caisson breakwater (Variable sate of waves)	0.98	1.17	— (1.00)
Overturning of sloping-top caisson breakwater covered with wave-dissipating blocks (Variable state of waves)	0.98	1.06	— (1.00)

(3) The partial factors above have been set with reference to the safety levels in the past standards.[40)] Furthermore, the partial factors are set under the conditions that the topographies of seabed where breakwaters are installed have gradients of less than 1/30. In cases of the topographies of seabed with gradients larger than 1/30, partial factors shall be appropriately set with reference to the descriptions in **Reference 40)**.

3.7.5 Performance Verification of Structural Members

The performance verification of structural members can be performed with reference to **Part III, Chapter 2, 2 Structural Members**.

3.8 Pile-Type Breakwaters

[Public Notice] (Performance Criteria of Pile-Type Breakwaters)

Article 36

The performance criteria for pile-type breakwaters under variable situations, in which the dominating actions are variable waves and Level 1 earthquake ground motion are prescribed in the following items:

(1) The risk that the axial force acting on the piles may exceed the resistance based on the failure of the ground shall be equal to or less than the threshold level.

(2) The risk that the stress generated in the piles may exceed the yield stress shall be equal to or less than the threshold level.

[Interpretation]

10. Protective Facilities for Harbors

(4) **Performance Criteria of Pile-Type Breakwaters** (Article 14, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 36 of the Public Notice)

① The required performance of pile-type breakwaters under the variable action situation in which the dominating actions are variable waves and Level 1 earthquake ground motions shall focus on Serviceability. **Attached Table 10-5** shows the performance verification items and standard indexes for determining the limit values with respect to the actions.

Attached Table 10-5 Performance Verification Items and Standard Indexes for Determining the Limit Values of Pile-Type Breakwaters

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index for determining the limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="2">14</td><td rowspan="2">1</td><td rowspan="2">2</td><td rowspan="2">36</td><td rowspan="2">-</td><td>1</td><td rowspan="2">Serviceability</td><td rowspan="2">Variable</td><td rowspan="2">Variable waves [Level 1 earthquake ground motion]</td><td rowspan="2">Self-weight, water pressure</td><td>Axial force acting on the pile</td><td>Action–resistance ratio with respect to the bearing capacity of the pile (Pushing and pulling out)</td></tr>
<tr><td>2</td><td>Yield stress of the pile</td><td>Design yield stress</td></tr>
</table>

* [ ] means alternative dominating action to be studied as design situations.

② In addition to the requirements above, the superstructures and curtains of pile-type breakwaters shall conform to the requirements and commentaries in Articles 23 to 27 of the Public Notice depending on the types of members constituting the pile-type breakwaters.

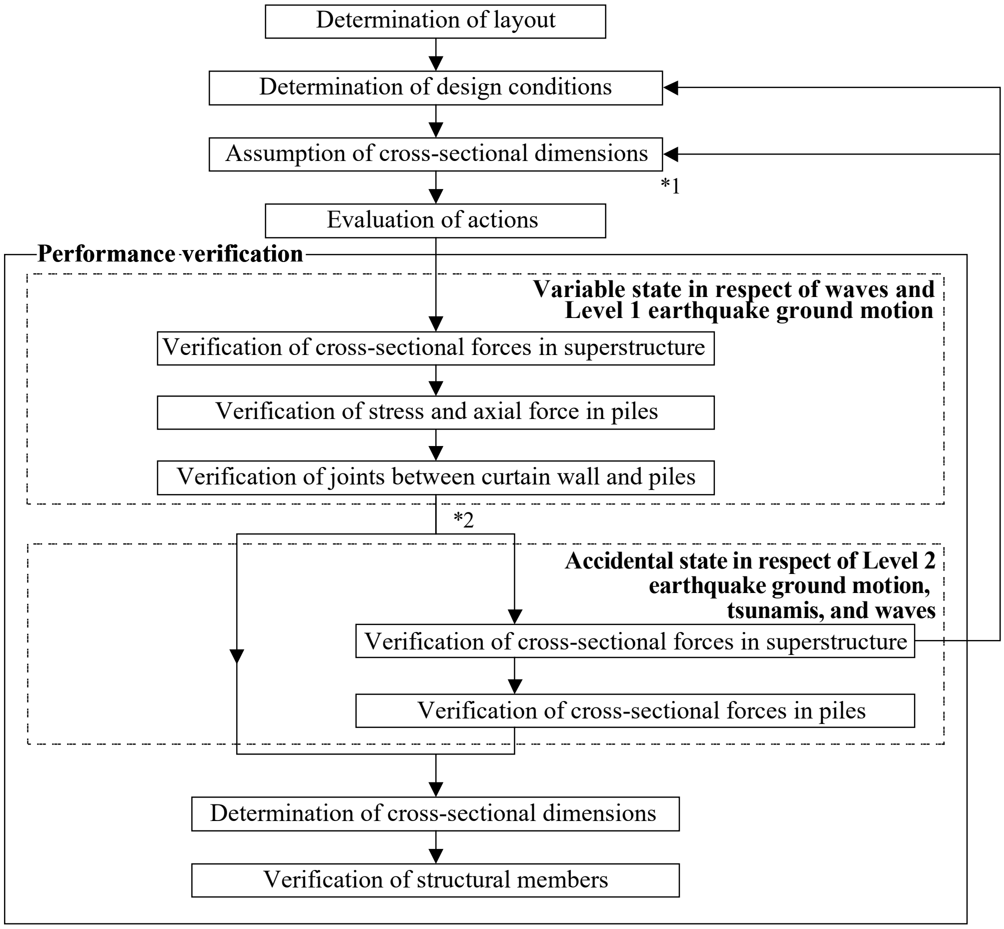
3.8.1 General

(1) Pile-type breakwaters can be broadly divided into curtain wall breakwaters and steel pipe breakwaters. Curtain wall breakwaters are permeable breakwaters of pile structure developed for use in water areas with comparatively low wave heights, such as enclosed bays, or locations with soft sea bottom ground, whereas steel pipe breakwaters are breakwaters that stop waves only by using steel pipe piles.

(2) The performance verification of curtain wall breakwaters should be performed on the basis of hydraulic model tests as needed, with their structures appropriately selected with consideration to reflection and transmission coefficients.

(3) **Fig. 3.8.1** shows an example of the performance verification procedure for curtain wall breakwaters. However, because **Fig. 3.8.1** does not show the evaluation of the effects of liquefaction due to earthquake ground motions, it

is necessary to appropriately deliberate the possibility of and the measures against liquefaction by referring to **Part II, Chapter 7 Liquefaction of Ground**.



*1: Because assessment of the effects of liquefaction is not shown, separate consideration is necessary.
*2: For facilities where damage to the facilities can be assumed to have a serious impact on life, property, and social activity, it is preferable to conduct verification for accidental situations when necessary. Verification for accidental situations in respect of waves shall be conducted in cases where facilities handling hazardous cargoes are located directly behind the breakwater and damage to the objective facilities would have a catastrophic impact.

Fig. 3.8.1 Example of the Performance Verification Procedure for Pile-Type Breakwaters

(4) Curtain wall breakwaters can be broadly divided into single-curtain-wall breakwaters and double-curtain-wall breakwaters depending on how the so-called curtain walls, such as concrete plates, are arranged relative to the directions of wave propagation. Furthermore, a variety of derived types can be conceived depending on the shapes of the pile structures supporting the curtain walls or the shapes of the slits provided in the curtain walls. **Fig. 3.8.2** shows examples of the cross sections of pile-type breakwaters.

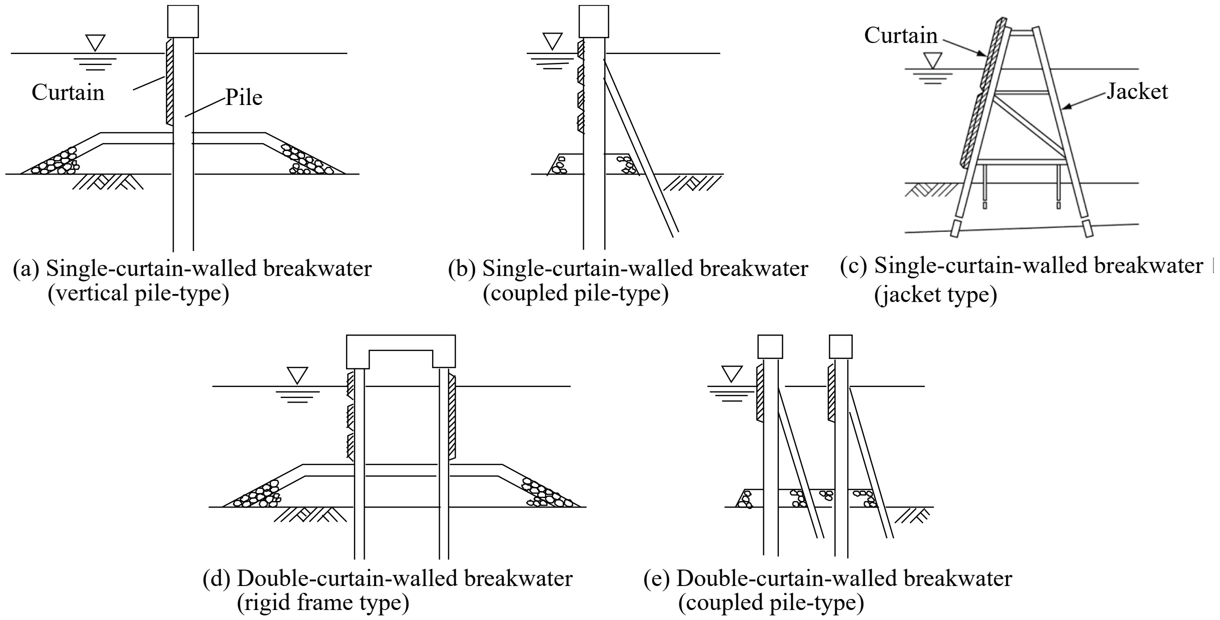


Fig. 3.8.2 Examples of Cross Sections of Pile-Type Breakwaters

(5) Curtain wall breakwaters generally have the following features:

① The reflection coefficients can be reduced to the level equal to or less than those of breakwaters covered with wave-dissipating blocks.

② The exchange of sea water can be expected by tides and waves passing through the slits provided in the curtain walls or the gaps between the lower edges of the curtain walls and the seabed.

③ The construction work such as pile driving, curtain fixing bracket installation, and curtain installation work, needs to be implemented with a certain level of accuracy.

④ Given the expected wave energy attenuation effect between front and back curtain walls, double-curtain-wall breakwaters can reduce reflected and transmitted waves more effectively than single-curtain-wall breakwater.

⑤ Given that the velocities of flows passing under the curtain walls are quite high, it is necessary to take appropriate countermeasures to prevent or suppress the washing out of sand.

(6) Steel pipe breakwaters are breakwaters that use steel pipe piles or steel pipe sheet piles. Steel pipe breakwaters have structures that are lighter than gravity-type breakwaters. Therefore, steel pipe breakwaters are suitable for locations with soft ground and relatively low wave heights.

(7) The performance verification of steel pipe breakwaters can be performed with reference to the concepts of curtain wall breakwaters.

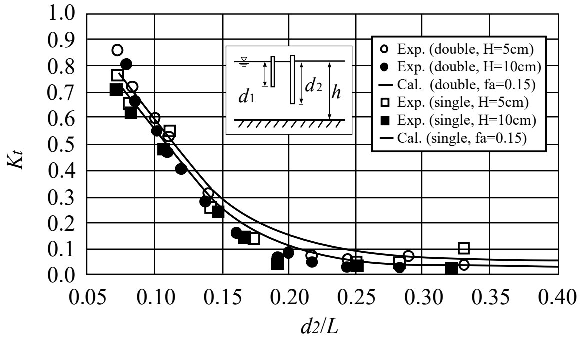
3.8.2 Setting of the Basic Cross Sections

(1) The structural types and shapes of curtain wall breakwaters shall be determined by considering the hydrographic conditions of object water areas, the target reflection and transmission coefficients, and workability.

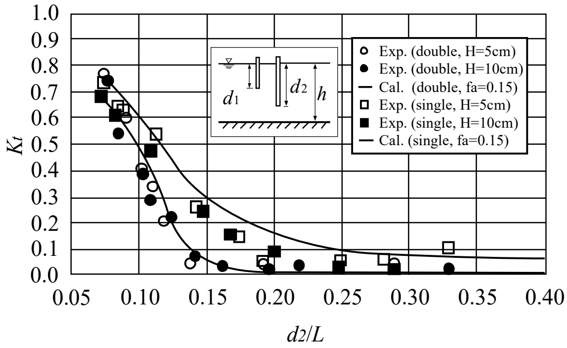
(2) The cross sections of curtain wall breakwaters, including the crown heights, depths of the lower ends of curtains, sizes of the slits provided in the curtains, and spacing between curtain walls in the cases of double-curtain-wall breakwaters, are preferably set on the basis of model tests that simulate actual conditions. It is preferable that the dimensions of members such as curtains and piles be determined appropriately by considering the spacing between the piles in the face line directions of breakwaters.

(3) Studies have been conducted on the obtainment of the reflection and transmission coefficients of curtain wall breakwaters by using model tests or numerical analyses. For example, as shown in **Fig. 3.8.3**, Nakamura et al.[41] conducted a model test to observe the reflection and transmission coefficients of a single-curtain-wall breakwater and a double-curtain-wall breakwater with the curtain spacing similar to the width of gravity-type upright

breakwaters by changing drafts and showed that the observation results of the reflection and transmission coefficients agreed well with the wave attenuation theory. Furthermore, Kyono et al.[42] conducted model tests and a numerical analysis based on the Volume of Fluid Method to deliberate the wave pressure distribution and wave force characteristics with respect to curtain wall breakwaters.

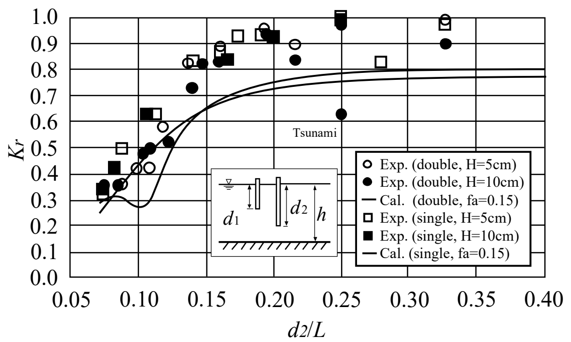


d_1/d_2=0.218, d_2/h=0.353

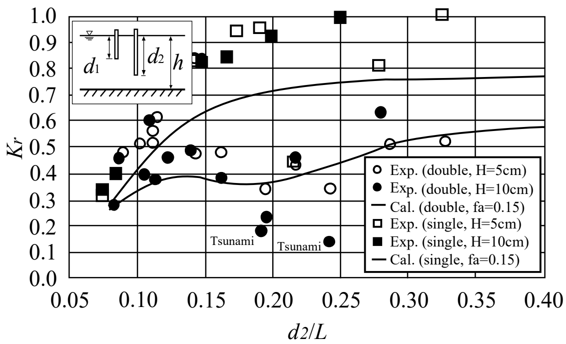


d_1/d_2=1.000, d_2/h=0.353

(a) Comparison of transmission coefficients between double-curtain-wall and single-curtain-wall breakwaters



d_1/d_2=0.218, d_2/h=0.353



d_1/d_2=1.000, d_2/h=0.353

(b) Comparison of reflection coefficients between double-curtain-wall and single-curtain-wall breakwaters

Fig. 3.8.3 Reflection and Transmission Coefficients of Curtain Wall Breakwaters

(4) Examples of model tests for single-curtain-wall breakwaters include the model tests by Morihira et al.[43] According to the model test result, the depths of the lower ends and the crown heights of curtain walls can be obtained in relation to the wave height transmission coefficients by using **Fig. 3.8.4** and **Fig. 3.8.5**, respectively, provided that the crown heights of the curtain walls in **Fig. 3.8.5** are corrected so that R/H = 1.25 at d/h = 1.0, that is, the crown heights are not those that are capable of completely preventing overtopping waves. In the figure, d is the depth of the lower end of a curtain, h is the water depth, L is the wave length, R is the crown height of a curtain, and H is the wave height. **Fig. 3.8.6** shows the relationship between wave reflection coefficients and d/h in single-curtain-wall breakwaters.

(5) With steel pipe piles driven at intervals, steel pipe breakwaters can be used as permeable-type breakwaters. Hayashi et al.[44] studied the relationship between the ratios of pile intervals to pile diameters b/D and wave transmission coefficients γ_T (**Fig. 3.8.7**).

Furthermore, the moment in piles due to waves decreases as the space between piles increases. However, this moment reduction effect reaches its limits at approximately d/D = 0.1. Note that the breakwaters of this type undergo the scouring of the ground between piles.

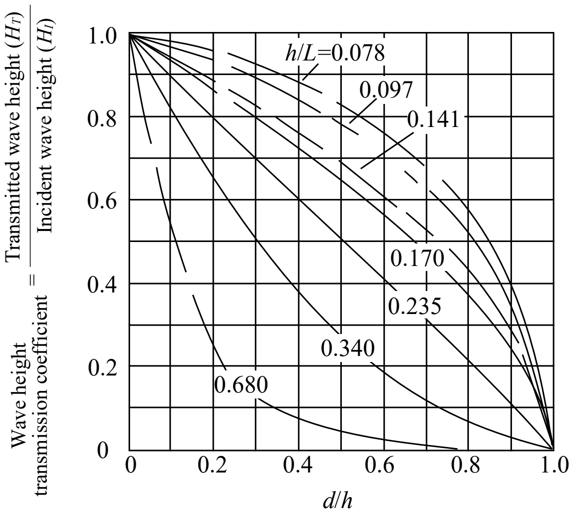


Fig. 3.8.4 Relationship between *d/h* and Wave Transmission Coefficients (Single Curtain Wall)

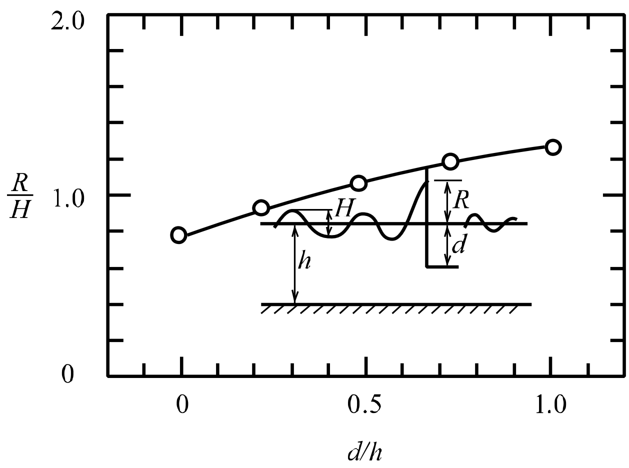


Fig. 3.8.5 Crown Height Calculation Curve (Single Curtain Wall)

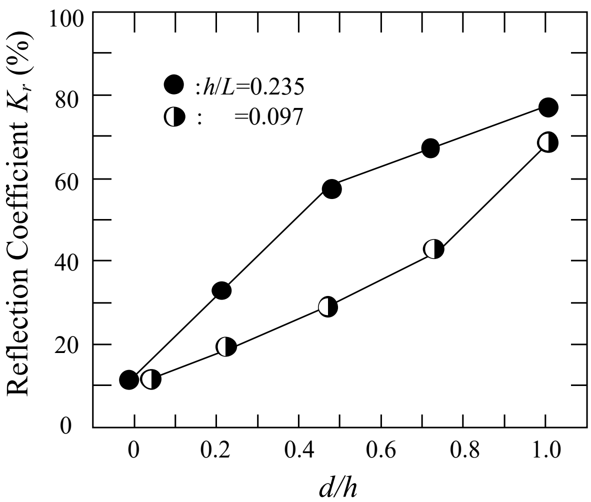


Fig. 3.8.6 Relationship between *d/h* and Wave Reflection Coefficients (Single Curtain Wall)

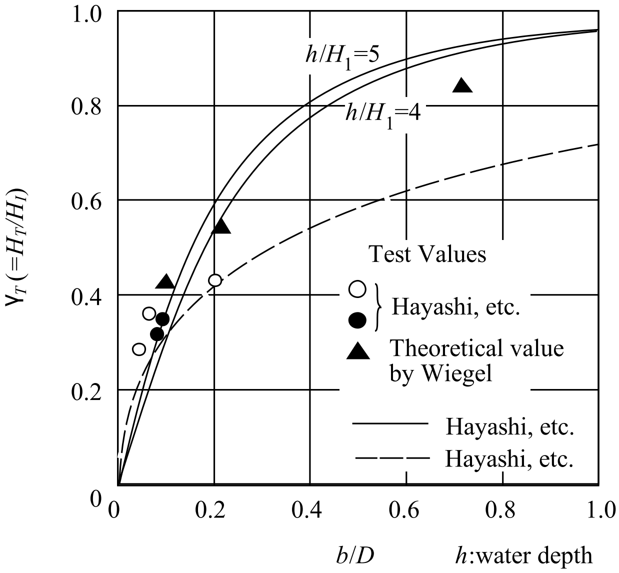


Fig. 3.8.7 Relationship between the Ratio of Pile Interval to Pile Diameter and Wave Transmission Coefficient[43)]

3.8.3 Actions

(1) The actions to be considered in the performance verification of pile-type breakwaters can be set in conformity with composite breakwaters with reference to **Part II, Chapter 4, 3.1.3 Actions**. However, because the reflection and transmission coefficients of pile-type breakwaters vary depending on wave steepness, the deliberation of the reflection and transmission coefficients can be based on the types of waves that have relatively high frequencies and possible risks of interference with the use of ports in general.

(2) Considering that pile-type breakwaters have structures that dissipate waves with slits in curtain sections or the chambers between front and rear curtain sections, the wave force acting on these curtain sections vary depending on their shapes and the characteristics of incoming waves.

(3) The wave force acting on curtain wall breakwaters shall be set on the basis of the results of hydraulic model tests, numerical analyses, or appropriate calculation equations. In the case of single curtain walls, the wave force acting on them can be calculated by subtracting the wave pressure distributed in the sections below the lower edges of curtains from the wave pressure distribution shown in **Part II, Chapter 2, 6 Wave Force**.

(4) Studies on wave force acting on steel pipe breakwaters include the studies conducted by Hayashi et al.[44)] and Nagai et al.[45)] According to their test results, it is considered that **Part II, Chapter 2, 6.3 Wave Force Acting on Submerged Members and Independent Structures** can be used as a reference when setting the wave force acting on steel pipe breakwaters. However, it is preferable not to use steel pipe breakwaters in water areas that are subjected to breaking waves.

(5) Although the performance verification of pile-type breakwaters with respect to Level 1 earthquake ground motions can be performed in a manner that directly calculates the deformation amounts via detailed analysis methods such as dynamic analyses and evaluates whether the required performance is ensured, simplified methods, such as the seismic coefficient method, can also be used for performance verification. When using simplified methods, the characteristic values of the seismic coefficients to be used in the performance verification shall be appropriately set by taking into consideration the structural characteristics of pile-type breakwaters. Depending on the structural types, the characteristic values of the seismic coefficients for pile-type breakwaters can be calculated in conformity with **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** for convenience.

3.8.4 Performance Verification

(1) For the verification of the stresses in steel pipe piles of pile-type breakwaters, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles**. The adjustment factors to be used in the performance verification shall be appropriately set with reference to the allowable stresses on the basis of past design methods.

(2) In the performance verification of stresses in piles, flexural moment and shear force can be calculated on the basis of hinged head piles in the case of pile structures comprising vertical piles or rigid frames having fixed head piles

with virtual fixed points $1/\beta$ below seafloor surfaces in the case of pile structures comprising a group of piles or front and rear piles with respective pile heads rigidly connected through superstructures. For the calculation of β, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles**.

(3) Respective piles shall have sufficient embedded lengths that ensure lateral resistance when subjected to wave actions and bearing capacity against pushing and pulling force. The embedded lengths of pile-type breakwaters can be calculated with reference to **Part III, Chapter 2, 3.4 Pile Foundations**.

3.8.5 Performance Verification of Structural Members

(1) For the performance verification of concrete curtain members, refer to **Part III, Chapter 2, 2 Structural Members**.

(2) For the performance verification of the superstructures of breakwaters, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles**.

3.8.6 Structural Details

The structural details of pile-type breakwaters can be set in conformity with composite breakwaters with reference to **Chapter 4, 3.1.9 Structural Details** or the structural details of other similar structural types.

3.9 Breakwaters with Wide Footings on Soft Ground

[Interpretation]

10. Protective Facilities of Harbors

(4) Performance Criteria of Pile-Type Breakwaters (Item 2, Paragraph 1, Article 14 of the Ministerial Ordinance and the Interpretation related to Article 36 of the Public Notice)

③ Because breakwaters with wide footings on soft ground with pile foundations are a structural type which has the respective structural features of the gravity-type breakwater and the pile-type breakwater, the performance criteria for breakwaters with wide footings on soft ground are equivalent to the respective settings in the Public Notice, Article 35 Performance Criteria for Gravity-type Breakwaters and Article 36 Performance Criteria for Pile-type Breakwaters.

3.9.1 Fundamentals of Performance Verification

(1) Breakwaters with wide footings on soft ground (hereafter, soft landing breakwaters) resist against the horizontal wave force by the piles and the cohesion between the bottoms of the breakwater bodies and the surface layers of cohesive soil. On the other hand, the bottom slabs and footings resist against the vertical force. In general, because this type of structure is developed for construction of breakwaters on soft cohesive soil, there are cases where this type is economically advantageous because the weights of the breakwater bodies can be reduced and soil improvement is not required.

(2) **Fig. 3.9.1** shows examples of cross sections of soft landing breakwaters. Although structural types can be broadly divided into the "flat base type" and the "flat base type with piles," the flat base type with piles is generally used.

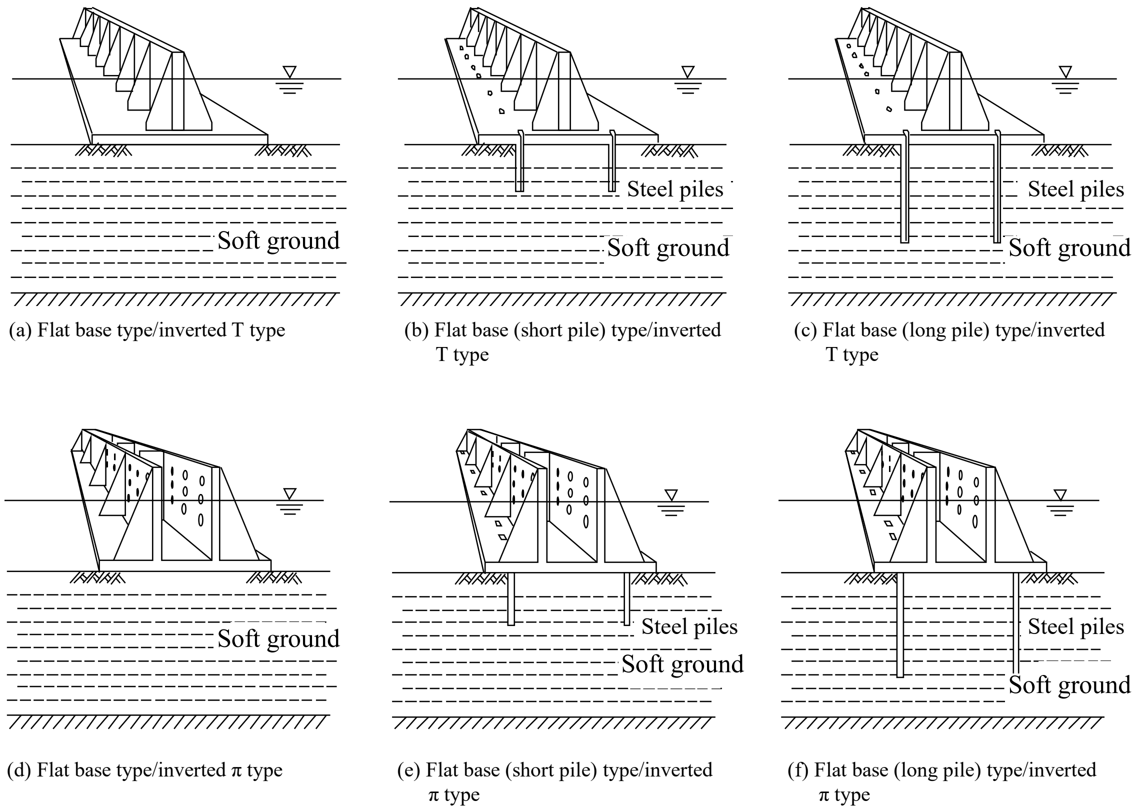


Fig. 3.9.1 Examples of Cross Sections of Wide Footing Breakwaters on Soft Ground

(3) Wide footing breakwaters on soft ground cannot achieve the minimization of both pile dimensions and the widths of breakwater bodies. Therefore, appropriate cross sections shall be selected after the comparative studies of the combinations of pile dimensions and the widths of breakwater bodies.

(4) Constructed directly on soft ground, wide footing breakwaters on soft ground are affected by scouring by waves and water currents in the areas around the breakwater bodies. Therefore, appropriate countermeasures shall be taken, as necessary.

3.9.2 Actions

(1) Actions shall be appropriately set considering natural, use and construction conditions as well as water quality environment. For the types of actions to be considered in the performance verification, reference can be made to **Part III, Chapter 4, 3.1.3 Actions** in accordance with the type of composite breakwaters.

(2) While the performance verification of this type of breakwaters regarding Level 1 earthquake ground motions can be carried out through can be carried out through detailed analysis methods such as dynamic analyses which directly calculate deformation amounts and evaluate performance, simplified methods such as the seismic coefficient method can also be used for the performance verification. When using the simplified methods, the characteristic values of the seismic coefficients to be used in the performance verification can be calculated by the following **equation (3.9.1)** using the maximum values of the time history of acceleration at the bottoms of breakwater bodies obtained through the one-dimensional earthquake response analyses with Level 1 earthquake ground motions in engineering bedrock as input earthquake ground motions. For the calculation of the time history of acceleration of the bottoms of breakwater bodies, reference can be made to **Reference (Part III), Chapter 1, 1.2.2, Group 2 Procedures for the calculation of seismic coefficients for verification (2)** and **(3)**.

$$k_h = \frac{\alpha_{\max}}{g} \quad \textbf{(3.9.1)}$$

where

k_h : a seismic coefficient for verification;

$\alpha_{\max}$: the maximum value of acceleration at the bottom of a breakwater body (cm/s^2); and

g : gravitational acceleration (cm/s^2)

3.9.3 Performance Verification

The performance verification of soft landing breakwaters can be carried out with reference to **References 46)** to **49)**.

3.10 Floating Breakwaters

[Public Notice] (Performance Criteria of Floating Breakwaters)

Article 37

The performance criteria of floating breakwaters under the variable situation, in which the dominating action is variable waves, shall be as prescribed respectively in the following items:

(1) The risk of capsizing of the floating body shall be equal to or less than the threshold level.

(2) The risk of impairing the integrity of the members of the floating body shall be equal to or less than the threshold level.

(3) The risk that the stress generated in mooring ropes may exceed the yield stress shall be equal to or less than the threshold level.

(4) The risk of losing the stability due to tractive forces acting on mooring anchors shall be equal to or less than the threshold level.

[Interpretation]

10. Protective Facilities for Harbors

(5) Performance Criteria of Floating Breakwaters (Article 14, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 37 of the Public Notice)

① Serviceability shall be the performance requirement of floating breakwaters under the variable situation in which the dominating action is variable waves. The performance verification items and standard indexes to determine the limit values regarding the actions shall be as shown in **Attached Table 10-6**.

Attached Table 10-6 Performance Verification Items and Standard Indexes to Determine Limit Values of Floating Breakwaters

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non–dominating action		
14	1	2	37	1	1	Serviceability	Variable	Variable waves	Self-weight, wind, water pressure, water currents	Capsizing of floating body	-
					2					Integrity of members	-
					3					Yielding of mooring ropes	Design yield stress
					4					Stability of mooring anchors, etc.	Resistance force of mooring anchors, etc. (horizontal, vertical)

② In **Attached Table 10-6**, the standard indexes to determine limit values shall be appropriately set for the performance verification of the capsizing of floating bodies and the integrity of members.

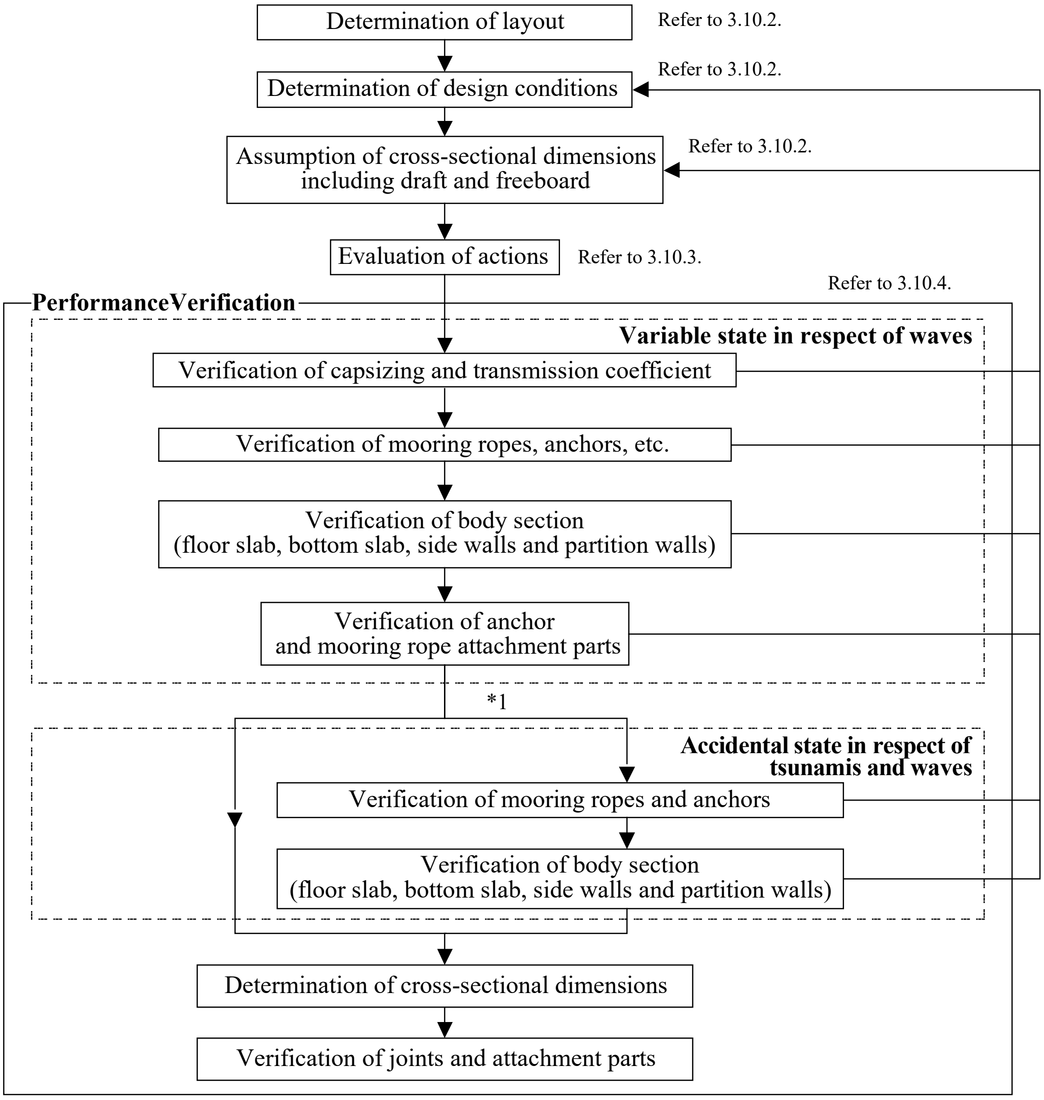
③ Mooring anchor is a collective term for equipment placed on the surface of the sea bottom to retain the floating body. Concretely, in addition to the mooring anchors, sinkers are also included.

3.10.1 Fundamentals of Performance Verification

(1) Floating breakwaters are breakwaters in which transmitted waves are reduced by moored floating bodies. Although the shapes of the floating bodies include many types, the pontoon type is widely used.

(2) It is preferable to carry out the performance verification of floating breakwaters on the basis of hydraulic model tests and theoretical analyses, as needed, with their structures appropriately selected in consideration of wave dissipating effects and stability.

(3) **Fig. 3.10.1** shows an example of the performance verification procedure for floating breakwaters. However, because **Fig. 3.10.1** does not show the evaluation of the effects of liquefaction due to earthquake ground motions on mooring anchors, it is necessary to appropriately deliberate the possibility of and the measures against liquefaction with reference to **Part II, Chapter 7 Liquefaction of Ground**.



*1: For facilities where damage to the facilities can be assumed to have a serious impact on life, property, and social activity, it is preferable to conduct verification for accidental situations when necessary. Verification for accidental situation in respect of waves shall be conducted in cases where facilities handling hazardous cargoes are located directly behind the breakwater and damage to the objective facilities would have a catastrophic impact.

Fig. 3.10.1 Example of Performance Verification Procedure for Floating Breakwaters

(4) The floating breakwaters have various advantages, including the fact that they do not prevent movement of sea water and littoral drift, they are not affected by tidal level changes or ground conditions, and they are moveable. However, they also have some problems in that they generate transmitted waves, their effects differ remarkably depending on the characteristics of waves, they can only be used in locations with small waves due to their limited wave resistance, and the mechanism of the anchor systems' resisting against repeated impulsive actions has not been understood fully. Furthermore, because there is a danger of secondary damage due to drifting of floating

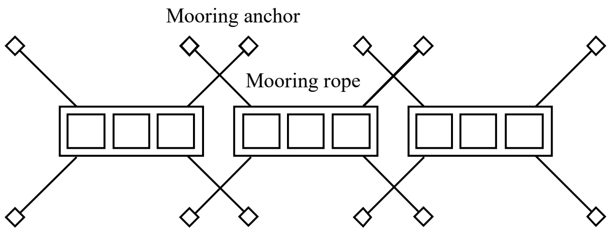
bodies if the mooring ropes break, appropriate measures should be taken including the arrangement of mooring ropes with due considerations to safety.

(5) When determining the quality of materials used for the structures of floating breakwaters, due consideration must be paid to the characteristics, durability, and economic performance of the materials.

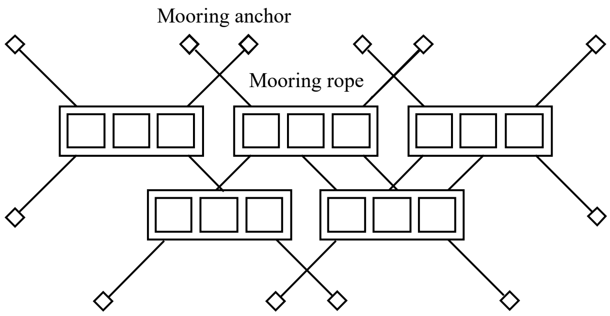
3.10.2 Setting of Basic Cross Sections

(1) The layouts and shapes of floating breakwaters shall be set to ensure predetermined harbor calmness. When setting the layouts and shapes, it is preferable to measure transmission coefficients of floating breakwaters through hydraulic model tests. The theoretical analysis methods which can be used as references include the approximate computation method for the motions of two-dimensional rectangular floating bodies by Ito et al.[50] and the theory pertaining to unmoored free floating bodies by Ijima.[51]

(2) The shapes of floating breakwaters range widely and the materials used for them include reinforced concrete, prestressed concrete, steel, etc. The layout patterns of floating breakwaters are largely divided into serial and alternating parallel patterns.[52] **Fig. 3.10.2** shows examples of the layouts of floating breakwaters. Floating breakwaters are generally moored with mooring anchors made of steel or concrete and mooring ropes consisting of chains or synthetic fiber ropes.



(a) Serial pattern



(b) Alternating parallel pattern

Fig. 3.10.2 Examples of the Layouts of Floating Breakwaters

(3) Mooring methods shall be selected with due consideration given to the actions (waves, currents, etc.) on floating bodies, water depths, tidal levels, seafloor topography, sea bottom soil property, and the lengths of mooring ropes.

3.10.3 Actions

For the actions on floating breakwaters, reference shall be made to **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions**.

3.10.4 Performance Verification

(1) The performance verification of floating breakwaters shall be carried out with reference to **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions** and **Part III, Chapter 5, 6 Floating Piers**. Also, reference can be made to the **Manual for Design and Construction of Floating Breakwaters (Draft)**.[53]

(2) The performance verification of the stability of floating bodies shall be carried out with reference to **Part III, Chapter 5, 6.4 Performance Verification** in compliance with the provisions for floating piers, however, that floating breakwaters are not subjected to the performance verification of the stability in accordance with their use conditions. Also, **Reference 54)** can be used as a reference for the alternative idea of studying the stability of floating bodies when inundated.

(3) The performance verification shall also be carried out for the stability of floating bodies when towed using counter ballast during construction.

(4) The performance verification of the mooring systems comprising mooring ropes and anchors can be carried out in the following manner: obtain tensile and other forces acting on mooring ropes and anchors through static and/or dynamic analyses by assuming various conditions concerning mooring systems such as mooring methods, the lengths of mooring ropes, etc. and confirm the stability of the mooring systems by carrying out the performance verification using the obtained tensile and other forces.

(5) The dynamic analyses of mooring ropes are generally to obtain variable tensions and variable displacements generated by the motions of floating bodies and largely divided into two types: one using static mooring characteristics[55] and the other using dynamic response characteristics of mooring ropes.[56]

(6) The performance verification of mooring anchors can be carried out in compliance with the provisions for floating piers in **Part III, Chapter 5, 6.4 Performance Verification** and with **Reference 57)**.

(7) The structures of the floating bodies of floating breakwaters shall sufficiently ensure overall safety and local strength. For those structures like floating breakwaters which have substantially large lengths compared to the widths and depths, it is generally preferable to examine the following points (see **Fig. 3.10.3)**.

Longitudinal strength:
Sectional force (longitudinal flexural moment, shear force and torsional moment) on entire floating bodies on static water or when subjected to wave actions

Lateral strength:
Sectional force (flexural moment and shear force) in the directions perpendicular to face lines on entire floating bodies when subjected to wave actions

Local strength:
Sectional force (flexural moment and shear force) on respective members such as wall and beam materials

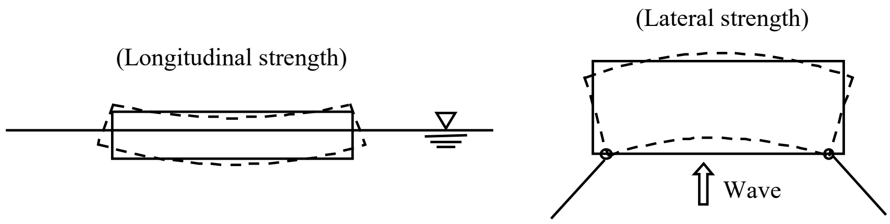


Fig. 3.10.3 Images of the Longitudinal and Lateral Strength of Floating Structure

(8) There are two types of longitudinal strength calculation methods depending on whether or not the motions of floating bodies are considered. Muller's equation is one of the popular methods which does not consider the motions of floating bodies. In contrast, Ueda's formula[58] is an example of a method which considers the motions of floating bodies. **Reference 58)** introduces the comparisons of the calculation results of the two types of methods which can be used as a reference when selecting the methods.

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4 Green Breakwaters

Green breakwaters shall comply with the requirements of **Part III, Chapter4, 3 Breakwaters with Basic Functions** and may be checked for performance verification as follows:

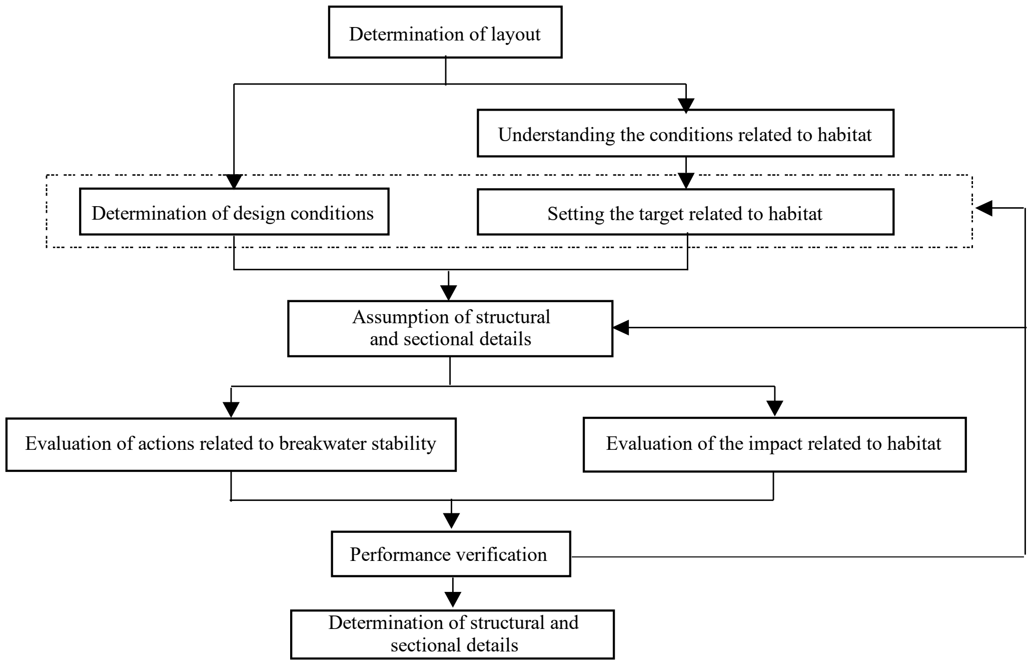
(1) Breakwaters contributing to the development of a good environment in port include green breakwaters[1] that are intended to allow inhabitation of marine organisms in tidal flats or shore reefs depending on the natural environment where the facilities are located (**Reference (Part I), Chapter 3, 2 Green Port Structures**). Existing breakwaters may be remodeled into green ones by attaching the habitat function during their improvement.

(2) The influence on the targets related to inhabitation of marine organisms (**Reference (Part I), Chapter 3, 2 Green Port Structures**) should be determined via environmental surveys and numerical models. The performance verification of the green breakwater is conducted by checking whether the structure and cross section or the ancillary facility is logically expected to achieve the target.

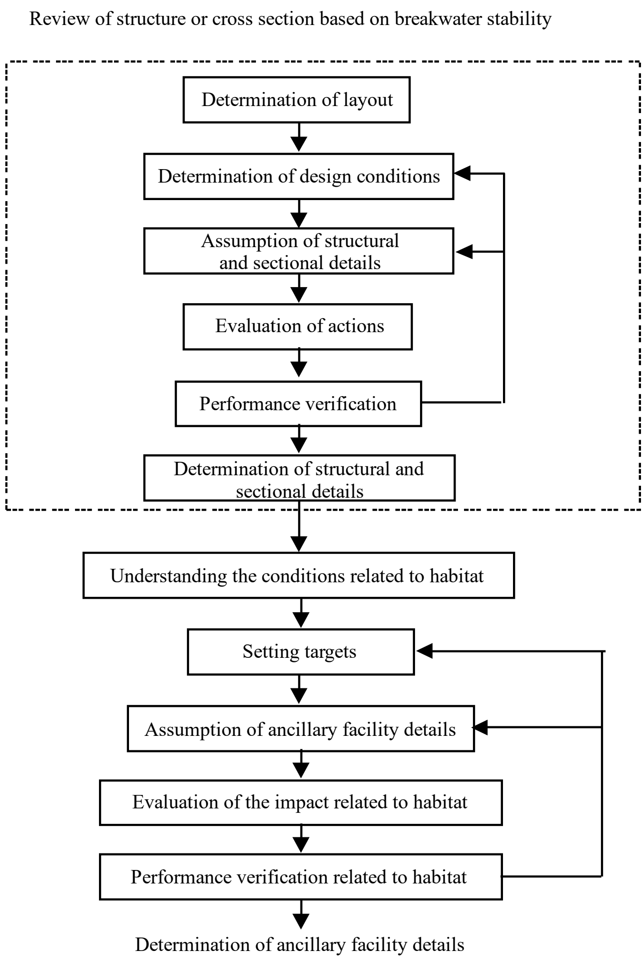
(3) Performance requirement of a green breakwater is that the breakwater should have a habitat function, and its impacts include the presence or absence of the ground related to the inhabitation of marine organisms, external forces such as waves or currents, and the environment necessary for the inhabitation of organisms. The environment necessary for the inhabitation of marine organisms includes, for example, the water depth and water transparency that affects the light intensity necessary for photosynthesis and the water temperature that affects marine organism activity. In particular, when aiming at the cultivation of a seaweed bed, the structure and cross section of a breakwater and the texture and gradient of the ancillary facility need to allow rooting of the target seaweed or seagrass. In addition, it is necessary that the sunlight shielding by them does not affect the light intensity required for the growth of seaweed or seagrass.

(4) Performance verification of a green breakwater should be conducted by ensuring based on the known finding that the environment of a location where one intends to allow marine organisms to inhabit is within the inhabitable range of the target marine organism. For example, during the performance verification of a breakwater used to grow a seaweed bed, the light intensity and water temperature that affect photosynthesis and respiration are assumed to be the environment that should be considered; such verification should be conducted by ensuring that these environmental conditions are within the range wherein a seaweed bed can be established. The location wherein changes in the environmental conditions after a green breakwater is constructed or its future environmental changes can be estimated, a verification method that uses a numerical model related to growth to ensure the environment is within the inhabitable range of marine organisms may be considered.

(5) The procedure for reviewing a green breakwater varies depending on whether the habitat function is provided to the structure and cross section of the breakwater or to its ancillary facility. An example of the review procedure is shown in **Fig. 4.1.1**.



(a) When the habitat function is provided to the structure and cross section of the breakwater



(b) When the habitat function is provided to the ancillary facility

Fig. 4.1.1 Example of a Review Procedure of Green Breakwater

(6) During the performance verification of a green breakwater, **Reference (Part I), Chapter 3, 2 Green Port Structures** and the **Guideline for Development and Maintenance of Green Port Structures**[1] may be used as reference.

(7) It may be feasible to develop breakwaters that can consider the environment with these synergetic effects by adding the amenity function to the symbiotic function.

[Reference]

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5 Amenity-oriented Breakwaters

Amenity-oriented breakwaters shall comply with **Part III, Chapter 4, 3 Breakwaters Having Basic Functions** with necessary modifications made in consideration of the structural type, and their performance verification may be performed as follows:

(1) For the performance verification of amenity-oriented breakwaters, refer to the **Technical Manual for the Improvement of Port Environment**[1].

(2) Breakwaters may be provided with amenity-oriented functions, such as fishing facilities, so that they can be used for multiple purposes.[2]

(3) Amenity-oriented breakwaters shall be equipped with fall prevention fences and other ancillary facilities as needed to prevent users from falling into the sea.

(4) The crown height of an amenity-oriented breakwater must be examined from the viewpoint of public use and safety considering splash, the extent of wave overtopping, and other factors.[3]

(5) Walkways and slopes of breakwaters shall have widths, pitches, and other dimensions that allow elderly users and physically disabled users, including those in wheelchairs, to move safely.[4] [5] [6]

(6) Amenity-oriented functions can be enhanced by considering the inhabitation of marine organisms (**Reference (Part I), Chapter 3, 2 Green Port Structures**).

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6 Storm Surge Protection Breakwaters

6.1 General

(1) The layout, crown height, and other structural details of a storm surge protection breakwater shall be appropriately set considering its effectiveness in reducing the impact of storm surges.

(2) The stability of a storm surge protection breakwater shall be secured not only against the action of waves but also against storm surges considering a rise in the water level and other characteristics of the port or harbor when it is attacked by storm surges.

(3) Storm surge protection breakwaters shall be designed in accordance with this document and by referring to the **Technical Standards and Commentary for Shore Protection Facilities**[1)], **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors**[2)], **Design Concept for Parapets against Tsunamis (Provisional Edition)**[3)], **Guidelines for Tsunami-Resistant Design of Breakwaters**[4)], **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores**[5)], and **JSCE Design Handbook for Shore Protection Facilities**[6)].

6.2 Setting of Basic Cross Section

(1) The basic cross section of a storm surge protection breakwater shall conform to **Part III, Chapter 4, 3 Breakwaters Having Basic Functions** with modifications made as necessary in consideration of the structural type.

(2) The crown height required for a storm surge protection breakwater shall be determined by giving appropriate consideration to waves and the tidal level in the place where it will be constructed. For waves and the tidal level, refer to **Part II, Chapter 2, 4 Waves** and **Part II, Chapter 2, 3 Tidal Level**, respectively.

6.3 Actions and Performance Verification

(1) For the performance verification of a storm surge protection breakwater, the design high water level shall be set appropriately by considering the largest possible storm surges. For setting the design high water level, refer to **Part II, Chapter 2, 3 Tidal Level** and **Part II, Chapter 2, 4 Waves**.

(2) For the performance verification of a storm surge protection breakwater, the rise in the water level shall be considered due to the entry of storm surges into the port or harbor as well as waves that simultaneously occur with storm surges. For setting design conditions and verifying performance in consideration of the simultaneous occurrence of storm surges and waves, refer to **Part II, Chapter 2, 3 Tidal Level** and **Part II, Chapter 2, 4 Waves**.

6.4 Structural Details

(1) Note that when the foundation of a storm surge protection breakwater has high permeability, water flows through the foundation and this decreases the effectiveness of the breakwater in reducing the rise of the water level behind the breakwater. Storm surge protection breakwaters should be provided with water sealing work as needed.

(2) There are cases wherein water flows through the rubble mound of a breakwater because of a difference in the water level between the inside and outside of the breakwater and results in scouring of the foundation ground. In these cases, small-sized rubble, scouring protection mats, or other materials should be laid as needed. For details regarding seepage flows through rubble mound, refer to **Part II, Chapter 3, 3 Underground Water Level and Seepage**.

[References]

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7 Tsunami Protection Breakwaters

7.1 General

(1) Tsunami protection breakwaters are mainly located at the mouths of bays and are intended to reduce a rise in water level behind them in the event of a tsunami. Tsunami protection breakwaters also work together with posterior facilities, such as revetments and dikes, to protect the lives of people and prevent damage to properties that are present behind the breakwaters. Similar to ordinary breakwaters, tsunami protection breakwaters also serve an important purpose, namely, to maintain the calmness of the harbor behind them against oncoming ocean waves.

As a type of tsunami protection facility, a high dike may be constructed along a coastline to protect the land area behind it. However, this approach might result in the loss of a waterfront area that otherwise could be utilized, for example, as a place for social and economic activities. In some cases, there is an insufficient amount of area available for constructing a large facility like a dike. A tsunami protection breakwater may be constructed instead of a dike to make better use of the land and the water surface behind it, with no concerns about the matters mentioned above.

(2) Tsunami protection breakwaters serve to reduce a rise in the water level behind them in the event of a tsunami. By reducing a rise in the water level behind the breakwaters, facilities such as revetments, dikes, and other posterior facilities do not need to have high crowns. In the event that the port or harbor is struck by a tsunami larger than the design tsunami, the tsunami protection breakwater works together with revetments, dikes, and other facilities as an integrated protection system to reduce the extent and depth of inundation on the coast and provide people a longer time to evacuate.

Tsunami protection breakwaters are capable of not only reducing a rise in water level in the event of a tsunami but also reducing the tsunami flow velocity behind them. Ito et al.[1)] made numerical calculations and proved that the breakwater at the mouth of Ofunato Bay is capable of reducing the flow velocity over the entire bay, except the gap. Furthermore, Goto and Sato[2)] proved that the breakwater at the mouth of Kamaishi Bay is effective for reducing the extent of land inundation and the flow velocity behind the breakwater.

(3) In the event of a tsunami, tsunami protection breakwaters serve to reduce a rise in water level and an increase in flow velocity behind them. However, to achieve the required level of protection, a disaster prevention plan that does not rely only on a tsunami protection breakwater but uses it in combination with seawalls and other facilities located behind it is commonly developed. Therefore, the details of a tsunami protection breakwater, including the type, face line, water depths at its gaps, and widths of the gaps, shall be determined to ensure that the water level behind it will not exceed the water level that was determined in consideration of the crown heights and other characteristics of seawalls and other facilities located on the seacoast behind the breakwater when the breakwater is struck by the design tsunami.

When determining the details of the tsunami protection breakwater, it is important to verify the safety and protection performance of the facilities against the design tsunami and to evaluate the safety and protection performance of the facilities against the largest possible tsunami (postulated tsunami) that can occur in the area of interest.[3)]

(4) A safe and economical type of tsunami protection breakwater should be selected on the basis of a comprehensive consideration of many factors, such as the hydraulic conditions of the design tsunami, the design tide level, the design waves and the like, the conditions of the foundation ground, the availability of materials, the constructability of the breakwater, its influences on the surrounding sea areas and adjacent seacoast, its influences on the ecological system, the landscape and the like, the conditions of use of areas behind the breakwater, and the level of difficulty in its maintenance and repair. Similar to ordinary breakwaters, tsunami protection breakwaters can be mainly classified as composite breakwaters, sloping breakwaters, upright breakwaters, and breakwaters covered with wave-dissipating blocks.

A composite breakwater consists of an upright wall body that is constructed on a rubble mound. It works similar to a sloping breakwater or an upright breakwater when the crown of the rubble mound is at a shallow or deep level relative to the wave height, respectively. A sloping breakwater consists of stones or concrete blocks that are piled in a trapezoidal shape, and its main purpose is to dissipate wave energy by making waves break on the slope. An upright breakwater consists of a wall body that is constructed on the seabed and has a vertical front surface. Its main purpose is to reflect wave energy. A breakwater covered with wave-dissipating blocks consists of an upright or composite breakwater and wave-dissipating blocks piled in front of the breakwater, and its purpose is to enable wave-dissipating blocks to dissipate wave energy and to enable the upright wall to prevent waves from passing through the breakwater.

(5) The presence of a tsunami protection breakwater might change the tsunami propagation characteristics and adversely affect the adjacent seacoast. Therefore, the effects of the breakwater should be considered when determining its layout. The effects of the reflection of ordinary waves on the surrounding sea areas should also be considered. When determining the position to construct a tsunami protection breakwater, it is necessary to avoid the positions of the nodes of the natural frequency of a bay and positions where resonance occurs when the design tsunami acts on the bay. If the tsunami protection breakwater is placed in any of these positions, water resonation will occur behind the breakwater because of the action of a tsunami and will result in a significant change in water level.

(6) Reducing the cross-sectional area of a gap of the breakwater is one of the ways to increase the effectiveness of a tsunami protection breakwater in mitigating the impact of a tsunami. However, the gap must have a width and a water depth that will not interfere with the sailing of ships and must face a direction that allows ships to navigate through it easily. Reducing the cross-sectional area of the gap will increase the effectiveness of obstructing the flow of a tsunami, thus preventing tsunami damage; however, this approach may impede the exchange of inshore and offshore waters at normal times. In view of this disadvantage, measures should be taken to prevent seawater from stagnating behind the breakwater and to avoid water quality deterioration.

(7) Tsunami protection breakwaters are constructed to protect areas that are used for material transport, production, fishery, recreational activities, and other purposes. Therefore, the face line of a tsunami protection breakwater shall be determined to minimize the limitation to the current and future use of the areas.

(8) The structural safety of tsunami protection breakwaters shall be secured against the design tsunami. In most cases, a tsunami is caused by an undersea earthquake, and a significant seismic force acts on a tsunami protection breakwater before it is struck by the tsunami. Therefore, the structural safety of facilities against earthquakes should be secured.

Tsunami waves in offshore deepwater areas are smaller than those on the coast, i.e., high waves in stormy weather may exert a larger force on a breakwater than the force of a tsunami. Therefore, the structural safety of facilities against high waves should be secured.

7.2 Basic Concept of Tsunami-Resistant Design

7.2.1 Design Tsunami

The tsunami-resistant design of a breakwater shall be developed to achieve a highly durable structure that will not be collapsed by tsunamis to ensure that its required capabilities remain active against the design tsunami and remain active for the longest extent possible even when it is struck by a tsunami larger than the design tsunami.[4)]

(1) Basic concept of the design tsunami

The tsunami-resistant design of a breakwater against the design tsunami shall be developed with consideration to the functions of a port or harbor behind it and the importance of the facility to ensure that the breakwater can secure the calmness of the harbor immediately after a tsunami and can work effectively to mitigate the possible damage from the tsunami, thus allowing for the possibility of over flowing. Therefore, it is basically necessary to set cross-sectional dimensions so that the capabilities of the breakwater will not be impaired by the action of waves and by the action of the design tsunami.

(2) Demonstration of high durability against a tsunami larger than the design tsunami

To ensure that a breakwater can demonstrate the effectiveness of mitigating possible damage to the greatest extent possible even when it is struck by a tsunami larger than the design tsunami, a highly durable structure that may get deformed but will be hardly collapsed by a tsunami larger than the design tsunami should be developed by taking additional measures for the breakwater on the basis of its cross-sectional dimensions set as described above in (1) and in consideration of the importance, cost-effectiveness, and other characteristics of the facility.

To ensure that a breakwater has a highly durable structure, additional measures shall be developed by [1] assuming that the scale of tsunamis will increase in stages and will ultimately exceed the scale of the design tsunami; [2] thoroughly examining the forms of breakwater failures that can occur owing to tsunamis with different scales, such as the scouring of the foundation mound and seabed and the sliding of the upright wall; [3] identifying the structural weaknesses of the breakwater against tsunamis with different scales; and [4] improving the structure to compensate for weaknesses.

When examining the highly durable structure, its effectiveness should be verified by performing hydraulic model tests and numerical analyses.

7.2.2 Design and Performance Verification Procedure

Fig. 7.2.1 shows an example of a procedure for a comprehensive verification of the overall stability of a breakwater when a tsunami and its preceding earthquake ground motions act on the breakwater. This procedure begins with the setting of the initial cross section against actions due to factors other than tsunamis and proceeds to the setting of cross-sectional dimensions in accordance with **Part III, Chapter 4, 7.4 Actions of Design Tsunami** to ensure that the overall stability of the breakwater will not be impaired by the design tsunami.

The procedure ends with the setting of the cross section of a highly durable structure by making a comprehensive judgment on the basis of the importance, cost-effectiveness, and other characteristics of the facility in accordance with **Part III, Chapter 4, 7.5 Examination of a Highly Durable Structure against Tsunamis Larger than the Design Tsunami**.

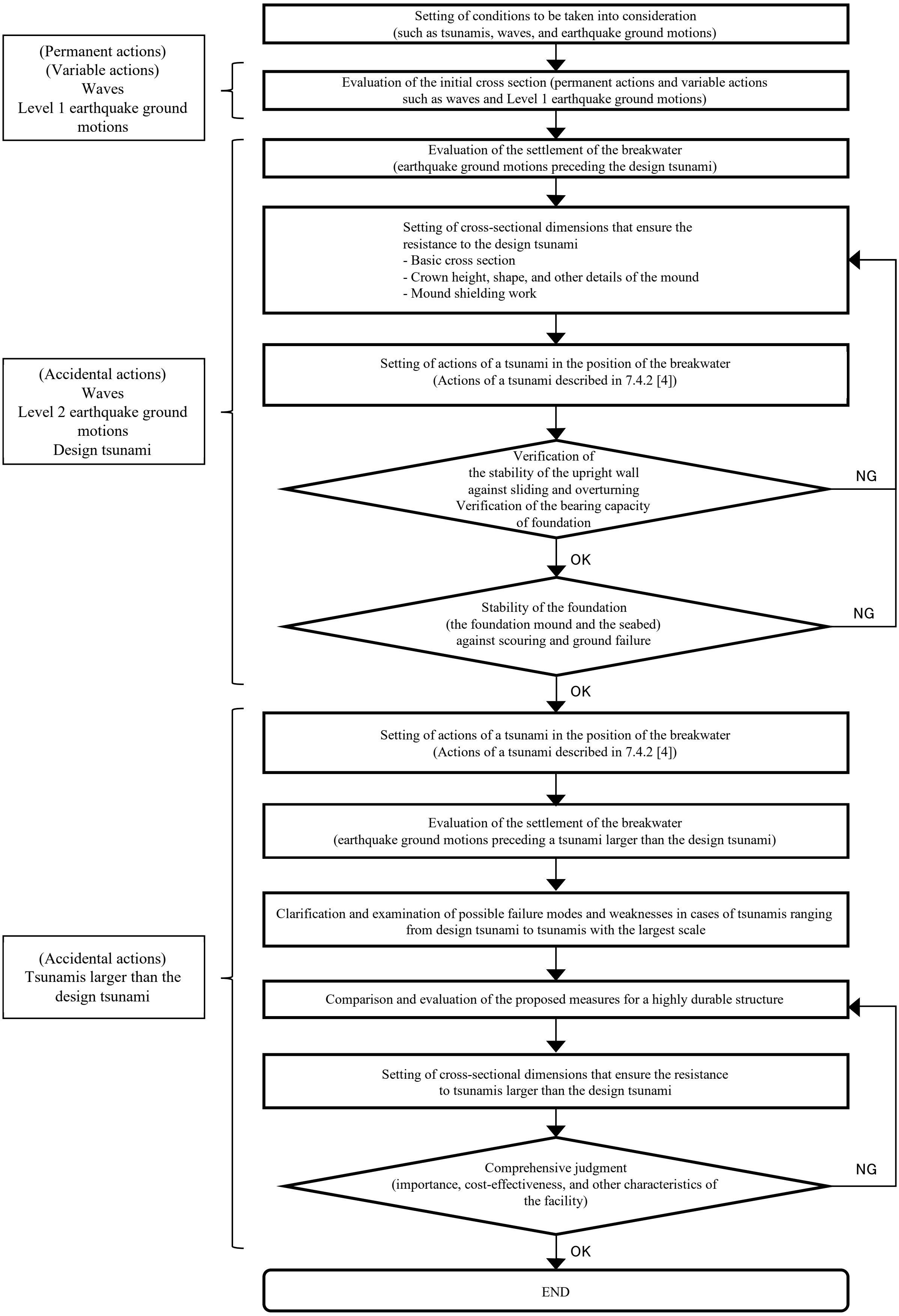


Fig. 7.2.1 Example of the Procedure for the Comprehensive Verification of the Overall Stability of a Breakwater

7.3 Setting of the Basic Cross Section and Required Considerations

7.3.1 Setting of the Basic Cross Section

(1) The crown height of a tsunami protection breakwater should be set to the height necessary for preventing wave overflowing when the design tsunami acts on the breakwater with the tide level set appropriately.

(2) Tsunami protection breakwaters shall conform to **Part III, Chapter 4, 3 Breakwaters Having Basic Functions** with necessary modifications in consideration of the structural type.

7.3.2 Considerations Required in Setting the Basic Cross Section

(1) In the performance verification of a breakwater, its overall stability against possible tsunamis and their preceding earthquake ground motions shall be verified comprehensively on the basis of the forms of breakwater failures caused by tsunamis and by giving due consideration to the various characteristics of the port or harbor, including its geographical features and facility layout.

(2) Factors that contributed to damage to breakwaters that were struck by tsunamis triggered by the earthquake off the Pacific coast of Tohoku[5)]

In the 2011 earthquake off the Pacific coast of Tohoku, a large tsunami struck the Pacific coast of eastern Japan and caused serious damage to ports and harbors on the Pacific coast, including breakwaters. The affected breakwaters included the north breakwater at the Port of Hachinohe, the breakwater at the mouth of Kamaishi Bay, the breakwater at the mouth of Ofunato Bay, and the offshore breakwater of the Port of Soma. According to the analysis results of the damage, the main factors that contributed to breakwater damage include the sliding of upright walls caused by the tsunami wave force, the loss of bearing capacity of their rubble mounds and the seabed inside the ports due to tsunami overflowing and resultant scouring, or the combinations of these actions. The results also indicated that the foundation may have become unstable owing to seepage flow through the foundation mound.

(3) Identification of failure factors and limiting conditions

In the performance verification of a breakwater, the weaknesses that can constitute the factors of failures should be identified on the basis of the forms of possible breakwater failures caused by tsunamis, propose two or more countermeasures, and compare and examine the countermeasures by taking into account the geographic features of the port or harbor, the layout of facilities, and other conditions.

For example, if a waterway (sea route) runs immediately behind the breakwater, this situation imposes a limitation to the area available for constructing countermeasure work inside the port or the harbor. Furthermore, the head of the breakwater is exposed to a high risk of scouring because of concentrated flow through the gap. In the event that the head of the breakwater is displaced, the resultant scouring will cause the successive displacement of adjacent caissons. Therefore, a highly durable structure for the breakwater should be developed, particularly for the head of the breakwater.

It is necessary to study the results of the latest researches and technology developments, including those achieved by private companies; identify the effective countermeasures proposed to cope with failure factors and weaknesses; and set the optimum cross-sectional dimensions on the basis of a comprehensive judgment.

For the identification of effective measures, refer to **Table 7.3.1**.

Table 7.3.1 Main Failure Factors of Breakwaters and Effective Countermeasures

Row: Countermeasure work Column: Failure factor	Countermeasure [1]* Increasing the weight	Countermeasure [2]* Increasing the resistance	Countermeasure [3]* Changing the structure	Countermeasure [4]* Supporting the breakwater from behind	Countermeasure [5]* Covering the mound	Countermeasure [6]* Controlling over flowing water (for example, diverting the flow)
Tsunami wave force	Changing the shape of the superstructure and/or adjusting the specific gravity of filling materials	Laying friction enhancement mats	Changing the shape of the main body**	Additional rubble mound	—	—
Scouring due to over flowing	—	—	—	Levee widening work or the like	Placing armor or foot protection blocks or the like	Changing the shape of the superstructure
Ground seepage flow	—	—	Changing the shape of the main body** Reducing the water permeability of the mound core	Levee widening or the like	—	—

*: It should be noted that the numbers in brackets do not show the order of priority among the countermeasures.

**: It must also be noted that the effectiveness of changing the shape of the main body varies depending on the structure. For example, for a breakwater with an embedded structure, changing the shape of its main body is effective for preventing seepage flow in the ground.

7.3.3 Setting of Cross-Sectional Dimensions

For the tsunami-resistant design of a breakwater, appropriate cross-sectional dimensions shall be determined to ensure that the overall stability of the breakwater will not be impaired by actions of the design tsunami and its preceding earthquake ground motions. The main cross-sectional dimensions that should be determined in the tsunami-resistant design of breakwaters are listed below:

Rubble mound : mound shape, crown height and width, and rubble specifications (weight)

Upright wall : caisson width, crown height, and crown shape (such as parapet)

Foot protection : specifications of foot protection work, including the quantity and layout of foot protection blocks

Armor units : specifications of armor units, including the layout and other details of armor stones and armor blocks

Wave-dissipating work : specifications of wave-dissipating work

Additional rubble mound : height, width, and shape of additional rubble mound and specifications of armor units for additional rubble mound

Scouring prevention work for seabed : specifications of scouring prevention work, including the layout of scouring prevention mats and the like

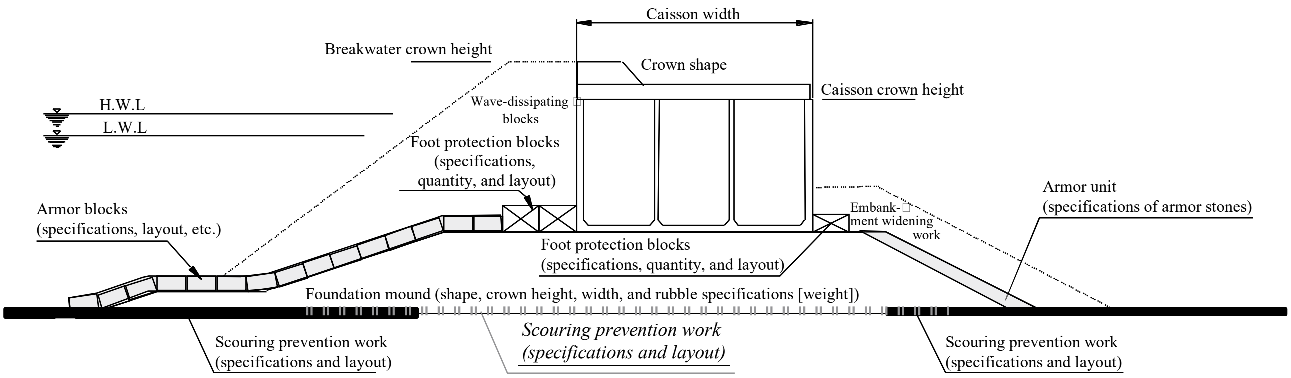


Fig. 7.3.1 Example of the Setting of Cross-Sectional Dimensions

7.4 Actions of Design Tsunami

7.4.1 Setting of Earthquake Ground Motions Preceding a Tsunami and the Evaluation of Their Effects

(1) Setting of earthquake ground motions preceding a tsunami

In the performance verification of a breakwater, earthquake ground motions preceding a tsunami should be appropriately set, and their effects should be appropriately evaluated.

① **Setting of earthquake ground motions preceding the design tsunami**

Earthquake ground motions preceding the design tsunami shall be set by making a model of faults in relation to an earthquake that triggers the design tsunami and by setting a time-history waveform on the engineering bedrock surface by using the technique described in **Part II, Chapter 6, 1.3 Level 2 Earthquake Ground Motions Used in Performance Verification of Facilities**.

② **Setting of earthquake ground motions preceding a tsunami larger than the design tsunami**

In some cases, it is possible to set earthquake ground motions preceding a tsunami larger than the design tsunami by using the same technique as that described above in ①. However, for convenience, appropriate earthquake ground motions can be set by taking into consideration the following earthquake ground motions:

(a) Level 2 earthquake ground motions set in the seismic design (only when subduction zone earthquakes are of interest)

(b) Other earthquake ground motions (including those based on a model of faults that take into account the strong motion pulse generation area with a non-exceedance probability of 50% or more)

(2) Setting of crustal movements preceding a tsunami

Crustal movements shall be set on the basis of an appropriate evaluation that takes into account the characteristics of earthquakes of interest and the amounts of crustal movements caused by past earthquakes.

(3) Evaluation of effects on the overall stability of breakwaters

The foundation ground immediately below a breakwater may become soft owing to liquefaction or other ground failure caused by earthquake ground motions preceding a tsunami, and this situation may result in the settlement of the breakwater. Crustal movements may cause the settlement of a breakwater. When a breakwater settles down, it becomes more likely to be overtopped by tsunami waves, thus increasing the possibility that the foundation mound or the seabed behind the breakwater might be scoured by tsunami waves and increasing the risk that the overall stability of the breakwater might deteriorate. Furthermore, the buoyancy and wave forces acting on the breakwater might become larger and cause a decrease in the stability of the upright wall against sliding and other failures.

Therefore, in the verification of the overall stability of a breakwater, it is necessary to appropriately evaluate the effects of the settlement of the breakwater due to earthquake ground motions and crustal movements preceding a tsunami.

The settlement of the breakwater due to the liquefaction of the seabed shall be estimated by allowing for the settlement associated with shearing deformation and the volume compression of the foundation mound and the seabed and by taking into consideration past cases of breakwater settlement and research studies. Furthermore, the preliminary inclusion of the margin for settlement in the crown height of a breakwater shall be determined as needed. However, it must be noted that increasing the crown height of the breakwater will result in increases in the wave force and other effects of variable waves on the breakwater.

7.4.2 Points to Remember regarding the Actions of Tsunamis

(1) For tsunamis, refer to **Part II, Chapter 2, 5 Tsunamis**.

(2) In the verification of performance of a breakwater against tsunamis, the difference in the water level between the inside and outside of the breakwater should be appropriately evaluated when it is exposed to the action of a tsunami on the basis of a numerical simulation. It should be noted that the water level behind the breakwater will not always be equivalent to the still water level owing to the incoming and outgoing waves of the tsunami.

(3) For the calculation of tsunami wave force, refer to **Part II, Chapter 2, 6 Wave Forces**. However, because there are many points that still need to be clarified, the wave force should be confirmed by using an appropriate method such as a hydraulic model test. Furthermore, it is necessary to consider a decrease in the bearing capacity of the foundation mound and the scouring of the sandy ground below it because of seepage flow.

(4) Points to remember about the actions of tsunamis

① **Various characteristics that affect actions of tsunamis**

Damage to a breakwater due to the actions of tsunamis varies significantly in terms of the affected area, the severity and form of damage, the characteristics of tsunamis of interest, and the characteristics of the port or harbor. Therefore, it is necessary to appropriately set the wave force, flow velocity, and other characteristics of actions of tsunamis that should be considered in the performance verification of the breakwater by giving due consideration to the results of numerical analyses (tsunami simulations), hydraulic model tests, and other similar methods.

- Characteristics of tsunamis: heights, flow velocities, directions, period characteristics, time-variation characteristics, durations, and other characteristics of tsunamis of interest
- Characteristics of the sea area: geographic features, water depth, facility layout (such as positions and widths of gaps), crown height (possibility of over flowing) of the breakwater, and other characteristics

② **Consideration of the time-variation characteristics of the actions of tsunamis**

A tsunami has a long duration and generates landward and seaward motions of waves repeatedly. Furthermore, the direction at which a tsunami wave hits a breakwater and the period characteristics of the tsunami vary significantly with time. In the verification of the stability of the breakwater against the tsunami wave force and in the verification of countermeasures against the scouring of the rubble mound and the seabed, it is necessary to verify the stability of the whole breakwater and to give due consideration to the maximum tsunami height and maximum flow velocity, as well as the duration and time-variation characteristics.

(5) Performance verification utilizing hydraulic model tests and numerical analyses

At present, sufficient knowledge is not always available about the methods for verifying performance against tsunamis. When conducting the performance verification of a breakwater, its overall stability should be confirmed by fully using hydraulic model tests and numerical analyses.

7.4.3 Performance Verification of the Stability of Breakwater Body

(1) When verifying the stability of a breakwater against the wave force of the design tsunami in terms of the stability of the upright wall against sliding and overturning and the bearing capacity of the rubble mound, it is necessary to appropriately evaluate the tsunami wave force acting on the upright wall and the effects of tsunami flow on the rubble mound and seabed.

(2) Setting of the tsunami wave force acting on the upright wall

When a tsunami strikes a breakwater, water pressures act on the following surfaces of the upright wall: the front and rear surfaces hit by the landward and seaward motions of waves respectively; the bottom surface; and the crown surface (as shown in **Fig. 7.4.1**). When verifying the stability of the upright wall, it is necessary to appropriately evaluate the magnitudes of these water pressures and their distribution characteristics and to set the tsunami wave force acting on the upright wall.

When a tsunami strikes and overtops a breakwater, its wave force acting on the breakwater varies depending on the shape of the parapet and other factors. Therefore, the tsunami wave force should be appropriately evaluated on the basis of the results of hydraulic model tests, numerical analyses, and other similar methods.

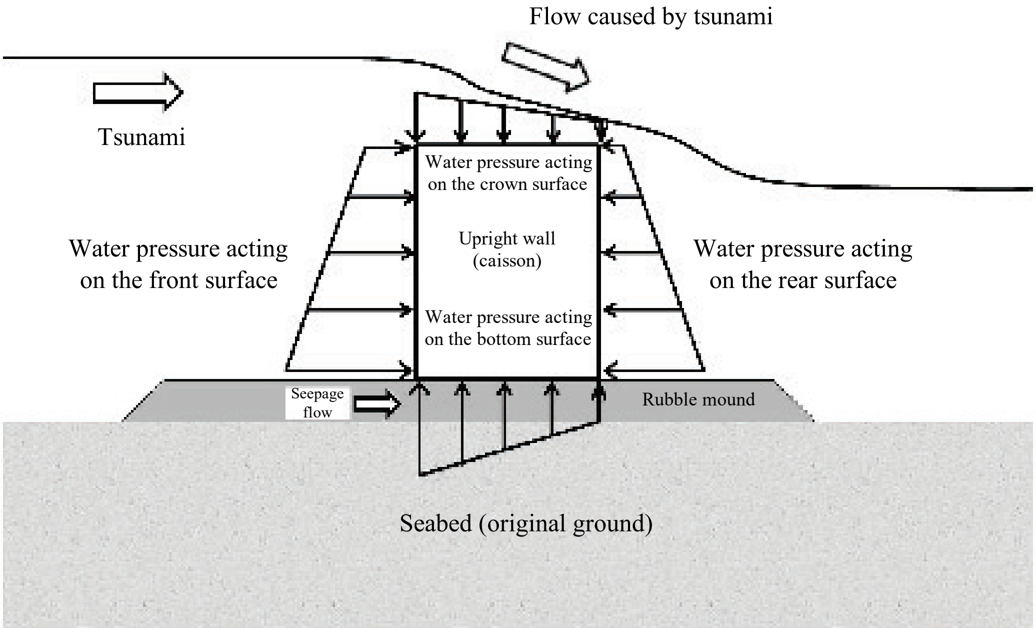


Fig. 7.4.1 Water Pressures Caused by a Tsunami and Acting on the Front and Rear Surfaces, the Crown Surface, and the Bottom Surface of an Upright Wall

(3) Stability of a breakwater against the tsunami wave force in terms of the stability of the vertical wall against sliding and overturning and the bearing capacity of the rubble mound

In general, the stability of a breakwater against the tsunami wave force should be evaluated in terms of the stability of the upright wall against sliding and overturning and the bearing capacity of the rubble mound in accordance with **Part III, Chapter 4, 3 Breakwaters Having Basic Functions**.

When applying the equations given below to the verification of the stability of a breakwater that is being designed, the characteristic values should be set by appropriately evaluating the tsunami wave force acting on the breakwater with consideration to various characteristics affecting the actions of a tsunami. It is also necessary to appropriately evaluate the effects of the seepage flow through the rubble mound because it decreases the bearing capacity of the rubble mound. Furthermore, the characteristic values should be set by thoroughly examining other factors that affect the stability of the upright wall, including the softening of the ground due to the liquefaction of the seabed or other ground failure, and by appropriately evaluating the friction coefficient and the shear strength of ground materials.

Equations (7.4.1), **(7.4.2)**, and **(7.4.3)** may be used to verify the stability against sliding, the stability against overturning, and the bearing capacity of the rubble ground, respectively. In the following equations, subscripts *k* and *d* indicate the characteristic value and the design value, respectively. The numerical values given in **Tables 7.4.1**, **7.4.2**, and **7.4.3** may be used for the partial factors in the equations. The "—" symbol shown in each table indicates that the numerical value in parentheses may be used to simplify verification calculations.

① **Verification of stability against sliding**

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$R_k = f_k (W_k - P_{B_k} - P_{U_k}) \tag{7.4.1}$$

$$S_k = P_{H\,k}$$

where

f : friction coefficient between caisson and rubble mound;

W : weight of the breakwater body (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

P_H : Horizontal wave force of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : resistance factor;

γ_S : load factor;

m : adjustment factor.

Table 7.4.1 Partial Factors to be Used for the Verification of the Stability of Breakwater Body against Sliding

Object of verification	Partial factor by which the resistance term is multiplied, γ_R	Partial factor by which the load term is multiplied, γ_S	Adjustment factor, m
Sliding of breakwater body (Accidental situation due to design tsunami)	— (1.00)	— (1.00)	1.20

② **Verification of stability against overturning**

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$R_k = \{a_1 W_k - a_2 P_{B_k} - a_3 P_{U_k}\} \qquad (7.4.2)$$
$$S_k = a_4 P_{H_k}$$

where

W : weight of breakwater body (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

P_H : horizontal wave force of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

a_1 to a_4 :arm length of each action (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : resistance factor;

γ_S : load factor;

m : adjustment factor.

Table 7.4.2 Partial Factors to Be Used for the Verification of the Stability of Breakwater Body against Overturning

Object of verification	Partial factor by which the resistance term is multiplied, γ_R	Partial factor by which the load term is multiplied, γ_S	Adjustment factor, m
Overturning of breakwater body (Accidental situation due to design tsunami)	— (1.00)	— (1.00)	1.20

③ **Verification of the bearing capacity of the rubble ground**

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$F_f = \frac{R_k(F_f)}{S_k}$$

$$R_k = \sum \left[\frac{\{c'_k s + (w'_k + q_k)\tan\phi'_k\}\sec\theta}{1 + \tan\theta \tan\phi'_k / F_f} \right] \qquad \textbf{(7.4.3)}$$

$$S_k = \sum\{(w_k' + q_k)\sin\theta\} + \frac{a_1 P_{H_k}}{r}$$

where

P_H : horizontal wave force of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

a_1 : arm length of the horizontal wave force of a tsunami (m);

c' : undrained shear strength for clayey ground or apparent cohesion under drained condition for sandy ground or stone (kN/m^2);

s : width of the slice segment (m);

w' : effective weight of the slice segment (kN/m) (weight in the air for the part above the water surface or weight in water for the part below the water surface);

q : surcharge acting on slice segment (kN/m);

ϕ' : apparent angle of shear resistance based on effective stress (°);

θ : angle formed by slice segment with bottom (°).

F_f : supplementary parameter showing ratio of design value of resistance and design value of effect of action;

r : radius of slip circle (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : resistance factor;

γ_S : load factor;

m : adjustment factor.

Table 7.4.3 Partial Factors to Be Used for the Verification of the Bearing Capacity of Foundation Ground

Object of verification	Partial factor by which the resistance term is multiplied, γ_R	Partial factor by which the load term is multiplied, γ_S	Adjustment factor, m
Failure of bearing capacity of foundation ground (Accidental situation due to design tsunami)	— (1.00)	— (1.00)	1.00

Equation (7.4.3) is the effective stress expression based on the simplified Bishop's method. When **equation (7.4.3)** is used, the supplementary parameter F_f shall be first determined by repeated calculations so that $R_k = F_f \times S_k$ is satisfied. Note that F_f is contained in the equation for R_k. Thereafter, the R_k and S_k obtained from these calculations shall be used for the verification of the stability in terms of bearing capacity.

Considering that the seepage flow in the rubble mound causes a decrease in the bearing capacity of the rubble, the effects of the seepage flow in the verification of bearing capacity should be appropriately evaluated. It was indicated that the bearing capacity may decrease by approximately 10 to 17% at a breakwater of ordinary shape and scale when there is a difference of approximately 9 to 10 m in the water level between the inside and outside of the port or harbor.[6), 7)] However, the effects of the seepage flow have not been fully understood yet; therefore, it is preferable to include a margin of approximately 20% in the design of a breakwater for a port or harbor where the difference in the water level is 10 m.[8)] The rate of decrease in the bearing capacity is proportional to the difference in water level. For example, when the difference in the water level between the inside and outside of the port or harbor is 5 m, the rate of decrease should be considered as 10%, and **equation (7.4.3)** should be satisfied by using the adjustment factor of 1.1. It is also possible to verify the bearing capacity in consideration of the effects of the seepage flow by using a finite element analysis in which the effects of the seepage flow can be considered or by using a full-scale centrifuge model test in which the bearing capacity characteristics can be simulated. When using the finite element method, it is necessary to use the apparent cohesion and the angle of shear resistance that allows an appropriate evaluation of the shear strength of the ground under the confining pressure that decreased due to the effects of seepage flow.[9)]

(4) Examples of damage to breakwaters due to the actions of the tsunami caused by the 2011 earthquake off the Pacific coast of Tohoku

Fig. 7.4.2 shows the results of a research on the frontline breakwaters of ports* that were exposed to the actions of the largest-scale tsunami triggered by the 2011 earthquake off the Pacific coast of Tohoku. To determine whether the breakwaters were significantly damaged by the tsunami**, data were analyzed using two indices: one is the safety factor against the sliding of the upright wall of each breakwater, and the other is the over flowing water depth, which is defined as the water depth from the crown surface of the breakwater to the tsunami water level on the seaward side of the breakwater.

*: Port of Hachinohe, Port of Ofunato, Port of Kamaishi, Port of Soma, Port of Kuji, Port of Sendai Shiogama, and Port of Onahama

**: Breakwaters that slid inside their design construction site are counted as breakwaters that were damaged by the tsunami.

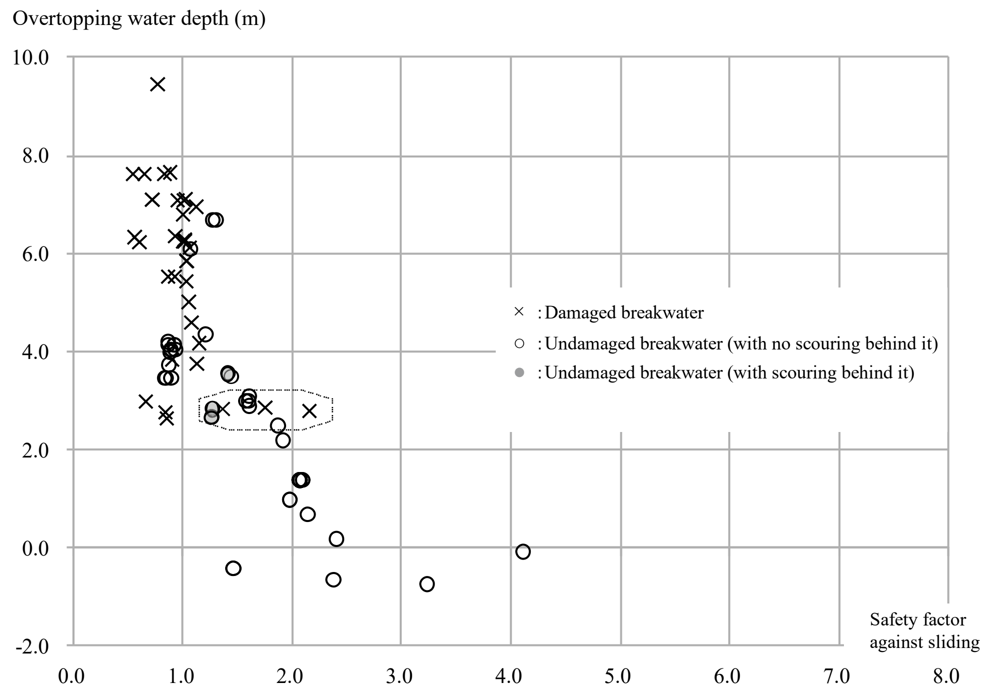


Fig. 7.4.2 (a) Relationship between the Over flowing Water Depth and the Safety Factor against Sliding for Damaged and Undamaged Breakwaters (Case Examples of Damage to Breakwaters Due to the 2011 Earthquake Off the Pacific Coast of Tohoku)

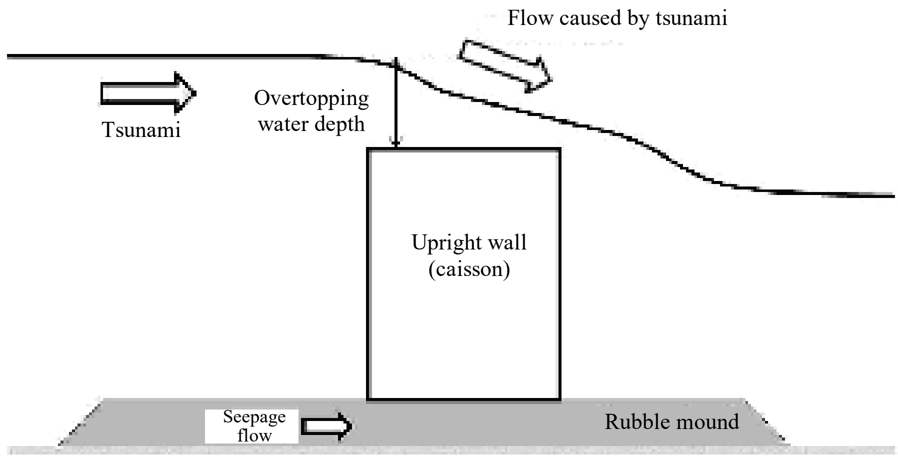


Fig. 7.4.2 (b) Definition of Over flowing Water Depth

This figure indicates that there were many damaged breakwaters in cases where the safety factor against sliding defined by conventional standards was lower than approximately 1.2. It also indicates that breakwaters were damaged from scouring in cases wherein the over flowing water depth exceeded approximately 2 m. Some breakwaters were damaged even in cases wherein the safety factor against sliding was higher than 1.2 (the three × marks enclosed by the dotted-line in the chart indicate these breakwaters). It is considered that these breakwaters were damaged from over flowing and the resultant scouring of the mound or seabed behind them (refer to **Part II, Chapter 4, 7.4.4 Stability of the Rubble Mound and Seabed against Tsunami Flow**). Some other breakwaters showed signs of scouring of the mound and the seabed behind them as a result of over flowing even though there was no displacement of caissons (the gray circles in the chart indicate these breakwaters).

7.4.4 Stability of the Rubble Mound and Seabed against Tsunami Flow

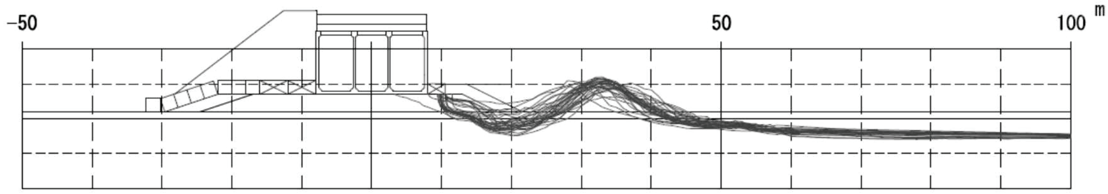
(1) In the evaluation of the stability of the rubble mound and the seabed against the tsunami flow caused by the design tsunami, it is necessary to appropriately evaluate over flowing, seepage flow, and other actions of the tsunami flow on the rubble mound and the seabed and to take appropriate measures to prevent the occurrences of scouring and rubble mound failures that may impair the stability of the upright wall.

(2) Significance of countermeasures against scouring

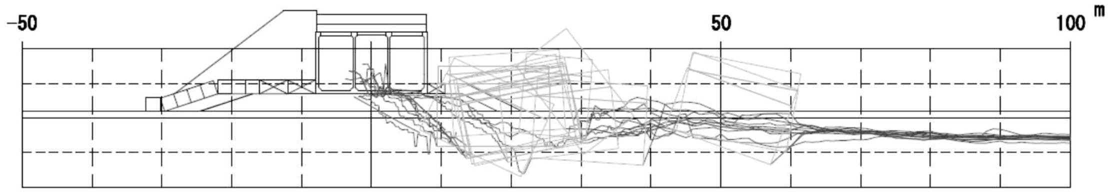
A large-scale tsunami has a large wave force and generates a long-lasting, strong, one-way current. When the rubble mound (including foot protection work and shielding work) of a breakwater and the seabed are scoured owing to the tsunami flow, including over flowing and seepage flow, this might cause the settlement of caissons and a decrease in the bearing capacity of the rubble mound, thus resulting in the collapse of the upright wall.

When a tsunami overtops the breakwater, it becomes more likely that the rubble mound and the seabed behind it will be scoured and results in the collapse of the upright wall. **Fig. 7.4.3** shows a specific case of scouring behind a breakwater. In the 2011 earthquake off the Pacific coast of Tohoku, the largest-scale tsunami struck the north breakwater at the Port of Hachinohe and caused scouring on the rubble mound and the seabed behind it, thus resulting in damage to the breakwater (in the 11th construction site of the breakwater). **Fig. 7.4.3(a)** shows the superposed cross sections along the survey lines for the remaining part of the damaged upright wall. **Fig. 7.4.3(b)** shows the superposed cross sections along the survey line for the slid part of the damaged upright wall in the said construction site.

As seen in these figures, the rubble mound and seabed behind the breakwater were significantly scoured even though the upright wall was not displaced. It can also be seen that scoured soil was deposited on the landward side of the scoured area. Considering that the seabed behind a breakwater is increasingly scoured, the rubble mound and seabed will become unable to bear the upright wall. The upright wall will ultimately fall down on the scoured area and will get damaged.



(a) Remaining part of the upright wall



(b) Slid part of the upright wall

Fig. 7.4.3 State of Damage Due to Over flowing in the Eleventh Construction Site of the North Breakwater at the Port of Hachinohe

The foundation mound and seabed are also likely to be scoured in areas around the head and gaps of a breakwater owing to the strong concentrated flow of a tsunami current. This increases the risk of the collapse of the upright wall of the breakwater. As a specific case of scouring around the head or gap of a breakwater, **Fig. 7.4.4** shows the location where the Port of Hachinohe scouring occurred due to the tsunami caused by the 2011 earthquake off the Pacific coast of Tohoku. Scouring down to a depth of more than 10 m from the original sand-bed was found around the heads and gaps of the breakwaters. Around the central part of the gap between the first central breakwater and the second central breakwater, scouring occurred and resulted in the collapse of caissons at the heads of the breakwaters.

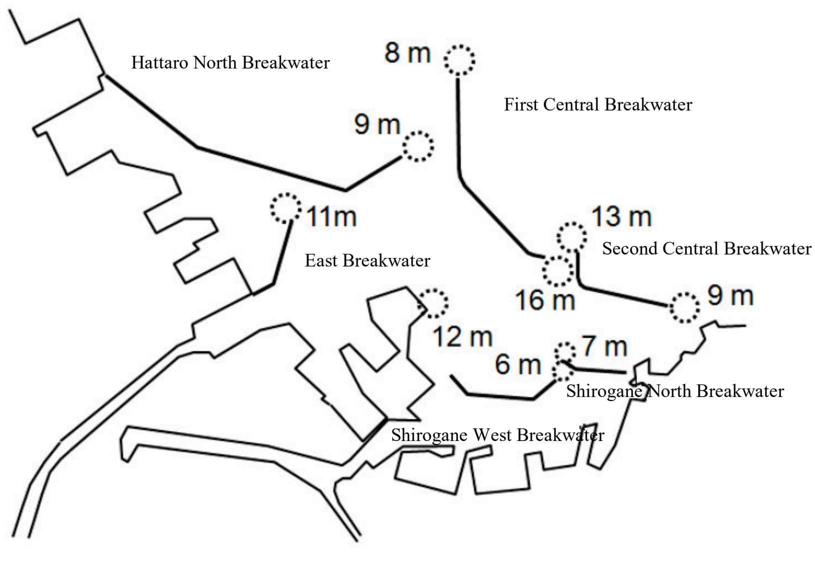


Fig. 7.4.4 Scouring of the Rubble Mounds and the Seabed (Port of Hachinohe)
The dotted-line circles indicate the scoured areas, and the numerical values indicate the depth of scouring.[10)]

In protecting a port or harbor from damage caused by tsunamis, it is more effective in some cases to take small-scale measures to reinforce the main body of a breakwater than to take additional measures, such as countermeasures against scouring. One measure that can be taken in such cases is to increase the width and/or crown height of the upright wall in consideration of the importance and cost-effectiveness of the facility. However, it must be noted that increasing the crown height of a breakwater will cause the breakwater to be subjected to the greater effects of waves.

(3) Points to remember in examining countermeasures against scouring

The tsunami-resistant performance of a breakwater against the design tsunami can be significantly affected by currents flowing at higher velocities in areas around the head and gaps and by over flowing. Therefore, the basic countermeasures against scouring involve ensuring that the rubble mound and seabed will not be scoured by waves and the design tsunami. If it is difficult to completely prevent the scouring of the seabed, measures should be taken to ensure that scouring will not cause the rubble mound to collapse in chain and impair the stability of the upright wall.

With regard to scouring due to over flowing, a situation wherein the tsunami height in front of a breakwater is maximized does not always result in the most severe scouring of the rubble mound and seabed. Therefore, when considering countermeasures against scouring due to over flowing, it is necessary to focus on the water levels and the other time-variation characteristics of tsunamis of interest, which are listed below:

- During the landward motion of waves (when the tsunami flow acts from the outside to the inside of a port or harbor)

  - Range of the tsunami water level on the front side (outside the port or harbor)
  - Range of the still water level on the rear side (inside the port or harbor)
  - Range of the settlement of the breakwater (allowing for rise)
  - Duration of over flowing

- During the seaward motion of waves (when the tsunami flow acts from the inside to the outside of the port or harbor)

  - The same items as those during the landward motion of waves shall be taken into consideration.

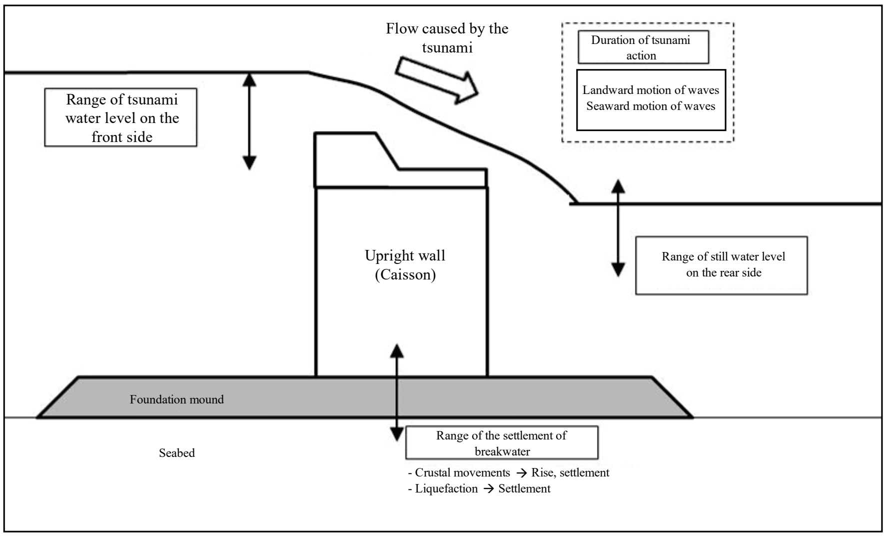


Fig. 7.4.5 Diagram of Conditions to Be Considered in the Examination of Countermeasures against Scouring

Adequate attention must be paid to the durability of armor materials to ensure that shielding work can fulfill its function when a tsunami strikes the port or harbor. For the required weight of shielding work, refer to **Part II, Chapter 2, 6 Wave Forces**. It is possible that the scouring of a rubble mound due to over flowing may be accelerated when the confining pressure of the mound decreases because of the effects of seepage flow.[11), 12)]

(4) Examination of countermeasures against scouring based on hydraulic model tests

Hydraulic model tests and numerical analyses should be used to examine and determine specific measures to protect the rubble mound of a breakwater and seabed from the scouring caused by over flowing or by very swift currents that can occur in areas around the head and gaps of the breakwater.

(5) Examination of piping

When seepage flow occurs in the rubble mound owing to differences in water level between the inside and outside of the port or harbor, piping might occur in the area of the mound that is close to the lowest part on the rear side of a caisson. Therefore, it is necessary to conduct piping verification and to construct the mound without using small stones to mitigate piping.[7)]

7.4.5 Additional rubble mound

Levee widening work can increase the sliding resistance of the upright wall and the bearing capacity of the rubble mound and can also reduce the scouring of the rubble mound and the sandbed behind the upright wall. Therefore, a breakwater with levee widening work is considered to have a structure that will hardly collapse even when it is struck by a large tsunami.

Fig. 7.4.6 shows a schematic diagram of levee widening work. It is possible to increase the stability against over flowing by constructing shielding work, foot protection work, and scouring prevention work. Levee widening work may be examined by using the method described in **Part III, Chapter 4, 3 Breakwaters Having Basic Functions**.

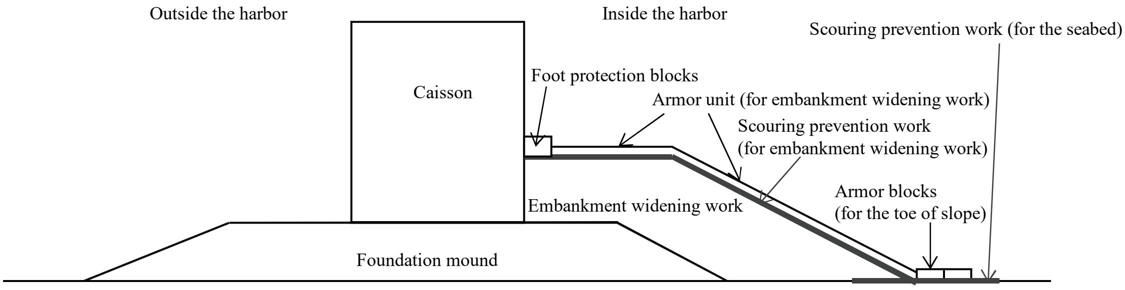


Fig. 7.4.6 Levee Widening Work Combined with Shielding Work, Foot Protection Work, and Scouring Prevention Work

In cases wherein it is difficult to construct extensive countermeasure work for a breakwater (e.g., at the head of the breakwater or in an area behind the breakwater where there is a sea route or anchorage area), it is suggested to use steel pipe piles and concrete block frames to make the cross section of the breakwater smaller than that of a breakwater with ordinary levee widening work on the basis of the tsunami over flowing condition and the flow velocity condition behind the breakwater.[13)]

7.5 Examination of a Highly Durable Structure against Tsunamis Larger than the Design Tsunami

7.5.1 General

When examining the highly durable structure of a breakwater against tsunamis larger than the design tsunami, cross-sectional dimensions that are appropriate for the security objectives of the port or harbor should be set to ensure that the highly durable structure can maintain the overall stability of the breakwater as much as possible even when it is struck by a tsunami larger than the design tsunami. This can be performed by thoroughly examining the forms of failures and structural weaknesses of the breakwater in relation to tsunamis of different scales and by taking additional measures to compensate the weaknesses.

7.5.2 Concept of the Highly Durable Structure of a Breakwater

When examining the resistance of a breakwater against tsunamis larger than the design tsunami, the breakwater should have a highly durable structure that allows it to deform but not to collapse when it is struck by a tsunami larger than the design tsunami, thereby allowing the breakwater to maintain the overall stability of the breakwater as much as possible. This can be accomplished by assuming that there are possible tsunamis with scales that increase in stages and exceed the scale of the design tsunami and that tsunamis work as external forces on the cross section that was set against waves and the design tsunami (refer to **Part III, Chapter 4, 7.4 Actions of Design Tsunami**), by thoroughly examining the forms of failures and structural weaknesses of the breakwater in relation to tsunamis of different scales on the basis of hydraulic model tests and other similar tests, and by taking additional measures to compensate for the weaknesses in consideration of the importance, cost-effectiveness, and other characteristics of the facility.

7.5.3 Examination of Additional Measures against Tsunamis of Scales Increased in Stages

When it is assumed that there are possible tsunamis with scales that increase in stages and exceed the scale of the design tsunami, the extent of over flowing of a breakwater becomes larger in stages, thus increasingly revealing the structural weaknesses of the breakwater and resulting in severe damage to the structure. Therefore, when examining the highly durable structure of the breakwater, it is preferable to assume that there are possible tsunamis with scales that increase in stages and exceed the scale of the design tsunami, to verify the effectiveness of additional measures to compensate structural weaknesses (or means of improving the structure), and to determine the specific cross-sectional dimensions in consideration of the importance, cost-effectiveness, and other characteristics of the facility.

Fig. 7.5.1 shows an example of the examination of measures against tsunamis with scales that increase in stages. The horizontal axis indicates the scales of tsunamis. The vertical axis indicates the cost required for taking additional measures (or means of improving the structure) for a breakwater to ensure that it has a highly durable structure; therefore, it will not collapse when it is struck by a tsunami larger than the design tsunami. The two curves show how the cost increases when measures are taken against tsunamis with scales that increase in stages.

In the figure, Breakwater A represents a breakwater that is designed to serve in an area where it is exposed to gentle waves, and the horizontal wave force generated by the waves and acting on the breakwater is larger than the tsunami wave force generated by the design tsunami but is smaller than the tsunami wave force generated by a tsunami of the largest scale. When it is assumed that there are possible tsunamis with scales that increase in stages and exceed the scale of the design tsunami, the stability of this breakwater against the sliding of the upright wall and other failures will decrease drastically against tsunamis with scales that exceed the stage (marked with a black circle in the figure) where a tsunami is not as large as a tsunami of the largest scale; therefore, a breakwater with a cross section that is designed to resist waves can resist a tsunami. Therefore, it is considered necessary to construct levee widening work behind the breakwater or take other large-scale means of improving the structure to ensure that the breakwater has high durability against tsunamis with scales that exceed the said stage.

By contrast, Breakwater B represents a breakwater that is designed for an area exposed to rough waves and wherein the horizontal wave force generated by the waves and acting on the breakwater is equivalent to the tsunami wave force generated by a tsunami of the largest scale. The stability of this breakwater against the sliding of the upright wall or other failure will not drastically decrease against tsunamis with scales that exceed the stage where a tsunami is not as large as a tsunami of the largest scale. Therefore, it is considered possible to maintain the high durability against tsunamis with scales up to the largest scale by taking small-scale means of improving the structure, that is, by taking countermeasures against scouring caused by over flowing.

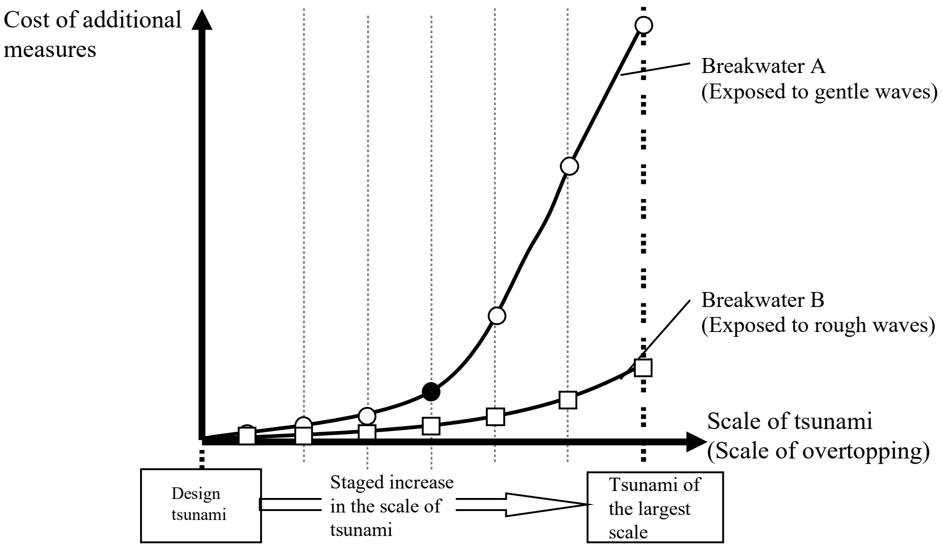


Fig. 7.5.1 Example of the Examination of Additional Measures against Tsunamis with Scales That Increase in Stages

In cases wherein a breakwater is designed to serve in an area where it is exposed to gentle waves, such as Breakwater A, and tsunamis of the largest scale are postulated as tsunamis that can exceed the scale of the design tsunami for the breakwater in consideration of its importance and other characteristics, it will cost a lot of money to construct standard levee widening work or other large-scale work as additional measures for the breakwater. In such a case, it must be noted that introducing a newly developed technology may be more economically efficient than constructing standard levee widening work because the new technology may cost significantly in terms of the initial investment for additional measures against tsunamis that are slightly larger than the design tsunami but may not cost much in terms of the overall investment for additional measures against tsunamis that have the largest scales.

7.5.4 Verification of Effectiveness of the Highly Durable Structure of a Breakwater

To verify the effectiveness of the highly durable structure of a breakwater, it is necessary to appropriately evaluate the actual modes of deformation and verify the stability of the breakwater against deformation. In particular, for the upright wall and the rubble mound, the modes of deformation in various situations should be appropriately evaluated, for example, where levee widening work is constructed, where seepage flow affects deformation, where the structure is reinforced with steel pipe piles or other pipes, and where rubble is covered with friction enhance mats or other mats.

As a convenient and indirect way of evaluating the high durability of a structure, it is possible to examine how much margin for the tsunami wave force is given to the structure in consideration of the passive resistance of levee widening work and other factors by using the equations given for the verification of the stability of the upright wall against sliding or overturning and for the bearing capacity of the rubble mound and calculating the safety factor against sliding as one of the criteria for high durability. If the safety factor against sliding is larger than 1.0, the structure can be considered highly durable. When verifying the high durability in this way, the effectiveness of specific measures should be verified, including countermeasures against scouring, on the basis of hydraulic model tests and numerical analyses.

For the final step, it is necessary to comprehensively evaluate from various points of view the scales of tsunamis that the breakwater can resist without collapsing and while maintaining its highly durable structure. This can be performed by carefully examining the effectiveness of measures by using hydraulic model tests and numerical analyses and by considering the importance and cost-effectiveness of the facility.

7.6 Effectiveness of Tsunami Protection Breakwaters in Mitigating the Impact of Tsunamis and Delaying the Rise in Water Level

The effectiveness of the tsunami protection breakwater in Ofunato Bay in mitigating the impact of a tsunami was evaluated by analyzing the harbor resonance in the bay on the basis of the tide level recorded in the earthquake off the coast of Tokachi (in May 1968). **Fig. 7.6.1** shows the results of the comparison of data measured before and after the construction of the tsunami protection breakwater wherein the wave height amplification ratio M is the ratio of the amplitude in the inner part of the bay to the amplitude of incident waves. For long-period oscillations in low-order modes, the wave height amplification ratio after the construction is significantly lower than that before the construction, thus indicating that the tsunami protection breakwater demonstrated effectiveness in mitigating the impact of the tsunami.[14] The effectiveness of this tsunami protection breakwater was also verified by Ito et al.[1] on the basis of numerical calculations.

In the earthquake off the Pacific coast of Tohoku (in March 2011), the breakwater at the mouth of Kamaishi Bay reduced the tsunami height by 40% and the maximum wave run-up height by 50% and delayed tsunami wave overt flowing of the seawall by 6 minutes, thus demonstrating effectiveness in delaying the rise in water level.[15]

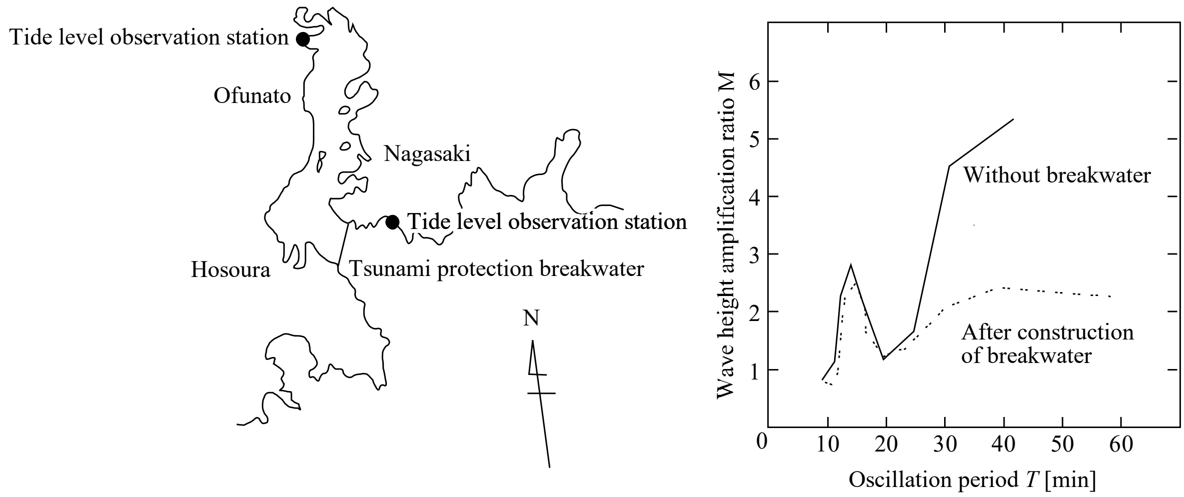


Fig. 7.6.1 Effectiveness of the Tsunami Protection Breakwater in Ofunato Bay

7.7 Others

(1) An experimental study by Tanimoto et al.[16] has verified that in a situation wherein a tsunami flows into a port or harbor through a narrow entrance, the tsunami flows at an increased velocity while generating vortices. This situation significantly affects the stability of the armor material of the mound of a submerged breakwater. Tsunamis also exert strong tractive forces on the bed that are said to be even greater than those by storm surges. Therefore, the upright walls and the foundation ground at the entrance of a port or harbor should be adequately reinforced (e.g., by increasing the stability of the upright walls and preventing scouring of the foundation ground).

(2) Given that the rubble mound becomes thicker as the water becomes deeper, it is necessary to pay careful attention to the stability of the rubble mound against wave forces and to the wave transformation on the slope surface of the rubble mound. It is also necessary to carefully determine dimensions, such as the height of the extra-banking of rubble, with consideration to the increased compression of the mound.

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8 Breakwaters for Timber Handling Facilities

8.1 General

Breakwaters for timber handling facilities shall comply with **Part III, Chapter 4**, **3 Ordinary Breakwaters** with modifications made, as necessary in consideration with the structural type, and their performance verification may be performed as follows:

8.2 Actions

(1) Generally, timber handling facilities, such as timber storage ponds and sorting ponds, are located in the innermost part of a port or harbor; therefore, they are not exposed to high waves. Unlike ordinary breakwaters, breakwaters for such facilities are mainly intended to prevent timbers from drifting out. Therefore, it is advisable to examine not only the force of waves but also the force that occurs when timbers collide with a breakwater under the effect of the wind, tidal current, and/or waves, depending on the situation.

(2) The colliding force of timbers has not been clarified yet. While conducting performance verification, refer to records and data of past cases.

8.3 Setting of Basic Cross Section

(1) The crown height of a breakwater for timber handling facilities should be set to an appropriate height in consideration of the structure of the breakwater, the usage of the water area behind it, and other factors to ensure that timbers do not drift out under conditions such as an abnormal tide level. It is preferable that the crown height is higher than the abnormal tide level by approximately 60% of the design significant wave height.

(2) In most cases, harbor calmness behind a breakwater is controlled to keep the wave height at approximately 50 cm even in rough weather.

8.4 Structural Details

(1) To prevent timbers from drifting out of a sorting pond, it is preferable to construct fences, as needed, to prevent timber drifting or piles to moor timbers.

(2) Fences to prevent timber drifting should have the crown height and pile interval appropriate for preventing timbers from drifting out and should be equipped with a superstructure, as needed.

(3) It is necessary that fences to prevent timber drifting and piles to moor timbers have structures that can resist the colliding or tractive force of timbers caused by winds, tidal currents, and waves. Additionally, it is preferable to consider wave forces and other forces depending on the situation.

9 Sediment Control Groins

[Ministerial Ordinance] (Performance Requirements for Sediment Control Groins)

Article 15

1 The performance requirements for sediment control groins shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied for the purpose of mitigating siltation in waterways and basins due to littoral drift through effective control of littoral drift.

2 The provisions of Article 14, paragraph (1), item (ii) shall apply mutatis mutandis to the performance requirements for sediment control groins.

[Public Notice] (Performance Criteria of Sediment Control Groins)

Article 38

1 The provisions of Article 35 or Article 36 shall apply mutatis mutandis to the performance criteria of sediment control groins in consideration of the structural type.

2 In addition to the provisions of the preceding paragraph, the performance criteria of sediment control groins shall be such that the facilities are located appropriately so as to control littoral drift, in consideration of the environmental conditions, etc. to which the facilities are subjected, and shall have the dimensions necessary for the functions of the sediment control groins.

[Interpretation]

10. Protective Facilities for Harbors

(6) Performance Criteria for Sediment Control Groins (Article 15 of the Ministerial Ordinance and the interpretation related to Article 38 of the Public Notice)

① Performance criteria of gravity-type breakwaters and pile-type breakwaters and their interpretations shall be applied correspondingly to sediment control groins with modifications made as necessary in consideration of the structural type.

② In the performance verification for sediment control groins, appropriate consideration shall be given to the effects of an increase in earth pressure due to sedimentation of littoral drift and the effects of river currents, as needed, in addition to the previous paragraph.

9.1 General

(1) A breakwater in a harbor surrounded by a sand beach also serves as a sediment control groin, and as a result, it is impossible to separate these functions. Therefore, in this section, breakwaters are referred to simply as "breakwaters," except when their sediment control function is especially important.

(2) Layout of Sediment Control Groins

① Sediment control groins shall be appropriately located by considering the characteristics of littoral drift so as to fulfill the expected sediment control function.

② A sediment control groin on the updrift side of longshore sediment transport shall run perpendicularly to the shoreline in shallow water including the surf zone. In deep water beyond the surf zone, the groin shall be located so that it can disperse littoral drift to the side opposite to the harbor entrance.

③ When a sediment control groin is constructed on the downdrift side of longshore sediment transport in order to prevent littoral drift from being carried into the harbor from the shore on the downdrift side, the groin shall run perpendicularly to the shoreline in principle and shall also have an appropriate length considering wave direction and wave transformation. However, in cases where a sediment control groin also functions as a breakwater, it shall be located appropriately considering its required functions as a breakwater.

④ If there is a need to provide a sediment control groin in the vicinity of a waterway inside a harbor, it shall be constructed in an appropriate location in consideration of the environmental conditions.

(3) Layout of Updrift Side Breakwater

It is preferable that the updrift side breakwater is extended perpendicularly to the shoreline beyond the surf zone or across the wave breaking line in order to prevent longshore sediment transport and allow sediment to deposit on the updrift side of the breakwater (refer to **Fig. 9.1.1**). When this part extending from the shore is short or deflected from the line perpendicular to the shoreline toward the downdrift side, the capability of the breakwater to catch sediment on the updrift side is reduced and sediment can easily move along the deflected part of the breakwater toward the harbor entrance. When this part is deflected from the line perpendicular to the shoreline toward the downdrift side, it is likely to become the cause of local scouring on the shore on the updrift side.[1] In deep water beyond the wave breaking line, the breakwater shall be deflected so that it can block waves as a breakwater, and at the same time, disperse littoral drift toward the opposite side of the harbor entrance with the aid of reflected waves or Mach-stem waves (refer to **Fig. 9.1.1**).

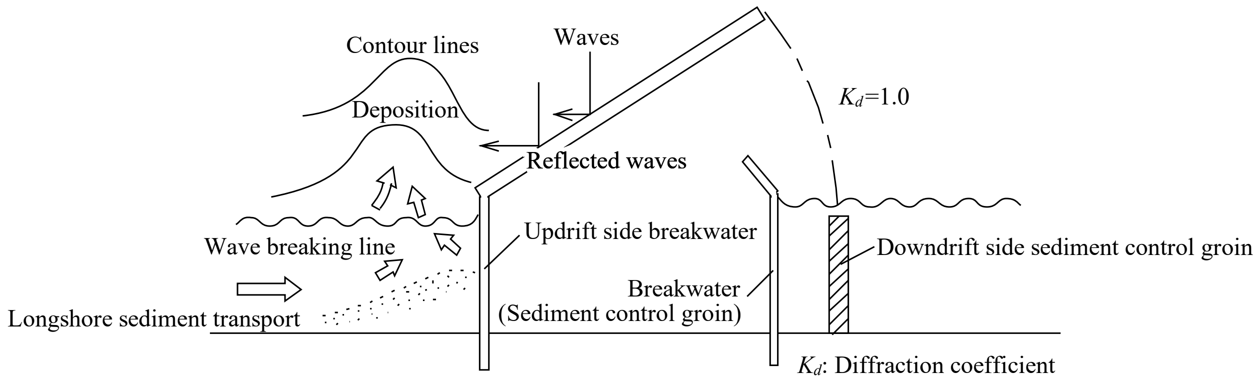
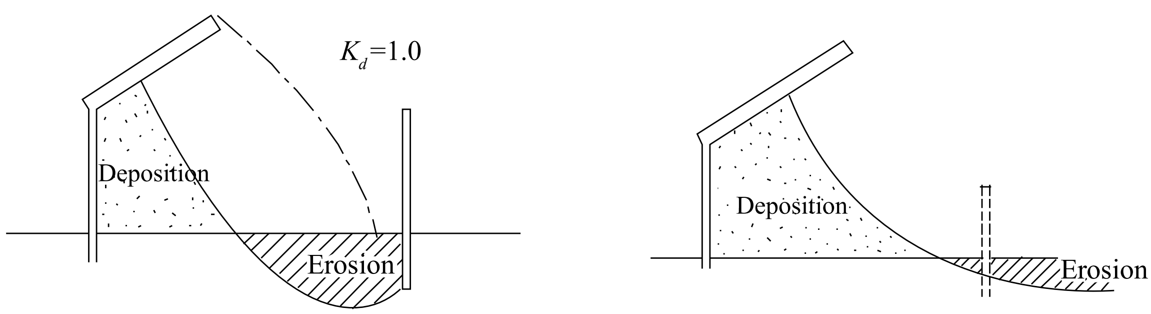


Fig. 9.1.1 Conceptual Diagram of Layout of Breakwater (Sediment Control Groin)

(4) Positioning and Construction Time of Downdrift Side Breakwater

When the updrift side breakwater is extended beyond the extension line of the downdrift side breakwater that runs perpendicular to the shoreline, this will allow sediment to deposit on the downdrift side of the latter breakwater, resulting in formation of a sandbar extending from the shore on the downdrift side toward the harbor entrance and erosion of the shore on the downdrift side.[2] If the downdrift side breakwater is extended before the deflected part of the updrift side breakwater is extended to a sufficient length, significant local erosion may occur on the harbor side of the downdrift side breakwater, as shown in **Fig. 9.1.2 (a)**. Conversely, if the extension work of the downdrift side breakwater is delayed, this may often cause sedimentation in the harbor and erosion of the shore on the downdrift side as shown in **Fig. 9.1.2 (b)**. The closest attention should therefore be paid to the speeds of extending the updrift and downdrift side breakwaters, and care must be taken to maintain the appropriate balance between the lengths of the breakwaters.



(a) Case with rapid extension of the downdrift side breakwater

(b) Case with slow extension of the downdrift side breakwater

Fig. 9.1.2 Construction Time of Downdrift Side Breakwater

(5) Length of the Breakwater and Water Depth at the Tip

Longshore sediment transport occurs mainly in the surf-zone so it is necessary to extend the breakwater to a point offshore beyond the surf zone. In small ports where the tip of the breakwater is within the surf zone during stormy weather, it is difficult to completely prevent littoral drift from entering the port. In major ports in Japan, it is common for the water depth at the tip of an updrift side breakwater to be approximately equal to the maximum depth of the navigation channels in the port concerned.

Fig. 9.1.3 shows case examples of sediment control groins that work effectively as ancillaries. **Fig. 9.1.3 (a)** shows a case in which sediment control groins serve to prevent sand from entering the waterway from both sides. **Fig. 9.1.3 (b)** shows a case in which a sediment control groin (i) serves to increase the capability of blocking littoral drift on the updrift side, and a sediment control groin (ii) serves to allow incoming sediment to deposit on the natural beach on the right side.[3)]

Even if a very long breakwater is constructed, it is hard to completely prevent sediment carried by the water flow along the breakwater from going around the tip of the breakwater and into the harbor. When a basin or waterway is located behind a breakwater, it requires some maintenance dredging. Therefore, it is preferable to determine the most economically efficient length of the breakwater in consideration of such maintenance dredging.

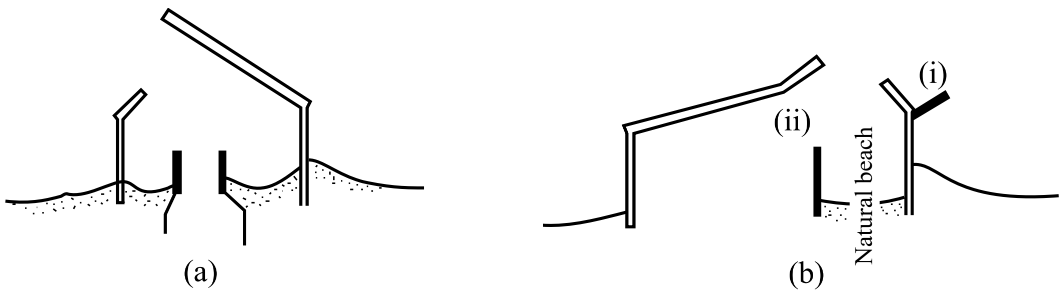


Fig. 9.1.3 Case Examples of Sediment Control Groins Provided as Ancillaries[3)]

(6) Structural Forms of Sediment Control Groins

Sediment control groins should have impermeable structures because they are expected to stop sediment transport completely. Where a rubble-mound or concrete-block structure is adopted to build the landward end of a sediment control groin, it may be filled with quarry run or rubble of up to 100 to 200 kg; there are also cases where sediment infiltration prevention work is constructed, as needed, on the harbor side of the sediment control groin by using impermeable materials such as sand mastic asphalt. In the following situations, it is preferable to additionally adopt a wave-absorbing structure.

① When there is significant concern about scouring by currents

② When there are concerns about reflected waves that can cause siltation or obstruction to the navigation of ships

9.2 Performance Verification

(1) For the performance verification of sediment control groins, refer to provisions concerning composite breakwaters given in **Part III, Chapter 4, 3.1.4 Performance Verification for Overall Stability of Breakwater Body**," as well as provisions concerning the performance verification for each structural type. Note, however, that it is necessary to appropriately consider the effects of an increase in earth pressure due to sedimentation of littoral drift.

(2) Crown Height of Sediment Control Groins

Although it is preferable that sediment control groins not allow waves to overtop them in order to prevent waves from carrying suspended sediment into the harbor, there are also cases where overtopping is allowed due to structural constraints, economic efficiency and other reasons. In principle, the crown height of each part of a sediment control groin should be determined by taking the following into consideration:

① **Part near the landward end of a sediment control groin**

It is preferable that the crown height of a sediment control groin at the part near the landward end be high enough to prevent run-up waves from overtopping it. Because sand carried by run-up waves may overtop the crown of the sediment control groin at the landward end, the crown should be sufficiently high. It is preferable to raise the crown height or extend the groin itself to the landward direction, in light of the conditions after construction.

② **Part on the landward side of the wave breaking line**

The crown height of the sediment control groin at the part on the landward side of the wave breaking line may be set to 0.6 $H_{1/3}$ above the mean monthly-highest water level (HWL), where $H_{1/3}$ is the significant wave height around the tip of the sediment control groin.

③ **Part on the seaward side of the wave breaking line**

The crown height of the sediment control groin at the part on the seaward side of the wave breaking line may be determined by adding a certain margin to the mean monthly-highest water level. In deep water offshore beyond the surf zone, the suspended sediment is concentrated near the seabed and the overtopping water contains only small amount of sediment, and therefore overtopping may be allowed in most cases.

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10 Seawalls

[Ministerial Ordinance] (Performance Requirements for Seawalls)

Article 16

1 The performance requirements for seawalls shall be as prescribed respectively in the following items so as to protect the hinterland of the seawall with modifications made as necessary in consideration of the structural type:

(1) Seawalls shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable the protection of the hinterland of the seawalls from waves and storm surges.

(2) Damage to seawalls, etc. due to self-weight, earth pressure, variable waves, Level 1 earthquake ground motions, etc. shall not impair functions of the seawalls and shall not adversely affect the continueous use of the seawalls.

2 In addition to the provisions of the preceding paragraph, the performance requirements for seawalls in the place where there is a risk of serious impact on human lives, property or socioeconomic activities shall be as prescribed in the following items with modifications made as necessary in consideration of the structural type.

(1) The performance requirements for seawalls which are required to protect the hinterland of the seawalls from design tsunami or accidental waves shall be such that the seawalls satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable the protection of hinterland of the seawalls from design tsunami or accidental waves.

(2) Damage to seawalls, etc. due to design tsunamis, accidental waves, Level 2 earthquake ground motions, etc. shall not have a serious impact on the structural stability of the seawalls, even in cases where functions of the seawalls are impaired. Provided, however, that for the performance requirements for a seawall which requires further improvement of the performances due to the environmental, social conditions, etc. to which the seawall is subjected, the damage due to the actions, etc. shall not adversely affect the restoration through minor repair work of the functions of the seawalls.

3 In addition to the provisions of the preceding two paragraphs, the performance requirements for breakwaters in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity, shall be such that a serious impact on the structural stability of the breakwaters caused by damage, etc. due to the actions of the tsunamis, even in cases where tsunami with intensity exceeding the design tsunami occurs at a place at which the breakwaters are installed, shall be delayed as much as possible.

[Public Notice] (Performance Criteria of Seawalls)

Article 39

1 The provisions concerning the structural stability in Article 49 through Article 52 (excluding the provisions concerning ship berthing and traction by ships) shall apply mutatis mutandis to the performance criteria of seawalls with modifications made as necessary in consideration of the structural type.

2 In addition to the provisions of the preceding paragraph, the performance criteria of seawalls shall be as prescribed respectively in the following items:

(1) Seawalls shall be located appropriately so as to enable the control of wave overtopping in consideration of the environmental conditions, etc. to which the facilities are subjected, and shall have the dimensions necessary for the function of the seawalls.

(2) Under the variable situation, in which the dominating action is water pressure, the risk of losing stability due to seepage failure of the ground shall be equal to or less than the threshold level.

(3) For structures with parapets, the risk of sliding and overturning of the parapet under the variable situation, in which the dominating actions are variable waves and Level 1 earthquake ground motion, shall be equal to or less than the threshold level.

3 In addition to the provisions of the preceding two paragraphs, the performance criteria of seawalls to which damage might significantly affect human lives, property or socioeconomic activity shall be as prescribed respectively in the following items:

(1) Seawalls which are required to protect the hinterland of the seawalls from design tsunamis or accidental waves shall have the dimensions necessary for the protection of the hinterland from tsunamis or accidental

waves.

(2) The degree of damage under the accidental situation, in which the dominating actions are design tsunamis, accidental waves, or Level 2 earthquake ground motion, shall be equal to or less than the threshold level in consideration of the performance requirements.

[Interpretation]

10. Protective Facilities for Harbors

(7) Performance Criteria for Seawalls

① **Common performance criteria for seawalls** (Article 16, paragraph 1, item 2 of the Ministerial Ordinance and the interpretation related to Article 39, paragraph 2, item 2 and 3 of the Public Notice)

(a) Serviceability shall be the required performance for seawalls under variable situations in which the dominating actions are water pressure, variable waves or level 1 earthquake ground motions. Performance verification items for those actions and standard indexes for setting limit values shall be in accordance with **Attached Table 10-7**.

Attached Table 10-7 Performance Verification Items and Standard Indexes for Setting Limit Values for Seawalls (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
16	1	2	39	2	2	Serviceability	Variable	Water pressure	Self-weight	Seepage failure of ground	—
					3			Variable waves [level 1 earthquake ground motion]	Self-weight, earth pressure, water pressure	Sliding or overturning of parapet* 1)	Action-to-resistance ratio for sliding Action-to-resistance ratio for overturning

Note: The action shown in brackets is an alternative dominating action.
*1): Limited to structures having a parapet.

(b) **Attached Table 10-7** shows no particular index for setting the limit value for seepage failure of the ground, so it is necessary to appropriately set the index when conducting the performance verification of a seawall for seepage failure.

(c) In addition to these provisions, the provisions of the Public Notice, Article 22, Paragraph 3 (Scouring and Sand Washing Out) and Article 28 (Performance Criteria of Armor Stones and Blocks) and their interpretation shall be applied to performance criteria of seawalls, as appropriate, and Article 23 through Article 27 shall also be applied, depending on the types of members comprising each seawall.

② **Seawalls serving as facilities prepared for accidental incidents** (the Ministerial Ordinance, Article 16, Paragraph 2, Item 2 and Paragraph 3, and the interpretation related to the Public Notice, Article 39, Paragraph 3, Item 2)

(a) Safety and restorability shall be the required performance for seawalls serving as facilities prepared for accidental incidents under accidental situations in which the dominating actions are level 2 earthquake ground motions, design tsunami or accidental waves, depending on the functions required for such seawalls. Performance verification items for those actions and standard indexes for setting limit values shall be in accordance with **Attached Table 10-8**. This table uses the non-specific term “damage” to describe the verification item because there will be different verification items depending on the structural type. When conducting the performance verification for this verification item, it is necessary to appropriately set a specific index for setting the limit value.

Attached Table 10-8 Performance Verification Items and Standard Indexes for Setting Limit Values for Seawalls Serving as Facilities Prepared for Accidental Incidents (Only Under Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
16	2	2	39	3	2	Safety, restorability	Accidental	Level 2 earthquake ground motion [Design tsunami] [Accidental waves]	Self-weight, earth pressure, water pressure	Damage	–

Note: The action shown in brackets is an alternative dominating action.

(b) In the performance verification of a seawall serving as a facility prepared for accidental incidents, the limit value of the degree of damage to the seawall under accidental situations in which the dominating actions are level 2 earthquake ground motions, design tsunami or accidental waves shall be set by giving comprehensive consideration to not only the functions of the seawall, but also the state of construction and maintenance of facilities serving to protect the land area behind the seawall, as well as intangible measures to reduce and prevent possible damage to the area. In cases where a seawall serves as a facility prepared for accidental incidents and its performance requirement is restorability, the limit value of the degree of damage shall be set by giving appropriate consideration to the allowable restoration period.

(c) When conducting the verification of a seawall serving as a facility prepared for accidental incidents in terms of performance for design tsunami, it is necessary to consider the effects of the action of earthquake ground motions preceding the tsunami. When doing so, it should be noted that the earthquake ground motions preceding the postulated design tsunami are not necessarily identical with the level 2 earthquake ground motions.

(d) In addition to these provisions, the provisions of the Public Notice, Article 22 (Common Performance Criteria for Members Comprising Facilities Subject to the Technical Standards) and their interpretation shall be applied to performance criteria of seawalls serving as facilities prepared for accidental incidents concerning their performance under accidental situations.

(e) A seawall serving as a facility prepared for accidental incidents shall have a structure designed to maintain its stability as long as possible in order to ensure that the seawall demonstrates effectiveness in reducing possible damage even when the seawall is exposed to the action of a tsunami of which the intensity, at the site where the seawall is located, is higher than that of the design tsunami.

10.1 General

(1) The purpose of seawalls is to protect the land areas behind them from waves, storm surges and tsunamis.

(2) The provisions of this section may also apply to revetments, coastal dikes, parapets, floodgates, locks and land locks that compose a tide barrier system.

(3) Seawalls based on the technical standards include both public and private facilities, unlike shore protection facilities, which are public facilities under the **Technical Standards for Shore Protection Facilities**. Therefore, it must be noted that the level of protection to be provided by a seawall may differ from the level of protection to be provided by shore protection facilities as specified by the seashore administrator depending on the importance of the posterior facilities that need to be protected by the seawall.

(4) Seawalls shall be designed in accordance with this document and by reference to the **Technical Standards and Commentary for Shore Protection Facilities**[1], the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Ports and Harbors**[2], the **Design Concept for Parapets for Tsunamis (Provisional Edition)**[3], the **Guidelines for Tsunami-Resistant Design of Breakwaters**[4] and the **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores.**[5] If the facility of interest is a shore protection facility, it shall conform to the **Technical Standards for Shore Protection Facilities**.

10.2 Layout

(1) Seawalls shall be located appropriately, in consideration of future plans of the port or harbor concerned, to ensure that they can protect the land areas behind them from waves, storm surges and tsunamis, and will obstruct as little as possible the physical distribution, traffic and other movements inside and outside the port or harbor.

(2) It is possible to choose to build seawalls alone or to build both seawalls and breakwaters as facilities that protect land areas behind them in a port or harbor.

10.3 Setting of Basic Cross Section

(1) The basic cross section of a seawall shall be set in accordance with **Part III, Chapter 4, 14 Revetments**, **Part III, Chapter 4, 15 Coastal Dikes and Part III, Chapter 4, 17 Parapets** with modifications made as necessary in consideration of the structural type.

(2) The required crown height of a seawall shall be determined according to the performance requirements of the facility and by giving appropriate consideration to waves, tidal levels (including during storm surges), tsunamis, settlement after construction (consolidation settlement and settlement due to earthquake ground motions) and other conditions in the location where it will be constructed. For waves, tidal levels and tsunamis, refer to **Part II, Chapter 2, 4 Waves**, **Part II, Chapter 2, 3 Tidal Level** and **Part II, Chapter 2, 5 Tsunamis**, respectively.

10.4 Actions and Performance Verification

(1) Seawalls shall be designed by appropriately setting actions and design situations to be considered as well as performance criteria in accordance with the performance requirements for the facilities.

(2) For the performance verification of seawalls, refer to the following descriptions:

① **Wave overtopping rate and permissible rate of wave overtopping**

When setting the layout and dimensions of a seawall (its structure and cross-sectional dimensions and ancillary facilities) in the performance verification, it is necessary to appropriately verify that the wave overtopping rate will not exceed the permissible rate of wave overtopping. When evaluating the wave overtopping rate in the performance verification of the seawall, it is necessary to appropriately consider environmental conditions to which the seawall will be subjected and its structural characteristics. When setting the permissible rate of wave overtopping in the performance verification of the seawall, it is necessary to appropriately consider the density of houses, public facilities and other buildings in the land area behind the seawall and the conditions of use of those buildings, as well as the capacities of drainage facilities in the land area behind the seawall.

② **Wave force and hydrostatic pressure acting on a seawall**

The wave force acting on a seawall shall be set appropriately by reference to **Part II, Chapter 2, 6 Wave Force**. The water pressure acting on a wall body, such as a parapet, needs to be set appropriately in consideration of the simultaneous actions of the increased water pressure due to a storm surge and the wave pressure caused by waves by referring to **Part II, Chapter 2, 6.2.10 Wave Force and Hydrostatic Pressure during a Storm Surge (When the Tide Level is High)**."

③ **Effects of settlement caused by level 1 earthquake ground motions**

Seawalls are required to fulfill the function of appropriately controlling wave overtopping even after ground settlement. Therefore, when evaluating the wave overtopping rate in the performance verification of a seawall, it is necessary to appropriately consider the effects of ground settlement caused by the action of level 1 earthquake ground motions.

④ **Ancillary facilities**

In the performance verification of a seawall, it is necessary to appropriately consider ancillary facilities, including apron work, drainage ditches, drainage holes and drainage facilities that are provided for protecting the land area behind the seawall from wave overtopping and getting flooded, in order to ensure that the land area behind the seawall can be protected appropriately from waves and storm surges.

⑤ **Measures to prevent washing-out**

In the performance verification of a seawall, it is necessary to pay attention to the prevention of washing-out of backfill soil behind the seawall in consideration of the structural type. It is also necessary to take measures to prevent washing-out of backfill soil, for example, by placing sand invasion prevention sheets or sand invasion prevention plates, as appropriate.

(3) For the performance verification of seawalls serving as facilities prepared for accidental incidents, refer to the following descriptions:

① **Common items**

Performance criteria for specifications common to all seawalls as described above in (2), ① through ⑤, shall be applied to seawalls serving as facilities prepared for accidental incidents, except for postulated environmental conditions such as level 2 earthquake ground motions, tsunamis and accidental waves.

② **Setting of the limit value of the degree of damage to seawalls under accidental situations**

When setting the limit value of the degree of damage to a seawall under accidental situations, it is necessary to give comprehensive consideration to not only the functions of the seawall, but also to the state of construction and maintenance of floodgates and other harbor protective facilities that compose a tide barrier system and other peripheral facilities, as well as intangible (non-structural) measures to reduce and prevent possible damage in the area concerned.

③ **Accidental situation in which the dominating action is a design tsunami**

(a) Stability against design tsunami and tsunamis with intensity higher than that of the design tsunami

For evaluating the stability against design tsunami and tsunamis with intensity higher than that of the design tsunami in order to develop a tsunami-resistant design, refer to the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Ports and Harbors**.[2)]

(b) Consideration of the effects of earthquake ground motions

In the verification of performance of a facility for design tsunami, it is necessary to appropriately consider the possibility that, when the postulated design tsunami is triggered by an earthquake of which the seismic center is close to the facility, the facility may be subjected to the action of earthquake ground motions caused by the earthquake before it is subjected to the action of the design tsunami. In view of this, it is necessary to consider the effects of the action of earthquake ground motions preceding the design tsunami when conducting the verification of performance for the design tsunami. When doing so, it must be noted that the earthquake ground motions preceding the postulated design tsunami are not necessarily identical with the level 2 earthquake ground motions.

④ **Accidental situation in which the dominating action is accidental waves**

(a) Conditions of accidental waves

Conditions of accidental waves shall be set appropriately by reference to **Part II, Chapter 2, 4 Waves** and **part II, Chapter 2, 3 Tidal Level**."

(b) Consideration of the effects of storm surges

In the verification of performance of a facility for accidental waves, it is necessary to appropriately consider a storm surge that occurs at the same time as the postulated waves. For setting the conditions of storm surges, refer to **Part II, Chapter 2, 4.1.1 Setting of Wave Conditions to be Used for Verification of Stability of Facilities and Safety (for Cross-Sectional Failure) of Structural Members**, **Part II, Chapter 2, 3.2 Storm Surges** and **Part II, Chapter 2, 3.6 Design Tide Level Conditions**.

(c) Design storm surges

In the performance verification of facilities prepared for accidental incidents, design storm surges shall be set appropriately in consideration of storm surges up to the largest class. For setting a design storm surge, refer to **Part II, Chapter 2, 4 Waves** and **Part II, Chapter 2, 3 Tidal Level**.

[References]

1) National Association of Agricultural Sea Coast, National Association of Fisheries Infrastructure, National Association of Sea Coast, Ports and Harbours Association of Japan: Technical Standards and Commentary for shore Protection Facilities, 2018 (in Japanese)
2) MLIT, Ports and Harbours Bureau: Guidelines for Earthquake-resistant Design of Storm Surge Barriers (Breast Walls) in Ports, 2013 (in Japanese)
3) Fisheries Agency, Fishery Infrastructure Department, Fishing Communities Promotion and Disaster Prevention Division and MLIT, Ports and Harbours Bureau, Coast Administration and Disaster Prevention Division: Design Concept of Breast Walls for Tsunami (Tentative Edition), 2015 (in Japanese)
4) MLIT, Ports and Harbours Bureau: Guideline for Tsunami-Resistant Design of Breakwaters (Partial Revision), 2015 (in Japanese)
5) Coastal Development Institute of Technology: Technical Manual for Flap-type Land Gates at Ports and Coasts, 2016 (in Japanese)

11 Training Jetties

[Ministerial Ordinance] (Performance Requirements for Training Jetties)

Article 17

1 The performance requirements for training jetties shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied for the purpose of mitigating siltation in waterways and basins and preventing the closure of river mouths due to littoral drift through effective control of littoral drift.

2 The provisions of Article 14, paragraph (1), item (ii) shall apply mutatis mutandis to the performance requirements for training jetties.

[Public Notice] (Performance Criteria of Training Jetties)

Article 40

The provisions of Article 38 shall apply mutatis mutandis to the performance criteria of training jetties.

[Interpretation]

10. Protective Facilities for Harbors

(8) Performance Criteria of Training Jetties (Article 17 of the Ministerial Ordinance and the interpretation related to Article 40 of the Public Notice)

① Performance criteria of sediment control groins and their interpretation shall be applied correspondingly to training jetties.

② In the performance verifications of training jetties, appropriate consideration shall be given to the effects of an increase in earth pressure due to sedimentation of littoral drift and the effects of waves and river currents, as needed, in addition to the previous paragraph.

11.1 General

(1) Layout and Shape of Training Jetties

Examples of the layout of training jetties in relation to the direction of longshore sediment transport are shown in **Fig. 11.1.1**.[1] The most preferable layout for maintaining the water depth at the mouth of a river is to extend two parallel training jetties because a single training jetty alone is not effective. Where two training jetties of different lengths are put in place, it is usually effective to make the training jetty on the downdrift side longer than the other. Bending the updrift side training jetty toward the downdrift side will prevent sediment from moving into the area between the two training jetties and allow smooth longshore sediment transport to the downdrift side. For actual examples of river mouth improvement, refer to **Reference 2)**.

(2) Water Depth at Training Jetty Tips

① The water depth at the tip of a training jetty should be equal to or greater than the water depth of the waterway in the vicinity of the training jetty.

② The tip of the training jetty should be located at a point where the water depth is equal to or greater than the wave breaking limit depth.

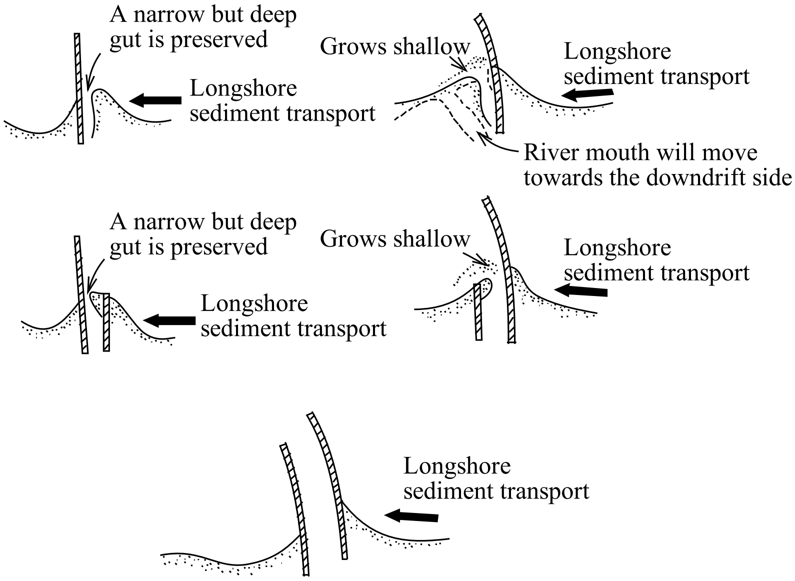


Fig. 11.1.1 Varieties of Training Jetty Layouts[1)]

11.2 Performance Verification

(1) For the performance verification of training jetties, refer to **Part III, Chapter 4, 3 Breakwaters Having Basic Functions**" in consideration of the structural type. Note, however, that it is necessary to give appropriate consideration to the effects of an increase in earth pressure due to sedimentation of littoral drift and the effects of waves and river currents.

(2) Training jetties are generally longer than groins and are exposed to intensive wave actions. Therefore, it is necessary to consider scouring at the tip and lateral sides of the training jetty. It is also necessary to consider that the river side of the training jetty will be subject to scouring actions of the river current.

[References]

1) JSCE: Handbook of Civil Engineering, (Vol. 2), pp.2268-2270, 1974 (in Japanese)

2) Uda, T., A. Takahashi and H. Matsuda: Nationwide classification of River Mouth Morphology and River Mouth Improvement, Technical Note of PWRI No. 3281, 123p., 1994. (in Japanese)

12 Floodgates

(English translation of this section from Japanese version is currently being prepared.)

12.1 General

(English translation of this section from Japanese version is currently being prepared.)

12.2 Setting of Layout and Dimensions of Floodgates

(English translation of this section from Japanese version is currently being prepared.)

12.3 Performance Verification of Floodgates

(English translation of this section from Japanese version is currently being prepared.)

13 Locks

(English translation of this section from Japanese version is currently being prepared.)

13.1 General

(English translation of this section from Japanese version is currently being prepared.)

13.2 Setting of Layout and Dimensions of Locks

(English translation of this section from Japanese version is currently being prepared.)

13.3 Structures and Performance Verification of Locks

(English translation of this section from Japanese version is currently being prepared.)

14 Revetments

[Ministerial Ordinance] (Performance Requirements for Revetments)

Article 20

1 The provisions of Article 16 shall apply mutatis mutandis to the performance requirements for revetments.

2 In addition to the provisions of the preceding paragraph, the revetments specified in the following items shall satisfy the performance requirements prescribed respectively in those items:

(1) “Performance requirements for revetments intended for environmental conservation” shall be such that revetments satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to contribute to conservation of the environment of ports and harbors without impairing the original functions of the revetments.

(2) “Performance requirements for revetments to be utilized by an unspecified large number of people” shall be such thatrevetments satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to secure the safety of users of the ports.

[Public Notice] (Performance Criteria of Revetments)

Article 43

1 The provisions of Article 39 shall apply mutatis mutandis to the performance criteria for revetments.

2 In addition to the provisions of the preceding paragraph, the revetments specified in the following items shall satisfy the performance criteria prescribed respectively in those items:

(1) “Performance criteria of revetments intended for environmental conservation” shall be such that revetments shall have the dimensions necessary to contribute to conservation of the environment of ports and harbors in consideration of the environmental conditions, etc. to which the revetments are subjected, without impairing their original functions.

(2) “Performance criteria of revetments to be utilized by an unspecified large number of people” shall be such that revetments shall have the dimensions necessary to secure the safety of users of the ports and harbors depending on the environmental conditions, usage conditions, etc. to which the revetments are subjected.

[Interpretation]

10. Protective Facilities for Harbors

(11) Performance criteria of revetments

① **Symbiotic revetments** (Article 20, Paragraph 2, Item 1 of the Ministerial Ordinance and the interpretation related to Article 43, Paragraph 2, Item 1 of the Public Notice)

(a) Performance criteria of seawalls and their interpretation shall be applied correspondingly to revetments.

(b) Revetments intended for environmental conservation are referred to as “symbiotic revetments.” In addition to the provisions applicable to revetments, the following items shall be applied to symbiotic revetments.

(c) Usability shall be the performance requirement for symbiotic revetments. The “usability” here denotes the performance of revetments in contributing to the conservation of environments of ports and harbors, including living things and ecological systems, without impairing the original functions of the revetments.

(d) The dimensions of revetments intended for environmental conservation denote their structures, cross-sectional dimensions, and ancillary facilities. When setting the structures and cross-sectional dimensions of revetments intended for environmental conservation in their performance verification and when installing their ancillary facilities, appropriate consideration shall be given to the factors that affect the capability of the revetments to fulfill the purpose to conserve environments of ports and harbors, including living things and ecological systems, without impairing the original functions of the revetments.

② **Amenity-oriented revetments** (Article 20, Paragraph 2, Item 2 of the Ministerial Ordinance and the interpretation related to Article 43, Paragraph 2, Item 2 of the Public Notice)

(a) Revetments to be utilized by an unspecified large number of people are referred to as "amenity-oriented revetments." In addition to the provisions applicable to revetments, the following items shall be applied to amenity-oriented revetments.

(b) Usability shall be the performance requirement for amenity-oriented revetments. The "usability" here denotes the performance of revetments in securing the safety of their users depending on conditions, including the environmental conditions to which the revetments are subjected and the conditions of use of the revetments.

(c) The dimensions of amenity-oriented revetments denote their structures, cross-sectional dimensions, and ancillary facilities. When setting the structures and cross-sectional dimensions of amenity-oriented revetments in the performance verification and when installing their ancillary facilities, consideration shall be given to the effects of wave overtopping and spray, prevention of their users from slipping, overturning or falling into the water, and smooth rescue of users who have fallen into the water. Ancillary facilities such as fall prevention fences shall be installed appropriately.

14.1 General

(1) The purpose of revetments is to protect the land areas behind them from waves, storm surges, and tsunamis.

(2) For the performance requirements and performance criteria of revetments, refer to the descriptions about seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(3) Revetments to which damage might significantly affect human lives, properties, and/or socioeconomic activities and which are required to protect the land areas behind them from the design tsunami and accidental waves shall be given a purpose to protect the land areas behind them from the actions concerned above, in addition to the purpose mentioned in (1).

(4) Revetments shall be designed in accordance with this document and by reference to the **Technical Standards and Commentary for Shore Protection Facilities** [1], the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors** [2], the **Design Concept for Parapets against Tsunamis (Provisional Edition)** [3], the **Guidelines for Tsunami-Resistant Design of Breakwaters** [4] and the **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores** [5]. If the facility of interest is a shore protection facility, it shall conform to the **Technical Standards for Shore Protection Facilities**.

(5) This section covers ordinary reclamation revetments. For reclamation revetments for reclaimed areas used as final disposal sites for general waste (disposal sites defined in Paragraph 2, Article 5 of the **Order for Enforcement of the Waste Management and Public Cleansing Act**) or used as final disposal sites for industrial wastes (disposal sites defined in Item (14), Article 7 of the said order), the performance verification shall be conducted in accordance with **Part III, Chapter 11**, **2 Waste Disposal Seawalls**.

14.2 Items to be Considered in Setting of Basic Cross Section

(1) In setting the basic cross section of a revetment, the following items shall be examined in principle:

① The revetment shall have the crown height that ensures prevention of waves and storm surges from affecting preservation and use of the reclaimed land.

② Stability shall be secured against the actions of waves, earth pressure, and others.

③ The revetment shall have a structure that prevents leakage of the landfill soil.

④ Consideration shall be given to the effect on surrounding water areas, including prevention of outflow of turbid water during reclamation work.

⑤ When the revetment is an amenity-oriented revetment, it shall have a structure that allows people to use it in safe and comfortable ways.

(2) Generally, reclaimed areas are surrounded by revetments unless there are mooring facilities. Therefore, reclamation revetments should serve as stable earth retaining work that is stable against waves, prevents the landfill soil from leaking out, and protects the reclaimed areas behind them from wave overtopping and storm surges. Reclamation revetments facing the open sea are exposed to more severe conditions of waves and other actions than ordinary reclamation revetments and require careful examination of performance under those conditions.

(3) When setting the cross section of a revetment, it is necessary to appropriately consider its ancillary facilities, including apron work, drainage ditches, and drainage holes that are provided for protecting the land area behind the revetment from wave overtopping and drainage facilities that are provided for preventing the land area behind the revetment from getting flooded, to ensure that the revetment can adequately protect the land area behind it from waves and storm surges.

(4) When setting the basic cross section of a revetment, it is necessary to pay attention to prevention of washing-out of backfill soil behind the revetment body in consideration of the structural type. It is also necessary to take measures to prevent washing-out of backfill soil, for example, by placing sand invasion prevention sheets or sand invasion prevention plates, as appropriate. In particular, when setting the basic cross section of an amenity-oriented revetment, it is necessary to consider appropriate measures to prevent washing-out of backfill soil behind the revetment body, as appropriate.

(5) There are cases where a temporary reclamation revetment is constructed with a structure just enough to prevent backfill soil from leaking out during reclamation work and a final reclamation revetment or mooring facilities are constructed after the completion of reclamation work. Temporary revetments can be classified into the following types:

① Temporary revetment that has a structure built using low-cost materials and construction and is not intended to be used in the future

② Temporary revetment that is intended to be used as a final revetment in the future after reinforcement of its structure

Temporary revetments may be composed of wood fences, stone frames, and the like Rubble-mound breakwaters may be used as temporary revetments. There are temporary revetments with semi-permanent structures composed of light-weight steel sheet piles in place of wood fences or composed of corrugated cells. There are also cases where structural types generally used for final revetments are used for temporary revetments.

In the performance verification of a temporary revetment, it is necessary to appropriately set the safety level and the limit value of allowable deformation in consideration of the purpose of the revetment. When doing so, it is necessary to ensure that the temporary revetment has the required stability against waves to which it will be exposed before completion of a final revetment or quaywall. It is also necessary to determine the crown height to ensure that the revetment prevents waves and storm surges from affecting the reclaimed area before it is replaced by the final revetment or quaywall.

14.3 Points to Remember Concerning Land Reclamation and Construction of Revetments

(1) For land reclamation, refer to **Part III, Chapter 2**, **6 Land Reclamation**.

(2) For points to remember concerning land reclamation and construction of revetments, refer to the followings:

① For reclamation of soft clayey soil, it is necessary to take measures, such as backfilling of rubble, to reduce the earth pressure acting on the revetment and prevent the landfill soil from leaking out through joints or the foundation.

② In cases where landfilling work is done by a suction dredger and the foundation ground of a reclamation revetment has high permeability, there is a possibility that the soil of the foundation ground and the dumped soil might flow out because of surplus water and result in a failure of the revetment body and/or outflow of soils. Therefore, attention should be paid to these possibilities in the performance verification and construction of the revetment. In general, materials discharged by suction dredgers into reclaimed areas are in the form of slurry. Therefore, it is necessary to carefully determine the position of the opening of the discharge pipe of each suction dredger and the layout of spillways to ensure that the rear side of the revetment body will not be directly exposed to slurry flows.

③ In cases where a reclamation revetment is built adjoining to an existing land area, construction of the revetment may cause the groundwater level to rise or may result in deterioration of groundwater quality. Adequate attention should be paid to these possibilities when studying the reclamation layout plan and the revetment structure. It is preferable to investigate the conditions of the groundwater in the land area in advance. In addition, in cases where it is likely that reclamation revetment construction will cause deterioration of the groundwater quality, countermeasures such as construction of a cut-off wall must be considered.

④ In the case of reclamation where a large water area is enclosed by revetments, the opening through which seawater flows into and out of the area because of the tidal range becomes smaller with the progress of revetment construction, and a considerably rapid flow occurs at closing sections because of the difference in the water level between the inside and the outside of revetments. Therefore, careful consideration is required for the structure of revetments at the final closing section, which should have a cross section that ensures adequate stability of the structure against the expected flow speed. The flow velocities at closing sections are affected by the water area being closed, the cross-sectional areas of the closing sections, the average water depth, the tidal range, and other factors.

In the closing sections, it is preferable that consolidation work be conducted at a location with good ground before the flow velocity increases as work progresses. There are also cases in which a submerged weir or broad-crested weir is used depending on the flow velocities at the closing sections.

⑤ There are cases where a reclaimed area is partitioned depending on the sequence of land reclamation work and the reclamation method. Generally, there are no strict requirements for partitions in regard to waves, the crown height, the degree of prevention of soil leakage, the importance, and others. The performance verification of partitions may be conducted in the same manner as conducted for final or temporary revetments.

14.4 Setting of Crown Height of Revetment

(1) Setting of Crown Height

① The crown height of a revetment shall be set to an appropriate height in consideration of the wave overtopping rate, the tidal level during a storm surge, and other conditions so that the revetment can contribute to preservation of the reclaimed area behind it and prevent waves and storm surges from affecting the use of the revetment and the land area behind it.

② It is preferable to examine measures to reduce the wave overtopping rate of a revetment from a comprehensive point of view in consideration of waves, the tide level and other environmental conditions, the geographical features of the seabed around the revetment, the possibilities of future construction of detached breakwaters and submerged breakwaters in the water area in front of the revetment, the shape of the cross section of the revetment, the shape of the crest of the revetment (the possibility of future construction of a recurved parapet), and other factors.

③ For calculating the wave overtopping rate and the wave run-up height, refer to **Part II, Chapter 2**, **4.4.7 Wave Run-up Height, Wave Overtopping and Transmitted Waves (1)**.

④ The crown height of a revetment can be set by using the following method [6]:

(a) The required crown height h_d above the design high water level of the revetment can be set as follows, using the required crown height h_c above the water level commensurate with the importance of the land area behind the revetment, or the required crown height h_c' allowing for earthquake ground motions and the crest settlement d_s determined from the ground conditions such as consolidation.

$$h_d = \max(h_c, h_c') + d_s \qquad \textbf{(14.4.1)}$$

The required crown height h_c above the water level in **equation (14.4.1)** shall be a value obtained by adding a margin height to the calculated crown height for the design wave above the design high water level of the revetment.

(b) The required crown height h_c above the water level can also be calculated by setting the exceedance probability P for the permissible rate of overtopping. The exceedance probability P for the permissible rate of overtopping can be calculated using **equation (14.4.2)**. The mean value and the standard deviation of h_c/h_{cd} can be assumed to be 1.00 and 0.15, respectively. These values can be applied to revetments of any structural type and with any permissible rate of overtopping, because they were obtained from the results of a statistical analysis of 89 facilities based on the Study on the Dimensions of Embankment and Seawall [7], which provides data on existing embankments and seawalls all over Japan.

$$P = 1 - \int_0^z \frac{1}{\sqrt{2\pi}\, z\zeta} \exp\left\{-\frac{1}{2}\left(\frac{\ln z - \lambda}{\zeta}\right)^2\right\} dz$$

provided, however, that **(14.4.2)**

$$z = \frac{h_c}{h_{c_d}}$$

where

P : exceedance probability of permissible rate of overtopping

h_c : required crown height above water level (m)

h_{cd} : calculated crown height for design wave above design high water level of revetment (m)

ζ : standard deviation of $\ln(h_c/h_{cd})$; given by $\zeta = \sqrt{\ln\left\{1 + \left(\frac{\sigma}{\mu}\right)^2\right\}}$

λ : mean value of $\ln(h_c/h_{cd})$; given by $\lambda = \ln\mu - \frac{1}{2}\zeta^2$

μ : mean value of h_c/h_{cd} (can be assumed to 1.00)

σ : standard deviation of h_c/h_{cd} (can be assumed to 0.15)

Equation (14.4.2) is shown graphically in **Fig. 14.4.1**. For example, assuming the exceedance probability of the permissible rate of overtopping is 0.01, the required crown height h_c above the water level, which is obtained by adding a margin height to the calculated crown height h_{cd}, is given as 1.40 times the calculated crown height h_{cd}.

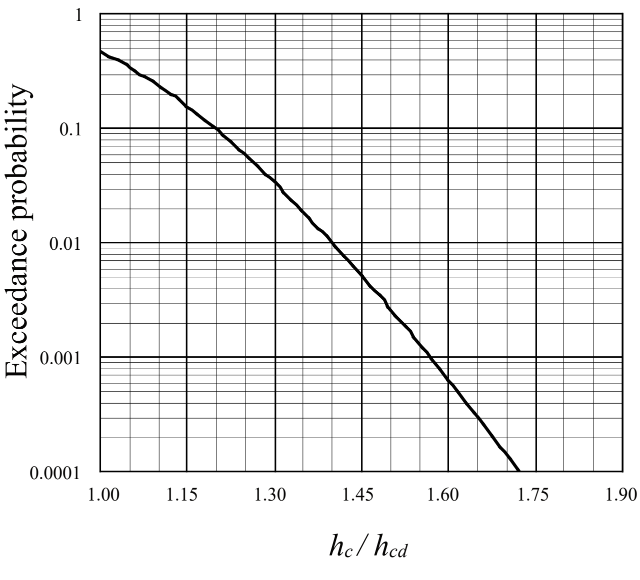
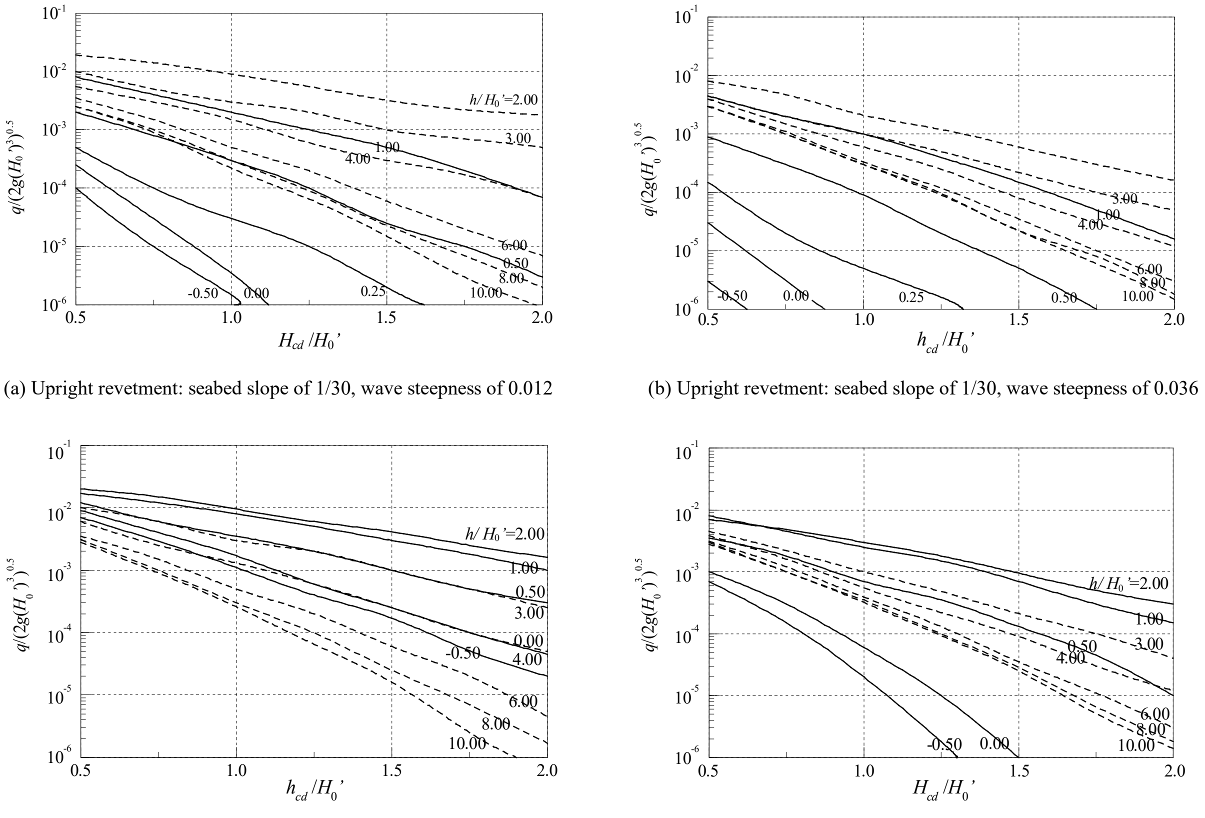


Fig. 14.4.1 Relationship of Exceedance Probability of Permissible Rate of Overtopping to h_c/h_{cd} (Required Crown Height above Water Level / Calculated Crown Height)

(c) The calculated crown height of a revetment shall be calculated as the crown height that satisfies the permissible rate of overtopping. For upright revetments and upright wave-absorbing revetments, this calculation shall be based on the diagrams for estimating wave overtopping rate given in **Part III, Chapter 2**, **4.4.7 Wave Run-up Height, Wave Overtopping and Transmitted Waves (2)**. For other types of revetments, this calculation shall be based on the diagrams for estimating allowable settlement that were drawn by Sekimoto et al.[8)] The diagrams for estimating wave overtopping rate given in **Part III,**

Chapter 2, **4.4.7 Wave Run-up Height, Wave Overtopping and Transmitted Waves (2)** show the relationship of the non-dimensional wave overtopping rate to the ratio of the wave height to the water depth at the toe of the slope h/H_0'. Diagrams for calculating allowable settlement have been proposed [9], which show the relationship of the non-dimensional wave overtopping rate to the relative crown height h_{cd}/H_0' based on the diagrams for estimating wave overtopping rate. Those proposed diagrams can be used for easily obtaining the calculated crown height of a revetment (See **Figures 14.4.2** and **14.4.3**). The notation of those diagrams is similar to that of the diagrams for estimating wave overtopping rate given in **Part III, Chapter 2**, **4.4.7 Wave Run-up Height, Wave Overtopping and Transmitted Waves (2)**.



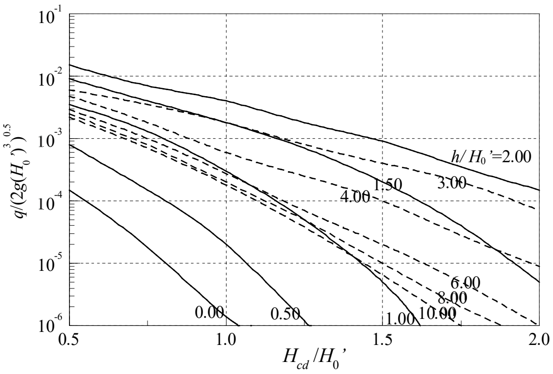
(a) Upright revetment: seabed slope of 1/30, wave steepness of 0.012

(b) Upright revetment: seabed slope of 1/30, wave steepness of 0.036

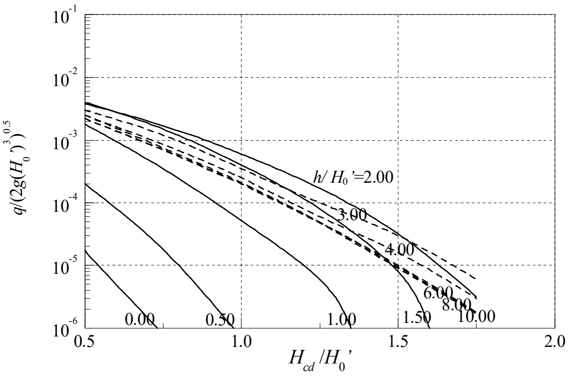
(c) Upright revetment: seabed slope of 1/10, wave steepness of 0.012

(d) Upright revetment: seabed slope of 1/10, wave steepness of 0.036

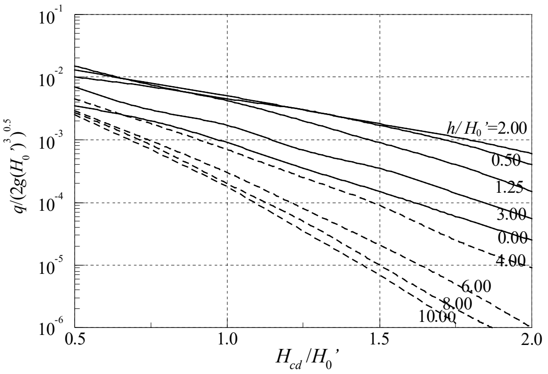
Fig. 14.4.2 Relationship of Non-dimensional Wave Overtopping Rate to the Relative Crown Height h_{cd}/H_0' (Uptight Revetments)



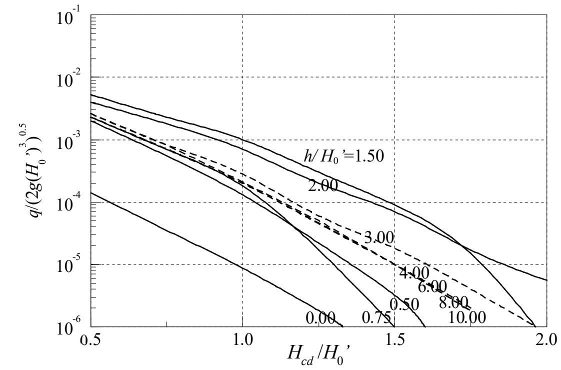
(a) Upright wave-absorbing revetment: seabed slope of 1/30, wave steepness of 0.012



(b) Upright wave-absorbing revetment: seabed slope of 1/30, wave steepness of 0.036



(c) Upright wave-absorbing revetment: seabed slope of 1/10, wave steepness of 0.012



(d) Upright wave-absorbing revetment: seabed slope of 1/10, wave steepness of 0.036

Fig. 14.4.3 Relationship of Non-dimensional Wave Overtopping Rate to the Relative Crown Height h_{cd}/H_0' (Uptight Wave-absorbing Revetments)

(d) The required crown height h_c' allowing for earthquake ground motions can be determined by adding the crest settlement because of the action of earthquake ground motions to the required crown height h_c above the water level. The required crown height h_c above the water level can be obtained by setting the return period for waves to be considered in the verification, considering the period required for recovery from damage caused by an earthquake, and determining the crown height that satisfies the permissible rate of overtopping of the said waves. For calculating the crest settlement because of the action of earthquake ground motions, refer to **Reference (Part III], Chapter 1**, **2 Fundamentals of Seismic Response Analysis**. The crest settlement because of earthquake ground motions can also be estimated from the horizontal deformation of a mooring facility of similar structural type. For example, for gravity-type revetments refer to reference 9).

(e) For the crest settlement d_s that can be determined from the ground conditions such as consolidation, refer to **Part III, Chapter 2**, **3.5 Settlement of Foundation**.

(f) For revetments that require consideration of accidental actions, including the design tsunami and accidental waves, the crown height required to resist these actions shall be considered in the first term of the right-hand side of **equation (14.4.1)**.

(g) When setting the crown height above the design high water level for a revetment, it is possible to include a margin height of up to 1 m above the required crown height h_d, as needed, considering uncertainties in design conditions and countermeasures against long-term sea level rise.[1]

⑤ The crown height of a reclamation revetment may be reduced when wave-dissipating work is constructed on the front of the revetment. It must be noted, however, that wave overtopping might occur when the difference between the crown height of the revetment and the water level in the reclaimed area becomes smaller during reclamation work.

⑥ For measures to reduce wave overtopping rate of an upright or sloping revetment because of swelling waves, refer to reference 10) and other literature.

14.5 Actions

(1) For actions due to the tide level, refer to **Part II, Chapter 2**, **3 Tidal Level**.

(2) For actions due to waves, refer to **Part II, Chapter 2**, **4 Waves**.

(3) The wave force acting on a revetment shall be set appropriately by reference to **Part II, Chapter 2**, **6 Wave Force**. The water pressure acting on a wall body like a parapet on top of the revetment needs to be set appropriately in consideration of the simultaneous actions of the increased static water pressure caused by water level rise and the wave pressure caused by waves, by reference to **Part II, Chapter 2**, **6.2.10 Wave Force and Hydrostatic Pressure during Storm Surge (When the Tide Level is High)**.

(4) For the soil conditions of landfill soil, foundation ground and the like, refer to **Part II, Chapter 3 Geotechnical Conditions**.

(5) For actions because of earthquake ground motions, refer to **Part II, Chapter 6 Earthquakes**.

(6) For dynamic water pressure during the action of earthquake ground motions, refer to **Part II, Chapter 4**, **3.2 Dynamic Water Pressure**.

(7) As water levels in reclaimed areas, two water levels are generally set, these being the water level in the reclamation site and the residual water level. The water level in the reclamation site is used in seepage calculations and the performance verification of surplus water treatment facilities. The residual water level is the water level immediately behind a revetment and is mainly used in examination of the stability of the revetment. However, in cases where the water level at a position near the revetment is higher than the residual water level, using the residual water level in the examination of circular slip failure may result in underestimation of the danger of circular slip failure. In such cases, it is necessary to examine the stability of the revetment for the water level in reclamation site in addition to the stability for the residual water level.

① **Water level in a reclamation site**

The water level in a reclamation site shall be set by considering the stability of the revetment both during construction and after completion, and the influence on the surrounding waters. Regarding the influence on the surrounding waters, particular attention should be paid to overtopping flows due to waves generated behind the revetment during construction. It must be noted that, if the water level in the reclamation site is excessively high in comparison with the water level at the front of the revetment, an increased amount of water, including polluted water, may seep out of the revetment and its foundation ground.

② **Residual water level**

(a) Most reclamation revetments have low-permeable structures in order to reduce the seepage of polluted water out of reclamation sites. For this reason, the residual water level behind them is generally higher than that behind ordinary mooring facilities or revetments.

(b) In past construction projects of reclamation revetments with gravity-type structures, there were more cases in which permeability is reduced by increasing the layer thickness of the levee-widening earth or the backfilling sand than cases in which the permeability of the revetment body itself was reduced. For a revetment having a structure like this, the water level just behind the revetment body shows behavior similar to that behind the body of an ordinary gravity-type revetment. Therefore, the performance verification of the revetment body may be conducted by using the same residual water level as that used for ordinary gravity-type revetments.

(c) For reclamation revetments with sheet pile structures, there are cases where grout is poured into sheet pile joints or a double sheet pile structure is adopted to increase the water-tightness of sheet piles. In these cases, the residual water level behind a reclamation revetment tends to be higher than that behind an ordinary sheet pile revetment. For a reclamation revetment having a sheet pile structure like this, it is necessary to set an appropriate residual water level, giving due consideration to the water-tightness of the revetment and, for a double sheet pile structure, taking account of the crown height of sheet piles and construction conditions.

14.6 Performance Verification

14.6.1 Common Items

(1) For the performance verification of revetments, refer to the descriptions about seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(2) In case of reclamation using suction dredgers, there are cases in which suspended soft soil concentrates behind the revetment and the greater-than-expected earth pressure acts on the revetment body, and cases in which the action of the water pressure at the back side of the structure extends as far as the crest of the revetment. Therefore, it is necessary to give adequate consideration to these possibilities when conducting the performance verifications.

(3) In general, it takes a long time to reclaim land. Therefore, it is necessary to conduct the performance verification in consideration of various conditions during reclamation work. Particularly, when there is a possibility that circular slip failure might occur, the stability during reclamation work shall be examined for each cross section at each construction step. In a case where the reclamation site is subjected to a great action of waves, it is also necessary to examine the stability against waves during reclamation work by reference to **Part III, Chapter 4**, **3 Breakwaters Having Basic Functions**.

(4) In order to estimate the quantity of polluted water seeping out of a reclamation revetment into the sea, it is necessary to perform an analysis of seepage flows. In general, Darcy's law can be applied to seepage flow analysis. However, as will be discussed in the following text, the cross section of a revetment consists of different materials, including sheet piles and concrete members, and backfilling sand. Furthermore, the permeability of sheet piles will differ from that of their joints. For this reason, there are cases in which Darcy's law cannot be applied.

In analysis of seepage flows in these cases, it is realistic to treat the cross section of the revetment as a structure comprising materials to which Darcy's law can be applied. Therefore, it is necessary to convert the coefficient of permeability and the wall width in order to apply Darcy's law in an approximate manner.

Though the seepage flow analysis should cover the area behind a reclamation revetment in which the water level can be considered uniform, the analysis can be performed by setting the area commensurate with the required accuracy, considering the structure of the revetment body, conditions of backfilling sand and other conditions. It must be noted, however, that when the permeability of the landfill soil deposited in the reclaimed area is low, the water level behind the reclamation revetment will have a steep gradient in the landfill soil.

① **Permeability of steel sheet pile structures**

(a) The permeability of steel sheet pile structures cannot be derived from Darcy's law. Therefore, it is common to use an appropriate equivalent width and the equivalent coefficient of permeability for that width to determine the permeability in the seepage flow analysis. When doing so, it is preferable to use results of in-situ measurements because it is difficult to say that on-site conditions of joints can be well-simulated in laboratory tests.

(b) Reference 11) gives an example of analyzing the permeability of steel sheet pile structures in situ. The analysis is based on measurements of residual water levels at five steel sheet pile quaywalls. In the analysis, it was assumed that the part of a sheet pile wall below the seabed is an impermeable layer and the part of the wall above the seabed is a 1-m thick permeable layer to which Darcy's law can be applied. The resultant coefficient of permeability, that is, the equivalent coefficient of permeability, was in the range of $1x10^{-5}$ to $3x10^{-5}$ cm/s. The same analysis was carried out for two steel pipe sheet pile quaywalls with a diameter of approximately 80 cm and having L-T joints, and the results indicated that the coefficient of permeability for those quaywalls was $6x10^{-5}$ cm/s. The coefficient of permeability for the backfilling material for the quaywalls mentioned above was in the order of 10^{-2} to 10^{-3} cm/s.

(c) The permeability of steel sheet pile joints has the following characteristics:

In cases of structures with no backfilling material, the permeability of sheet pile joints is similar to that of orifices with abrupt reduction in the cross-sectional area and can be expressed in **equation (14.6.1)** with the constant $n = 0.5$. [12), 13)]

$$q = Kh^n \qquad \textbf{(14.6.1)}$$

where

q : flow rate per unit joint length ($cm^3/s/cm$)

h : difference in the water level between the front and the rear of the sheet pile (cm)

K,n : constants

In cases of structures with a backfilling material, the characteristics of the backfilling material greatly affect the quantity of seepage through joints. In a part of the backfilling material near a sheet pile joint, there are areas to which Darcy's law cannot be applied. There has been an effort to evaluate the permeability in this part as a composite joint composed of a certain thickness of soil, including the backfilling material, and the sheet pile joint. This idea is effective for conducting the seepage flow analysis. Shoji et al. [14] proposed an empirical equation based on the comprehensive permeability tests considering both the difference in the degree of tensile force in joints and conditions with or without sand filling. The test results indicated that, for backfilled structures with joints filled with sand, the constant n could be approximated to 1.0 and the K value representing the results of the tests was derived.

(d) The degree of reduction in permeability as a result of sealing sheet pile joints against water varies depending on conditions such as the type and use of water sealant, and should be determined based on reliable data, such as results of tests conducted in consideration of the construction conditions at the site. According to results of field tests [15], there were cases in which the quantity of water seeping out of joints with water sealant applied was about 20% to 40% of that of joints with no water sealant applied.

② **Permeability of foundation ground**

(a) Permeability of natural ground

The permeability of the natural ground as a whole can be evaluated using the coefficients of permeability for each soil layer composing the natural ground. In calculating the coefficients of permeability for each soil layer, refer to **Part II, Chapter 3**, **2.2.3 Hydraulic Conductivity of Soil**. In ground which was formed by natural sedimentation, the coefficient of permeability displays directionality, and in many cases, it is larger in the horizontal direction than in the vertical direction. In a case where a structure is placed on the natural ground, the void ratio decreases because of the compression or consolidation of the ground, resulting in a decrease in the coefficient of permeability.

When evaluating the coefficient of permeability based on a laboratory test or Hazen's formula, it is important to accurately grasp the conditions of soil layers based on careful sampling.

(b) Permeability of improved ground

In cases where soil improvement is to be carried out as part of construction of a reclamation revetment, it is necessary to not only evaluate the permeability of the natural ground but also examine how the permeability will be changed by soil improvement.

In the soil between sand piles, in the ground below a replaced sand layer, and in parts of soil that have not been improved by using the deep mixing method, the coefficient of permeability decreases over a long period of time because of consolidation. In addition, sand piles may cause changes in the coefficient of permeability due to disturbance of clayey soil around them and due to clogging of the piles themselves.

When determining the coefficient of permeability of improved parts and non-improved parts of the foundation ground after soil improvement, it is necessary to conduct a well-balanced examination considering the simplification of compositions of the foundation ground and the revetment in the seepage flow analysis, the revetment structure, the accuracy of the coefficient of permeability of water sealing work, and other factors. It is also possible to examine approximate values obtained from a research about similar existing facilities.

(c) In case that the foundation ground is made of rocks, the permeability shall be determined based on thorough preliminary research, because the rock ground may contain cracks, fissures and/or fracture zones, which affect the permeability.[16]

(5) Verification of performance against Level 2 earthquake ground motions

① For the verification of performance against Level 2 earthquake ground motions, refer to **Part II, Chapter 5**, **2.2.4 Performance Verification for Deformation of Facilities during Earthquake** and **Reference (Part III), Chapter 1**, **2 Fundamentals of Seismic Response Analysis**.

② Because revetments have various shapes, it is necessary to set analysis conditions appropriate for the shape of the revetment to be analyzed. For example, gravity-type revetments and sheet pile revetments are considered to exhibit behaviors similar to those of gravity-type quaywalls and sheet pile quaywalls, respectively. Therefore, it can be considered that analysis programs that have been proved to be applicable to gravity-type quaywalls or sheet pile quaywalls are also applicable to gravity-type revetments or sheet pile revetments, respectively.

③ For revetments that are should be quickly inspected of earthquake resistance to prepare for tsunamis that might be triggered by trench earthquakes, such as Tonankai and Nankai earthquakes, a chart-type earthquake-resistance diagnosis system has been proposed. The system allows users to easily predict deformation, such as settlement, from a chart created based on many results of analyses with the Finite Element Analysis Program for Liquefaction Process (FLIP) and facility conditions input by users.[17)] It should be noted, however, that this technique was established for simplified diagnosis of deformation of elongated shore facilities and thus cannot be used for verification of the performance of revetments against Level 2 earthquake ground motions in principle.

14.6.2 Performance Verification of Gravity-type Revetments

(1) Structural types of gravity-type revetments include the caisson type, the L-block type, the cellular-block type, the type made of precast concrete members such as blocks, and the cast-in-place concrete type.

(2) There are common provisions applicable to all revetments and specific provisions applicable to each structural type of revetments. For the performance verification to be conducted in accordance with the former, refer to **Part III, Chapter 4**, **14.6 Performance Verification**. For the performance verification to be conducted in accordance with the latter, refer to **Part III, Chapter 5**, **2.2 Gravity-type Quaywalls** and **Part III, Chapter 5**, **2.11 Upright Wave-absorbing Type Quaywalls**.

14.6.3 Performance Verification of Sheet Pile Revetments

(1) Sheet pile revetments are composed of steel sheet piles, concrete sheet piles, or other sheet piles, and include cantilevered sheet pile revetments, sheet pile revetments with anchorage work, and double sheet pile revetments. It is difficult to start construction of a sheet pile revetment with anchorage work before reclamation has progressed to a certain stage, and it is necessary to manage construction of the revetment by checking the progress of reclamation and the stability conditions examined in advance.

(2) There are common provisions applicable to all revetments and specific provisions applicable to each structural type of revetments. For the performance verification to be conducted in accordance with the former, refer to **Part III, Chapter 4**, **14.6 Performance Verification**. For the performance verification to be conducted in accordance with the latter, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**, **Part III, Chapter 5**, **2.4 Cantilevered Sheet Pile Quaywalls**, **Part III, Chapter 5**, **2.5 Sheet Pile Quaywalls with Raking Pile Anchorages**, **Part III, Chapter 5**, **2.6 Open-type Quaywall with Sheet Pile Wall Anchored by Forward Batter Piles** and **Part III, Chapter 5**, **2.7 Double Sheet Pile Quaywalls**.

14.6.4 Performance Verification of Cellular-bulkhead Revetments

(1) This type of revetment has a cellular structure composed of steel sheet piles, steel plates, or other members. It has high watertightness, just like a steel sheet pile revetment; therefore, it is appropriate for prevention of leakage of landfill soil.

(2) There are common provisions applicable to all revetments and specific provisions applicable to each structural type of revetments. For the performance verification to be conducted in accordance with the former, refer to **Part III, Chapter 4**, **14.6 Performance Verification**. For the performance verification to be conducted in accordance with the latter, refer to **Part III, Chapter 5**, **2.9 Embedded-type Cellular-bulkhead Quaywalls** and **Part III, Chapter 5**, **2.10 Placement-type Cellular-bulkhead Quaywalls**.

14.6.5 Performance Verification of Rubble Mound Revetments

(1) This type of revetment is built in areas where water is relatively shallow, and has a body composed of rubble. It is necessary to take measures to prevent leakage of landfill soil. Shielding work shall be constructed on the front of the revetment to make it resistant to waves. In cases where rubble is available at low price, a rubble mound sloping revetment may be built in an area where water is considerably deep, in expectation of the effectiveness of rubble in purifying seawater and making fish swarm around it.

(2) For the performance verification of rubble mound revetments, refer to **Part III, Chapter 4**, **14.6 Performance Verification**, **Part III, Chapter 4**, **3.3 Gravity-type Breakwaters (Sloping Breakwaters)** and **Part III, Chapter 2**, **2.7 Armor Stone and Blocks**.

14.6.6 Performance Verification of Revetments Covered with Wave-dissipating Blocks

(1) This type of revetment is built in areas exposed to the intensive wave force and composed of the revetment body of each structural type with wave-dissipating work in front of it.

(2) For the performance verification of revetments covered with wave-dissipating blocks, refer to **Part III, Chapter 4**, **14.6 Performance Verification**, **Part III, Chapter 4**, **3.4 Gravity-type Breakwaters (Breakwaters Covered with Wave-dissipating Blocks)** and the descriptions about the performance verification of revetments of each structural type.

14.6.7 Revetments Serving as Facilities Prepared for Accidental Incidents

(1) For the performance verification of revetments serving as facilities prepared for accidental incidents, refer to the descriptions about seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(2) For evaluating the stability against the design tsunami and tsunamis with intensity higher than that of the design tsunami to develop the tsunami-resistant design, refer to the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors** [2], the **Design Concept for Parapets against Tsunamis (Provisional Edition)** [3] and the **Guidelines for Tsunami-Resistant Design of Breakwaters** [4].

14.6.8 Structural Details

(1) Revetments should be provided with scouring prevention work depending on wave conditions.

(2) Revetments shall be provided with appropriate leakage prevention work in consideration of the properties of landfill soil, the revetment structure, the residual water level, and other factors.

(3) If there is a possibility that sediment might flow or wash out of the ground behind a revetment because of the action of waves and others, it is necessary to take appropriate countermeasures by reference to **Part II, Chapter 2**, **6.5 Pressure of Waves Transmitted through Mound and Pressure of Waves in Joints that Act Behind a Revetment** and other references.

(4) Revetments shall be provided with stairs and other ancillary facilities, as necessary.

(5) Revetments may be provided with a parapet to reduce wave overtopping.

(6) If there is a possibility that waves might overtop a revetment, it is necessary to construct apron work to protect the area behind the revetment. The width of the apron work shall be determined in consideration of the wave overtopping rate, the wave run-up height, the structural type of the revetment, and other factors. In addition, drainage ditches, drainage holes, and other appropriate drainage facilities shall be provided to eliminate seawater that has gone beyond the revetment because of wave overtopping. The cross-sectional areas of these drainage facilities shall be determined appropriately in consideration of the wave overtopping rate, the rainfall, and other conditions.

(7) For other structural details, refer to **Part III, Chapter 4**, **Protective Facilities for Harbors** and **Part III, Chapter 5**, **Mooring Facilities**.

14.7 Symbiotic Revetments

(1) As revetments contributing to maintenance of good environments of ports and harbors, symbiotic revetments [18] are designed to allow living beings to grow in mudflats, rocky shores, and other areas in the ports and harbors according to environmental conditions to which the revetments are subjected (**References (Part I), Chapter 3**, **2 Symbiotic Port Facilities**). Existing revetments may be improved to be symbiotic revetments by adding a function that allows living beings to grow.

(2) Factors that affect the capability of a symbiotic revetment to fulfill the purpose to allow living beings to grow (**Reference (Part I)**, **Chapter 3**, **2 Symbiotic Port Facilities**) shall be clarified through environmental research,

numerical modeling, and other techniques. In the performance verification of the revetment, it shall be verified that its structure, cross section, and ancillary facilities are appropriate for the revetment to fulfill its purpose.

(3) The performance requirement for symbiotic revetments is that they shall have a function that allows living things to grow. Factors (dominating actions) that affect the function include the presence or absence of a foundation for living things to grow, external forces such as waves and currents, and an environment necessary for living things to grow. Conditions of the environment necessary for living things to grow include the water depth and underwater visibility that affect the light intensity necessary for photosynthesis and the water temperature that affects the activity of living things. Specifically, for a revetment intended to form seaweed beds, it is necessary that the structure and cross section of the revetment and the foundation and inclination of its ancillary facilities be appropriate for desired seaweed and seagrass to attach to the revetment. It is also necessary that the revetment and its ancillary facilities not block sunlight to such an extent as to affect the light intensity necessary for seaweed and seagrass to grow.

(4) The performance verification of a symbiotic revetment shall be conducted by confirming, based on available knowledge, that the environment of the place where symbiosis of living things is desired is within the range of conditions under which desired living things can grow. For example, for revetments intended to form seaweed beds, the light intensity and water temperature that affect photosynthesis and breathing of seaweed are considered as environmental conditions to be considered in the performance verification. Therefore, in the performance verification of those revetments, it shall be verified that these environmental conditions are within the range of conditions that allow formation of desired seaweed beds. If it is possible to predict changes in the environmental conditions after construction of a symbiotic revetment, environmental changes in the future, and other future possibilities, techniques such as numerical modeling concerning growth of living things may be used for verifying that the environmental conditions are within the range of conditions under which living things can grow.

(5) For the performance verification of symbiotic revetments, refer to **Part III, Chapter 4**, **4 Symbiotic Breakwaters**, **Reference (Part I)**, **Chapter 3**, **2 Symbiotic Port Facilities** and the **Guidelines for Development and Maintenance of Symbiotic Port Facilities** [18].

(6) Amenity functions may be added to symbiotic functions to make an environmentally friendly revetment that has synergistic effects of those functions.

14.8 Revetments Having Amenity Functions

(1) For the performance verification of amenity-oriented revetments, refer to reference 19).

(2) Provisions about the performance verification of revetments of each structural type may be applied to the performance verification of amenity-oriented revetments.

(3) It is preferable that a revetment to be constructed in a green area having a waterfront line should be designed as an amenity-oriented revetment[20] and have additional functions that allow users to look at the sea, get close to the sea, and get familiar with the sea.

(4) Amenity functions, such as fishing facilities, may be added to a revetment to make it a multipurpose revetment.[21]

(5) An amenity-oriented revetment shall have the cross section determined in consideration of the risk that users might fall into the sea, and shall be provided with fall prevention fences and other appropriate ancillary facilities, as needed.

(6) In cases where a facility has an area that is available for people to walk in normal times and likely to be exposed to overtopping waves when waves are high, it is necessary to alert people to the danger of wave overtopping by setting up a sign or taking other appropriate means.

(7) Walkways and slopes of revetments shall have widths, pitches, and other dimensions that allow elderly users and physically disabled users, including those in wheelchairs, to move safely.[22, 23, 24]

(8) Amenity-oriented functions of a revetment may be enhanced with consideration for inhabitation of living things (**References (Part I)**, **Chapter 3**, **2 Symbiotic Port Facilities**).

[References]

1) National Association of Agricultural Sea Coast, National Association of Fisheries Infrastructure, National Association of Sea Coast, Ports and Harbours Association of Japan: Technical Standards and Commentary for shore Protection Facilities, 2018 (in Japanese)
2) MLIT, Ports and Harbours Bureau: Guidelines for Earthquake-resistant Design of Storm Surge Barriers (Breast Walls) in Ports, 2013 (in Japanese)
3) Fisheries Agency, Fishery Infrastructure Department, Fishing Communities Promotion and Disaster Prevention Division and MLIT, Ports and Harbours Bureau, Coast Administration and Disaster Prevention Division: Design Concept of Breast Walls for Tsunami (Tentative Edition), 2015 (in Japanese)
4) MLIT, Ports and Harbours Bureau: Guideline for Tsunami-Resistant Design of Breakwaters (Partial Revision), 2015 (in Japanese)
5) Coastal Development Institute of Technology: Technical Manual for Flap-type Land Gates at Ports and Coasts, 2016 (in Japanese)
6) Nagao, T., M. Fujimura and Y. Moriya: A Study on the Setting of the Crown Height for Revetments Considering the Estimation Accuracy of Wave Overtopping Quantity, Journals of JSCE Division B3 (Ocean Engineering), Vol.21, pp.773-778, 2005 (in Japanese)
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21) MLIT, Ports and Harbours Bureau: Guidelines for the Multipurpose Use of Breakwaters (Second Edition) (Draft), 2017 (in Japanese)
22) Transport Economy Research Center: Guideline of the facilities for elderly and handicapped people in public transport terminal, 1994
23) Yoshimura, M. and K. Ueshima: Study on Barrier Free Design in External Spaces-Issues and Solutions in Port and Coastal Spaces, Research Report of National Institute of Land and Infrastructure Management No.6, 2003 (in Japanese)
24) MLIT: Policy Outline for Universal Design, 2005 (in Japanese)

15 Coastal Dikes

[Ministerial Ordinance] (Performance Requirements for Coastal Dikes)

Article 21

The provisions of Article 16 shall apply mutatis mutandis to the performance requirements for coastal dikes.

[Public Notice] (Performance Criteria of Coastal Dikes)

Article 44

The provisions of Article 39 shall apply mutatis mutandis to the performance criteria for coastal dikes.

[Interpretation]

10. Protective Facilities for Harbors

(12) Performance criteria of coastal dikes (Article 21 of the Ministerial Ordinance and the interpretation related to Article 44 of the Public Notice)

Performance criteria of seawalls and their interpretation shall be applied correspondingly to coastal dikes.

15.1 General

(1) The purpose of coastal dikes is to protect the land areas behind them from waves, storm surges, and tsunamis.

(2) For the performance requirements and performance criteria of coastal dikes, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(3) Coastal dikes to which damage might significantly affect human lives, properties, and/or socioeconomic activities and which are required to protect the land areas behind them from the design tsunami and accidental waves shall be built to protect the land areas behind them from the actions concerned, in addition to the purpose mentioned above in (1).

(4) Coastal dikes shall be designed in accordance with this document and by reference to the **Technical Standards and Commentary for Shore Protection Facilities** [1)], the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors** [2)], the **Design Concept for Parapets against Tsunamis (Provisional Edition)** [3)], the **Guidelines for Tsunami-Resistant Design of Breakwaters** [4)], the **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores** [5)] and references 6) and 7). If the facility of interest is a shore protection facility, it shall conform to the **Technical Standards for Shore Protection Facilities**.

15.2 Items to be Considered in Setting of Basic Cross Section

For items to be considered in setting the basic cross section of a coastal dike, refer to **Part III, Chapter 4**, **14.2 Items to be Considered in Setting of Basic Cross Section**.

15.3 Setting of Crown Height of Coastal Dike

For setting the crown height of a coastal dike, refer to **Part III, Chapter 4**, **14.4 Setting of Crown Height of Revetment**.

15.4 Actions

For setting actions to be considered in the performance verification of coastal dikes, refer to **Part III, Chapter 4**, **14.5 Actions**.

15.5 Performance Verification

15.5.1 Performance Verification

For the performance verification of coastal dikes, refer to **Part III, Chapter 4**, **14.6 Performance Verification**.

15.5.2 Coastal Dikes Serving as Facilities Prepared for Accidental Incidents

For the performance verification of coastal dikes serving as facilities prepared for accidental incidents, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**).

[References]

1) National Association of Agricultural Sea Coast, National Association of Fisheries Infrastructure, National Association of Sea Coast, Ports and Harbours Association of Japan: Technical Standards and Commentary for shore Protection Facilities, 2018 (in Japanese)

2) MLIT, Ports and Harbours Bureau: Guidelines for Earthquake-resistant Design of Storm Surge Barriers (Breast Walls) in Ports, 2013 (in Japanese)

3) Fisheries Agency, Fishery Infrastructure Department, Fishing Communities Promotion and Disaster Prevention Division and MLIT, Ports and Harbours Bureau, Coast Administration and Disaster Prevention Division: Design Concept of Breast Walls for Tsunami (Tentative Edition), 2015 (in Japanese)

4) MLIT, Ports and Harbours Bureau: Guideline for Tsunami-Resistant Design of Breakwaters (Partial Revision), 2015 (in Japanese)(http://www.nilim.go.jp/lab/bcg/sokuhou/.le/120514.pdf).

5) Coastal Development Institute of Technology: Technical Manual for Flap-type Land Gates at Ports and Coasts, 2016 (in Japanese)(http://www.nilim.go.jp/lab/fcg/labo/report_ver2.pdf).

16 Jetties

[Ministerial Ordinance] (Performance Requirements for Jetties)

Article 22

1 The performance requirements for jetties shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied for the purpose of mitigating the influence of littoral drift through effective control of littoral drift.

2 The provisions of Article 14, paragraph (1), item (ii) shall apply mutatis mutandis to the performance requirements for jetties.

[Public Notice] (Performance Criteria of Jetties)

Article 45

The provisions of Article 38 shall apply mutatis mutandis to the performance criteria for jetties.

[Interpretation]

10. Protective Facilities for Harbors

(13) Performance Criteria of Jetties (Article 22 of the Ministerial Ordinance and the interpretation related to Article 45 of the Public Notice)

① Performance criteria of sediment control jetties and their interpretations shall be applied correspondingly to jetties.

② In the performance verifications of jetties, appropriate consideration shall be given to the effects of an increase in the earth pressure due to sedimentation of littoral drift and the effects of waves and river currents, as needed, in addition to the previous paragraph.

③ In setting the layout and dimensions of a jetty, appropriate consideration shall be given to the predominant direction of waves and water currents, topography, expected conditions of use of the jetty, the impact on the natural environment and other factors in order to ensure that the jetty has serviceability.

④ The layout of jetties includes the positions where they are built as well as their directions and the spacing between them. The dimensions of jetties include their structures, crown heights, crown widths, and lengths. In determining the layout of jetties, attention shall be paid to the fact that construction of jetties may cause excessive reduction in longshore sediment transport and thus increase the possibility of shoreline retreat on the surrounding coast.

(1) For the performance verification of jetties, refer to **Part III, Chapter 4**, **3 Breakwaters Having Basic Functions**, with modifications made as necessary in consideration of the structural type. Note that it is necessary to provide appropriate consideration to the effects of an increase in the earth pressure because of sedimentation of littoral drift and the effects of scouring caused by waves and river currents.

(2) For the lengths, spacing, structures, and other details of jetties to be constructed on the updrift side of a port for the purpose of siltation prevention, refer to **Part III, Chapter 4**, **9 Sediment Control Jetties** and the **Technical Standards and Commentary of Shore Protection Facilities** [1].

[References]

1) Technical Committee for Coastal Protection Facilities: Technical standards and commentary of coastal protection facilities, Japan Port Association, pp.3-77-3-85, 2004

17 Parapets

[Ministerial Ordinance] (Performance Requirements for Parapets)

Article 23

The provisions of Article 16 shall apply mutatis mutandis to the performance requirements for parapets.

[Public Notice] (Performance Criteria of Parapets)

Article 46

The provisions of Article 39 shall apply mutatis mutandis to the performance criteria for parapets.

[Interpretation]

10. Protective Facilities for Harbors

(14) Performance Criteria of Parapets (Article 23 of the Ministerial Ordinance and the interpretation related to Article 46 of the Public Notice)

Performance criteria of seawalls and their interpretations shall be applied correspondingly to parapets.

17.1 General

(1) The purpose of parapets is to protect the land areas behind them from waves, storm surges, and tsunamis.

(2) Parapets are structures built on the landward side of a shoreline in cases where there is a fishing port, a port, a harbor, or the like along the shoreline and it is difficult to build coastal dikes, revetments, and similar structures that might interfere with the use of the port or harbor. **Fig. 17.1.1** shows the conceptual diagram of a parapet.

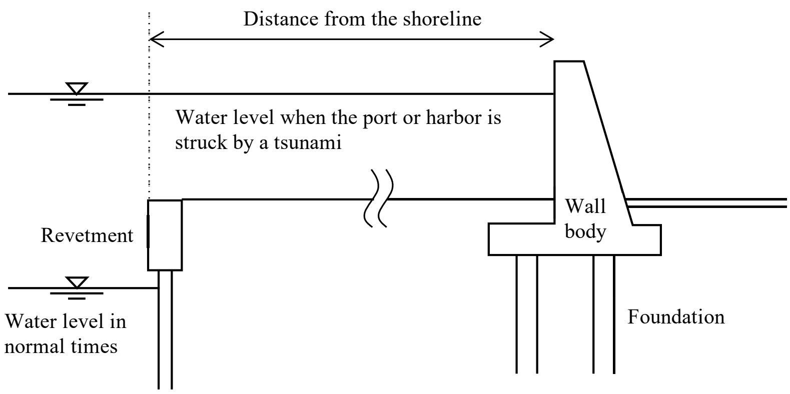

Fig. 17.1.1 Conceptual Diagram of a Parapet

(3) For the performance requirements and performance criteria of parapets, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(4) Parapets to which damage might significantly affect human lives, properties, and/or socioeconomic activities and which are required to protect the land areas behind them from the design tsunami and accidental waves shall be built to protect the land areas behind them from the actions concerned, in addition to the purpose mentioned above in (1).

(5) Parapets shall be designed in accordance with this document and by reference to the **Technical Standards and Commentary for Shore Protection Facilities** [1], the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors** [2], the **Design Concept for Parapets against Tsunamis (Provisional Edition)** [3], the **Guidelines for Tsunami-Resistant Design of Breakwaters** [4] and the **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores** [5]. If the facility of interest is a shore protection facility, it shall conform to the **Technical Standards for Shore Protection Facilities**.

17.2 Items to be Considered in Setting of Basic Cross Section

For items to be considered in setting the basic cross section of a parapet, refer to **Part III, Chapter 4**, **14.2 Items to be Considered in Setting of Basic Cross Section**.

17.3 Setting of Crown Height of Parapet

For setting the crown height of a parapet, refer to Part III, Chapter 4, 14.4 Setting of Crown Height of Revetment.

17.4 Actions

For setting actions to be considered in the performance verification of parapets, refer to **Part III, Chapter 4**, **14.5 Actions**.

17.5 Performance Verification

17.5.1 Performance Verification

(1) General

For the performance verification of parapets, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**) and the descriptions about revetments (**Part III, Chapter 4**, **14.6 Performance Verification**).

(2) Performance verification of parapets under the variable situation associated with Level 1 earthquake ground motions

In the performance verification of a parapet under the variable situation associated with Level 1 earthquake ground motions, it is possible to use the seismic coefficient for verification obtained by determining the natural period of the parapet through a frame-structure analysis or other technique and calculating the seismic coefficient for verification from this natural period and the acceleration response spectrum.[6), 7)] This is based on the rule that the performance should be evaluated under the condition that the parapet is not affected by deformation or other failure of the foundation ground. Therefore, it is necessary to undertake measures to ensure that the parapet will not be affected by factors such as a lateral flow because of deformation of a revetment. In cases where the ground may have liquefied because of earthquake ground motions, it is necessary to assess whether or not the ground has liquefied and appropriately consider measures against liquefaction by reference to **Part II, Chapter 7**, **Ground Liquefaction**.

An earthquake ground motion to be considered in the performance verification of earthquake-resistance shall be set by considering the effects of the surface ground through the calculation of seismic response of the ground. It is necessary to use the seismic response analysis code that allows appropriate evaluation of the amplification of the earthquake ground motion in soft ground. (Refer to **Part II, Chapter 6**, **1.2.3 Calculation of Seismic Response of Surface Ground**.) The time history of acceleration on the ground surface shall be calculated through the one-dimensional seismic response analysis as described in **Part II, Chapter 6**, **1.2.3 Calculation of Seismic Response of Surface Ground** using the time history of acceleration of the earthquake ground motion set for the engineering bedrock as the input earthquake ground motion. The acceleration response spectrum can be obtained from the calculated acceleration time history and used for calculating the response acceleration for the natural period of the parapet, and the calculated response acceleration can be divided by the acceleration of gravity to obtain the characteristic value of the seismic coefficient for verification. The damping constant for calculating the acceleration response spectrum may be assumed to be 0.4. **Fig. 17.5.1** shows an example of an ordinary procedure for setting the seismic coefficient for verification.

The natural period for a parapet with pile foundation can be calculated through a frame-structure analysis as described in **Part III, Chapter 5**, **5.2.3 (14) Earthquake Ground Motion to be Considered in Performance Verification of Earthquake-resistance**. The frame-structure of the parapet shall be modeled taking account of the ground spring that expresses the subgrade reaction in a part for which the resistance caused by ground deformation is considered. As for the wall body, the horizontal beam element corresponding to the footing and the vertical beam element corresponding to the part of the wall body above ground shall be used in the analysis. The natural period of the parapet can be calculated by placing a node corresponding to the center of gravity of the wall body, making a horizontal load act there, determining the spring constant for the entire parapet based on the relation with horizontal displacement under a very small load, and taking account of the weight of the wall body.

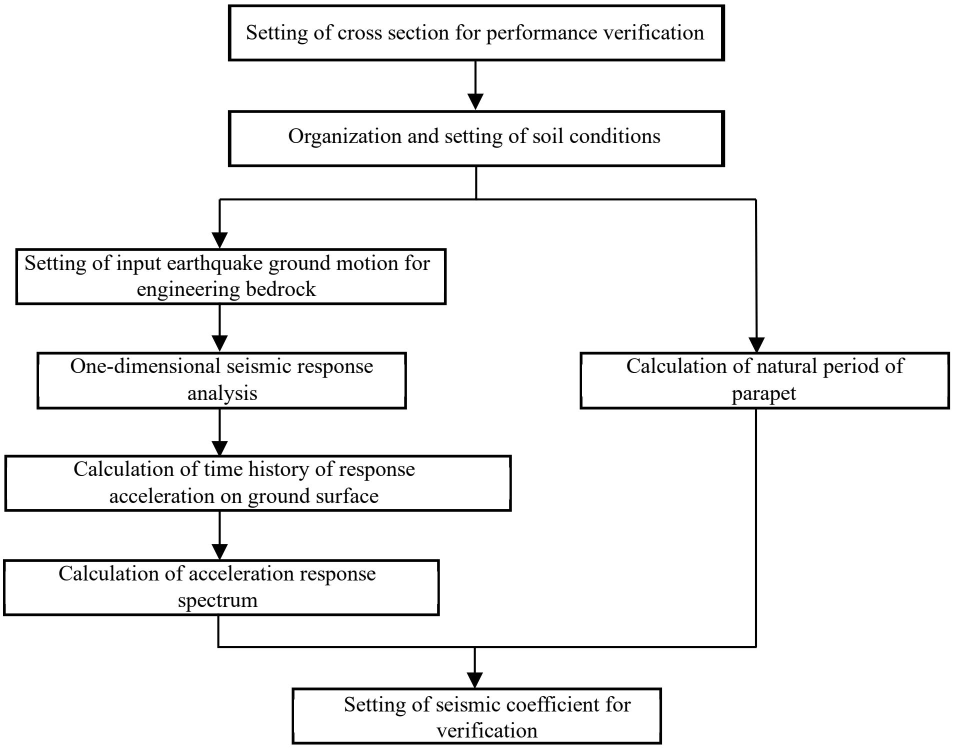

Fig. 17.5.1 Ordinary Procedure for Setting Seismic Coefficient for Verification of a Parapet

17.5.2 Parapets Serving as Facilities Prepared for Accidental Incidents

For the performance verification of parapets serving as facilities prepared for accidental incidents, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**).

[References]

1) Japan Port Association (JPA): Technical standards and commentaries for shore protection facilities, 2018.

2) Port and Harbour Bureau, Ministry of Land Infrastructure, Transport and Truism (MLIT): Guideline for tsunami-resistant design for parapet-type seawalls , 2013.

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7) H. Fukawa and E. Kohama: A study on the applicability of frame analysis and response spectrum method to evaluate seismic coefficients of coastal parapet levees with pile foundations, Journal of JSCE A1, Vol.73, No.4, pp.I-431-I-442, 2017.

18 Siltation Prevention Facilities

18.1 General

(1) In cases where siltation of harbors and waterways is expected, the mode of siltation needs to be understood on the basis of an adequate investigation of the phenomena of the potential causes of siltation and appropriate countermeasures should be taken considering the various types of effects caused by siltation prevention works, safe navigation of ships, economy, and so forth.

(2) Causes of Siltation

Siltation is a phenomenon where littoral drift, windblown sand, discharged sediments through river, and so on invade harbor water areas, such as waterways and basins, and settle on to the sea bottom of the area, hindering port functions with the water depth to be shallow. There are cases where the water depth becomes shallower than the required depth without any substantial change of sediment volume, such as the formation of a sand wave[1] or collapse of side slopes of dredged waterways. The causes of siltation are listed below:

① Invasion and accumulation of littoral drift (mainly caused by waves and wave induced current)

② Settling and accumulation of river discharged sediments

③ Plunging and deposition of windblown sand

④ Movement of sediments within the objective area and change in the location of deposition

⑤ Movement of sediments because of disturbances in the harbor, collapse of slopes in waterways, and formation of sand waves

(3) Modes of Siltation

The modes of siltation in water areas surrounded by breakwater and such can be classified as shown in **Fig. 18.1.1** according to the source of sediment, courses and modes of sediment invasion, and deposition processes. Siltation in open water areas, such as off-the-harbor waterways, has, for example, the following various modes:

① Siltation often accompanies scouring of a neighboring area, as shown in **Fig. 18.1.2 (a)**, in relatively shallowly dredged waterways and others if waves are dominating action and the sea bottom is sandy.

② Relatively equal siltation including the side slopes in waterways often occurs as shown in **Fig. 18.1.2 (b)** in relatively shallowly dredged waterways where the bottom sediment is soft mud with silt and clay.

③ The siltation rate is the higher at the bottom of the waterway as shown in **Fig. 18.1.2 (c)** in the waterways deeply dredged from the surrounding sea bottom.

④ Waterways dredged by cutting a natural sand bar in straits and others tend to silt up so that the bar topography is restored.

⑤ When dredging the sea bottom where a sand wave naturally exists, the sand wave tends to restore the sea bottom topography.

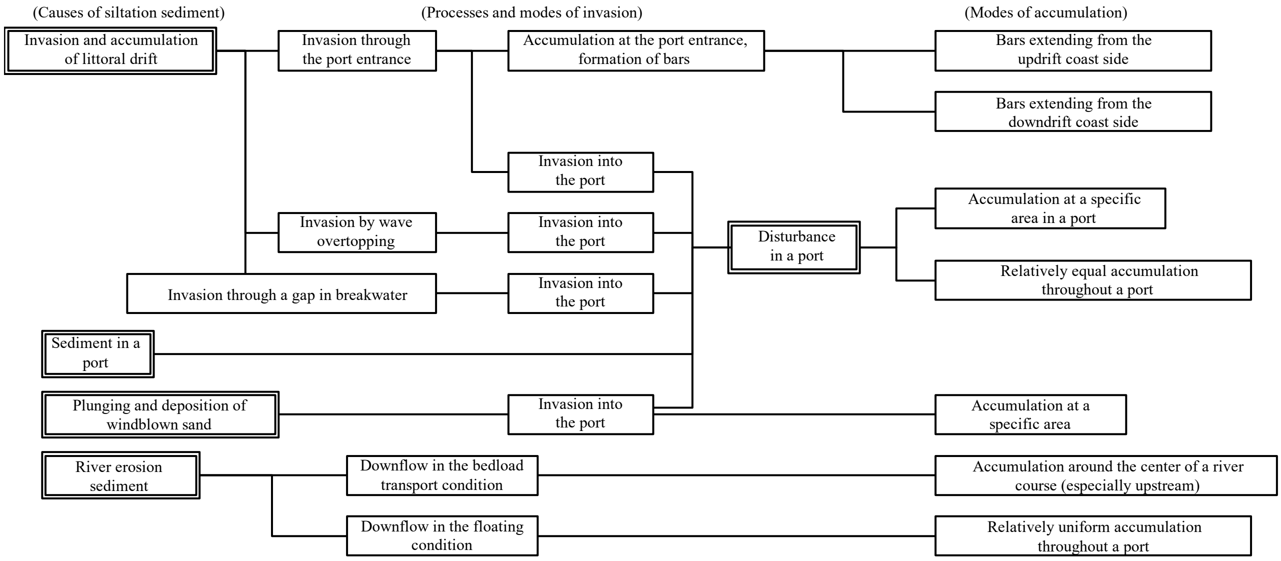

Fig. 18.1.1 Modes of Siltation in Harbor Water Areas

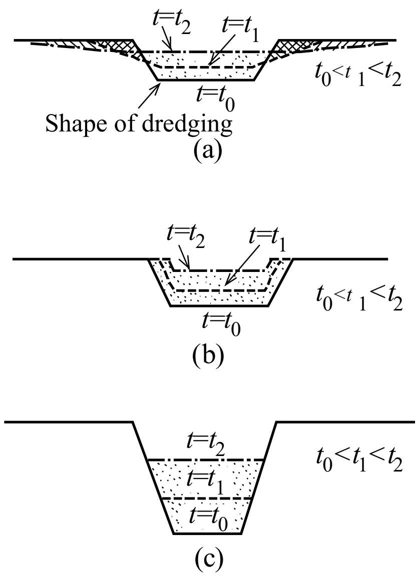

Fig. 18.1.2 Modes of Siltation in Waterways (Where t Denotes the Elapsed Time)

(4) Types of Siltation Prevention Measure Works

The siltation prevention measure works are listed below:

① To prevent siltation by building breakwater and such, as shown in **Table 18.1.1**.

② To effectively trap sediments by outbreak, pocket dredging upstream of an estuary harbor, or other measures and to perform its maintenance dredging.

③ To perform maintenance dredging.

Table 18.1.1 Facilities Used as Semi-permanent Siltation Prevention Measure Works

Countermeasure against longshore sediment transport	Prevention of invasion through the port entrance	Breakwater, jetty (training jetty)
	Prevention of invasion by wave overtopping	Raising of breakwaters
	Prevention of sediment transparent invasion	Sediment infiltration prevention work
River erosion control facility	Increase in the river bedload transport power	Training jetty
	Prevention of erosion sediment invasion	Separation levee
	Reduction of erosion sediment	Division works
Windblown sand prevention work	Reduction of windblown sand	Afforestation, windblown sand control forest
	Prevention of windblown sand invasion	Sand invasion prevention fence, among others

Among those measure works, sediment invasion prevention measure works by laying structures may include building submerged breakwaters by driving sheet panels as well as concrete blocks.

(5) Selection of Siltation Prevention Measure Works

Since the concept of siltation prevention measure works differs for each measure, the most appropriate measure will be selected. When selecting a siltation prevention measure work, it should be determined by adequately investigating the actual conditions and mechanisms of siltation, thoroughly considering the influence on the surrounding environment, and referring to past practical examples, among others. It is also beneficial to perform a study using a movable bed model experiment.

18.2 Facilities for Trapping Littoral Drift or River Erosion Sediment

(1) When the aim is to prevent siltation due to longshore littoral drift by means of maintenance dredging, an appropriate facility to trap the sand should be built accordingly at a proper location, at which the facility can trap and prevent littoral drift from invading waterways and basins. This facility should take measures to improve the wave conditions when dredging and increase the dredging efficiency.

It is preferable to fully study and determine the type and layout, among other criteria, of these longshore sediment transport trap facilities by considering their capability to trap littoral drift, the dredging conditions, the economy, and so forth.

(2) Facilities to Trap Littoral Drift

As a method for trapping longshore littoral drift, provisions to limit sand deposition areas are commonly employed in various countries, by means of building detached breakwaters or partially reducing the crown height of updrift breakwaters. Besides, countermeasures against local siltation by a sand bar's restoration action in open-cut waterways crossing a sand bar in the sea floor of straits and other structures and pocket dredging considered as a measure against siltation due to, for example, river discharged sediment, can be considered to be trapping facilities of littoral drift.

(3) Proper Positioning of Littoral Drift Trapping Facilities

Littoral drift trapping facilities can be installed at areas where sediment deposition occurs easily under natural conditions, as shown in **Fig. 18.2.1 (a)**, **(b)**, and **(c)**, and artificial conditions can be created to encourage sediments to settle out of flows with a high concentration of littoral drift, as shown in **Fig. 18.2.1 (d)**, **(e)**, and **(f)**. In order to identify such specific locations and capture littoral drift in the most efficient manner, an adequate understanding of the moving condition and mechanism of littoral drift is indispensable. Furthermore, when selecting the positions for littoral drift trapping facilities, in addition to the littoral drift trapping efficiency, in cases where trapped sediments are dredged, it is preferable to give adequate consideration to dredging conditions, in other words, to easily maintain the water depth necessary for the navigation of dredgers and to keep calm conditions during dredging works, among other goals.

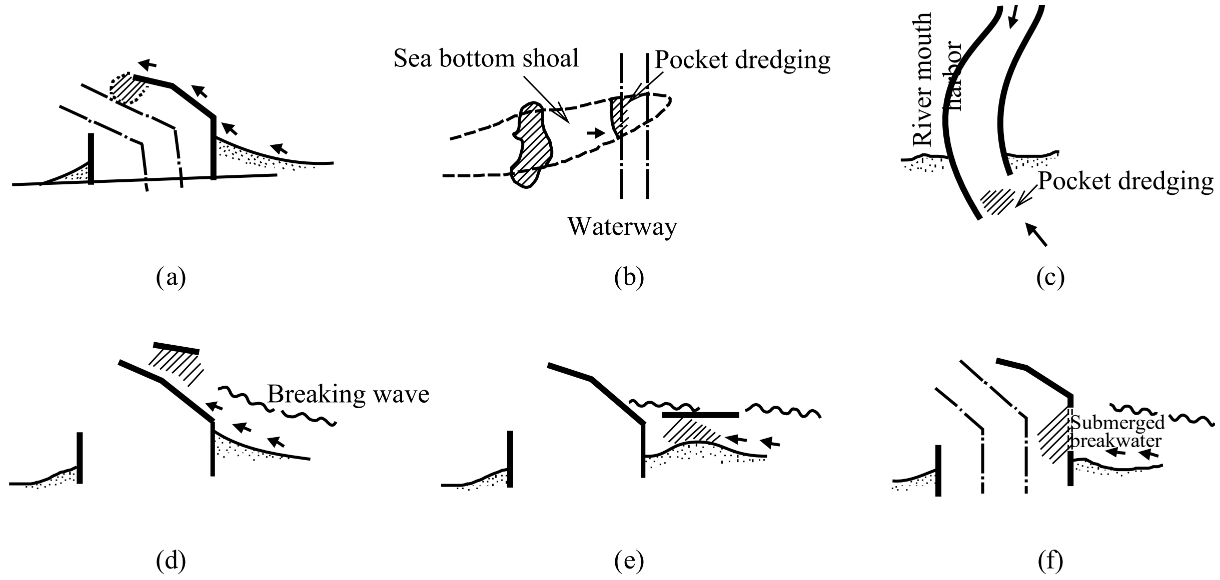

Fig. 18.2.1 Positioning of the Trapping Facility of Littoral Drift

(4) Size of the Littoral Drift Trapping Facility

The size of the littoral drift trapping facility generally depends on the volume of trapped sediments and physical conditions required for settlement and accumulation of littoral drift. The required conditions for settlement and accumulation of littoral drift are determined by the result of field measurements, past performance, movable bed model experiment, and so forth.

18.3 Windblown Sand Prevention Work

18.3.1 General

(1) When windblown sand becomes a concern for siltation of ports and waterways or for the environmental preservation of surrounding areas, adequate means shall be taken to prevent windblown sand depending on the situation.

(2) Windblown sand, that is, sand that is moved by winds, is carried into harbors or waterways where it settles and deposits, causing siltation. In some cases, it also accumulates on road surfaces or gets dispersed into residential areas as dust, disrupting the daily living of the residents. In particular, there are many instances where open digging of dunes or land reclamation causes problems related to windblown sand, and thorough countermeasures need to be prepared in advance.

18.3.2 Selection of a Work Method

(1) The windblown sand prevention work method is determined by deeply understanding the characteristics of each work method after adequately investigating and studying the current situation of the windblown sand and its expected situation in the future.

(2) The windblown sand phenomenon depends on natural conditions, such as the wind direction and wind velocity, and the characteristics of the sediment (grain size distribution and degree of ground humidity), and these determine characteristics such as the direction, amount, and distribution of the windblown sand. When taking some countermeasures against windblown sand, there is a need to investigate these characteristics and to select a proper method considering the nature of the problems concerning the windblown sand, land use plan of the windblown-sand-prone area, or social conditions such as the economy.

(3) The following windblown sand prevention work methods are generally used.

① **Sediment trapping works and windbreaker fences**

Traditionally, a method where multiple rows of low (about 1 m high) sand fences are built to trap windblown sand, thus growing artificial sand dunes to enhance the effectiveness of sand breaking, is used. In some cases, relatively tall windbreaker fences can be built around to prevent the blowing of sediments and other particles from around the reclaimed ground or the powder stockyard.

② **Sand retaining fences**

A low sand control hedge is built to improve the surface roughness, weaken the wind shear force on the ground level, and reduce the surface sand.

③ **Shielding works**

The sand surface is shielded with artificial materials to reduce the movement of sand.

④ **Afforestation works**

Adequate plants are grown on the sand surface to shield the sand surface. This may be considered to be a kind of shielding method.

⑤ **Plantation works**

Trees are planted on the leeward direction of the windblown sand area to prevent windblown sand.

(4) Windblown sand prevention works conducted from the standpoint of the so-called coastal sand erosion control intended to stabilize sandy seashores normally combine several works. These procedures and works include those indicated in **Fig. 18.3.1**. Refer to Reference 2) for details. For trees suitable for coasts, refer to the **Civil Engineering Handbook**.[3)]

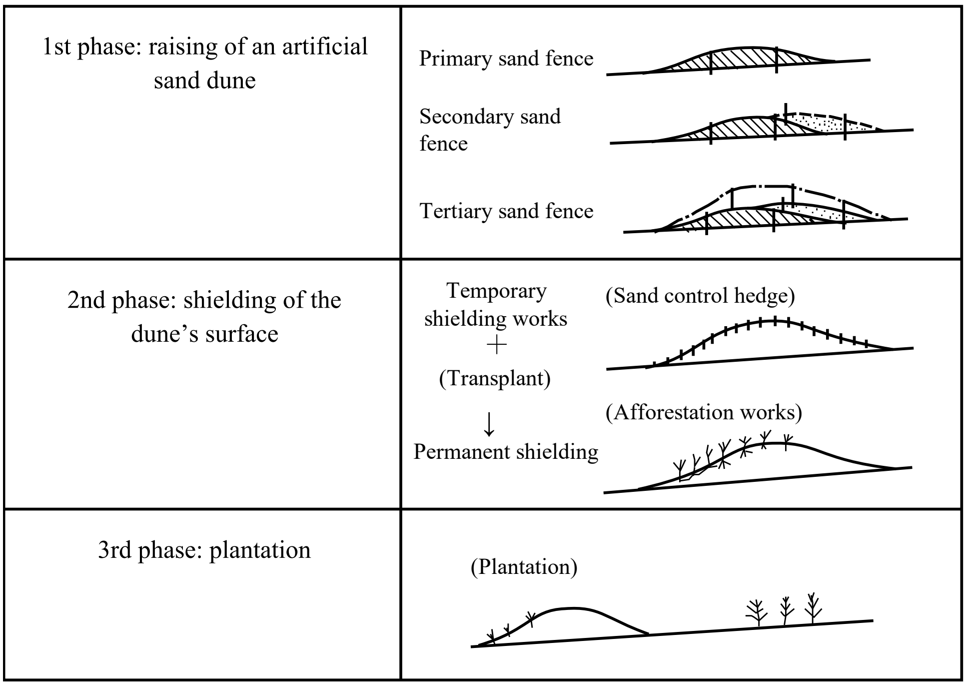

Fig. 18.3.1 Procedure for Artificial Sand Dune Raising Work

[References]

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2) Tanaka, K., Y. Nakajima, H. Endou and E. Kinnai: Sabo at coast (Coastal erosion control), Sabo Science, Compendium of Sabo Series, III-9, Japan Society of Erosion Control Engineers, Ishibashi-shoten Publishing,1985 (in Japanese)

3) JSCE, Civil Engineering Handbook, Vol. II, pp. 2135-2136, 1989 (in Japanese)

Chapter 5 Mooring Facilities

1 General

[Ministerial Ordinance] (General Provisions)

Article 25

Mooring facilities shall be installed in appropriate locations in light of geotechnical characteristics, meteorological characteristics, sea states, and other environmental conditions, as well as ship navigation and other usage conditions of the water area around the facilities so as to secure the safe and smooth usage by ships.

[Ministerial Ordinance] (Necessary Items concerning Mooring Facilities)

Article 34

The items necessary for the performance requirements of mooring facilities as specified in this Chapter by the Minister of Land, Infrastructure, Transport and Tourism and other requirements shall be provided by the Public Notice.

[Public Notice] (Mooring Facilities)

Article 47

The items to be specified by the Public Notice under Article 34 of the Ministerial Ordinance concerning the performance requirements of mooring facilities shall be as provided in the following Article to Article 74.

1.1 Purpose of Mooring Facilities

The purpose of installing mooring facilities is to ensure the safety and smoothness of the mooring and landing operation of ships, the embarkation and disembarkation of passengers, and the loading and unloading of cargo.

1.2 General

(1) Mooring facilities include quaywalls, piers, lighter's wharves, floating piers, docks, mooring buoys, mooring piles, dolphins, detached piers, and air-cushion-craft landing facilities. Among quaywalls, piers, and lighter's wharves, facilities that are particularly important from the viewpoint of earthquake preparedness and require the strengthening of earthquake-resistant performance are considered high earthquake-resistant facilities are classified as high earthquake-resistant facilities (specially designated emergency supply transport), high earthquake-resistance facilities (specially designated trunk line cargo transport), and high earthquake-resistance facilities (standard emergency supply transport), which correspond to the functions required in the objective facilities after the action of ground motion.

(2) The structural types of mooring facilities shall be determined by taking into consideration natural conditions, use conditions, construction conditions, and economic efficiency. The structural types of mooring facilities are classified into gravity-type quaywalls, sheet pile quaywalls, cantilevered sheet pile quaywalls, double sheet pile quaywalls, quaywalls with relieving platforms, embedded-type cellular-bulkhead quaywalls, placement-type cellular-bulkhead quaywalls, open-type wharves on vertical piles, open-type wharves on coupled raking piles, and jacket piers.

1.3 Dimensions and Layout of Mooring Facilities

(1) The dimensions of mooring facilities are preferably determined on the basis of actual circumstances, including the number and type of cargoes and passengers utilizing the port; packing type; marine and land transportation; and other relevant factors, with due consideration given to future trends in cargo and passenger volumes, increase in vessel size, changes in transportation systems, etc.

(2) The layout of mooring facilities is preferably determined to facilitate the berthing and unberthing of ships with due consideration given to oceanographic, topographic, and subsoil conditions and to identify the relationships of land transport network systems and land use in the hinterland. In particular, the locations of the following mooring facilities shall be determined in accordance with the provisions in respective items.

① Mooring facilities for passenger ships: They should be isolated from areas where hazardous cargoes are handled and should be provided with sufficient areas of land for passenger-related facilities such as waiting rooms and parking lots in the vicinity of the facilities.

② Mooring facilities used by vessels loaded with hazardous cargoes: They should be isolated from facilities for which a healthy living environment needs to be preserved (e.g., houses, schools, and hospitals), and kept at a predetermined safe distance from other mooring facilities and sailing vessels. In addition, measures should be in place to quickly respond to incidents such as hazardous material spills.

③ Mooring facilities that are accommodating ships or cargo-handling machines that are generating considerable noise: They should be isolated from facilities for which a healthy living environment needs to be preserved (e.g., houses, schools, and hospitals).

④ Mooring facilities for ships loaded with cargoes that may generate significant dust or offensive odors while being handled: They should be isolated from facilities for which a healthy living environment needs to be preserved (e.g., houses, schools, and hospitals).

⑤ Offshore mooring facilities: They should be in a location that does not hinder the navigation or anchorage of vessels.

⑥ High earthquake-resistant facilities and large-scale mooring facilities: They should be located in areas with good ground conditions that do not amplify earthquake motions as much as possible to avoid huge investments in possible ground improvement depending on ground conditions and earthquake motion amplification characteristics. Refer to [Part II], Chapter 1, 4.6 Application of Microtremor Observation Results to Port Planning for the explanation of the method for simply estimating the distribution of earthquake motion amplification characteristics inside ports by utilizing the microtremor observation results and by applying the estimated distribution to port planning.

⑦ Facilities and high earthquake-resistant facilities that may have serious effects on human lives, property, and socioeconomic activities when damaged by disasters: When located close to inland active faults as hypocenters, these facilities should be installed with their face line directions arranged perpendicular to earthquake source faults. Regarding the occurrence of earthquakes with inland active faults as hypocenters, the areas close to the hypocenters may experience particularly strong ground motions in the direction perpendicular to the faults. Therefore, facilities that are installed with their face line directions arranged perpendicular to possible source faults are structurally advantageous for alleviating the actions of the earthquake ground motions caused by the source faults.

1.4 Selection of the Structural Types of Mooring Facilities

(1) The structural types of mooring facilities are preferably selected on the basis of the comparative examination of the following items and by taking into consideration the characteristics and economic efficiency of the respective structural types:

① **Natural conditions**

Natural conditions relevant to mooring facilities mainly include the mechanical property of soil, earthquakes, waves, tidal levels, and currents. In many cases, the mechanical property of soil becomes a decisive factor in selecting the structural types of mooring facilities because major Japanese ports are located within the vicinities of river mouths or inner bays with seafloors that are generally formed by the development of alluvium and mostly comprise soft soil. In cases of constructing mooring facilities on soft ground, structures that are capable of reducing burdens on the ground are selected, and ground improvement may be implemented if needed.

② **Use conditions**

Use conditions indicate the constraints in the use of mooring facilities after their construction imposed by the types of berthing ships, the types and quantities of cargoes to be handled, and the cargo-handling methods. Use conditions are the decisive factors of the fender reaction force, ship traction force, surcharges, and allowable deformation amounts to be set for the performance verification of mooring facilities.

③ **Construction conditions**

Considering that mooring facilities are often located offshore, they are subjected to various constraints in offshore work, i.e., the working hours involved in constructing mooring facilities offshore are largely affected not only by weather and temperature but also by waves, tides, and currents. Given that offshore work may cause serious problems with turbid seawater, it is necessary to take measures to prevent the surrounding environment from taking in turbid seawater. Furthermore, it is preferable to sufficiently examine the construction methods for mooring facilities by taking into consideration the difficulty in achieving and confirming the accuracy of undersea construction work. Onshore fabrication facilities such as caisson and block fabrication yards may become constraining factors in selecting the structural types of mooring facilities.

(2) The outlines and characteristics of the structural types of mooring facilities are as follows:

① **Gravity-type quaywalls**

(a) Outline:

Gravity-type quaywalls resist horizontal actions such as earth and water pressure by the weight of wall bodies. Fig. 1.4.1 shows an example of a cross section of a gravity-type quaywall.

(b) Characteristics:

1) Wall bodies are relatively firm and durable because they are made of concrete or similar materials.

2) The use of precast concrete members facilitates construction work and prevents reworking and accidents during construction.

3) The horizontal actions such as earth and water pressure acting on wall bodies increase when the water depths involved in mooring facility installations become deeper. This situation requires the weights of wall bodies to be drastically increased. Therefore, ground improvements may be required when constructing gravity-type quaywalls on soft ground where a large bearing capacity cannot be expected.

4) To prevent the ground behind quaywalls from being washed out, sand invasion prevention plates are installed between caissons and backfill stones, or sand invasion prevention sheets are laid on backfill stones. It is necessary to consider the damage that will be incurred by sand invasion prevention sheets because this type of damage may cause the sagging of aprons and reclaimed land behind quaywalls.

5) The actions on wall bodies due to earthquake ground motions are proportional to the weight of the wall bodies. Therefore, when wall bodies are designed to resist strong earthquakes by increasing quaywall widths, the wall bodies are also subjected to increased actions, i.e., it is difficult to prevent wall bodies from experiencing deformation during strong earthquakes. By contrast, gravity-type quaywalls, particularly caisson-type quaywalls, do not suddenly lose their stability in many cases even when they undergo deformation. Therefore, gravity-type quaywalls are advantageous in terms of serviceability after earthquakes. As described, the structures of gravity-type quaywalls have both advantages and disadvantages with respect to earthquakes.

6) Gravity-type quaywalls require large-scale onshore facilities such as caisson and block fabrication yards and special workboats such as crane barges and tug boats. Therefore, the construction of small-scale gravity-type quaywalls that do not require long construction periods is uneconomical if construction areas do not have onshore fabrication facilities and workboats.

7) In cases wherein existing seafloor surfaces are shallower than the design depths, gravity-type quaywalls are not advantageous because of the possible increase in dredging volume.

8) Gravity-type quaywalls should be carefully designed on soft cohesive ground because of the possible consolidation of cohesive layers, which cause gradual settlement over a long period of time.

9) Gravity-type quaywalls are classified into the following categories on the basis of the forms of wall bodies and construction methods:

i) Caisson-type quaywalls;

ii) L-shaped block quaywalls;

iii) Cellular-bulkhead-type quaywalls;

iv) Concrete block quaywalls;

v) Cast-in-place concrete quaywalls;

vi) Upright wave-absorbing-type quaywalls.

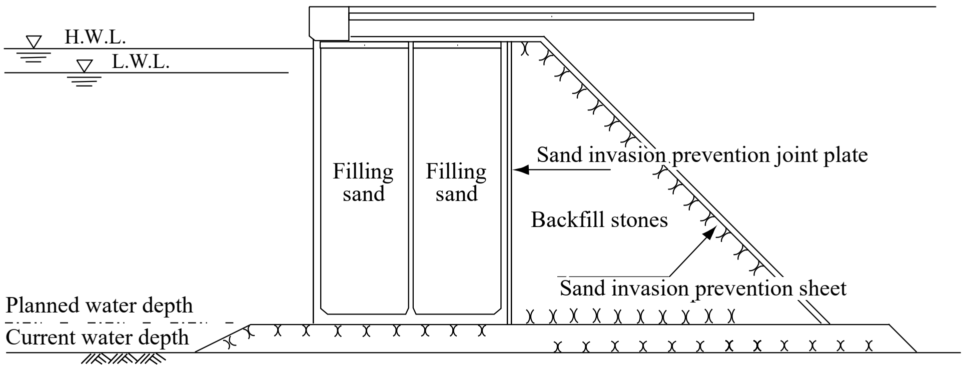


Fig. 1.4.1 Cross Section of a Gravity-Type Quaywall

② **Sheet pile quaywalls**

(a) Outline:

1) Sheet pile quaywalls are quaywalls with sheet piles driven as earth-retaining walls. Sheet piles can be made of steel, reinforced concrete, pre-stressed concrete, or wood. Among these materials, steel has been most frequently used for sheet piles. Given the large yield stress and availability of models with large section moduli, steel sheet piles can be used for quaywalls with large depths.

2) The cross-sectional shapes of normally used steel piles are classified into three types: hat-type, U-shaped, and steel pipes with joints. When using sheet piles with U-shaped cross sections, it is necessary to design their joints in a manner that prevents them from sliding because the configuration of the cross sections of U-shaped sheet piles with joints arranged along the neutral axes of sheet pile walls is subjected to reductions in section moduli obtained from sheet pile walls as integrated structures when sliding occurs between joints.

3) Steel pipe sheet piles fabricated by connecting large diameter steel pipes with joints can enlarge section moduli without large increments in the weight of steel per unit width, thereby enabling sheet piles with larger section moduli than steel sheet piles to be easily fabricated.

4) Reinforced concrete and pre-stressed concrete sheet piles are not often used for large-scale quaywalls because it is difficult to drive sheet piles with increased thicknesses owing to large section moduli. Even if they can be driven, there may be cases wherein sheet piles are damaged while being driven into stiff ground. Therefore, when driving reinforced concrete or pre-stressed concrete sheet piles into stiff ground, possible damage should be inspected by pullout tests or the sheet piles should be driven using water jetting to protect them from being damaged. Furthermore, sheet piles need to have joint plates to prevent the possible washing out of backfill soil through the joints.

(b) Characteristics:

1) Sheet piles quaywalls can be constructed using relatively simple machines at low costs.

2) In many cases, sheet pile quaywalls do not require undersea construction as foundation; therefore, such quaywalls can be rapidly constructed.

3) In cases wherein the existing seafloor is deep, sheet pile walls are placed into a state that is vulnerable to waves until backfill or anchorage work is constructed.

4) Sheet pile quaywalls are classified into the following categories on the basis of the method used to resist the earth and water pressure acting on the quaywalls:

i) Sheet pile quaywalls (in which sheet pile walls are connected to anchorage work via tie rods. Refer to **Fig. 1.4.2**);

ii) Cantilevered sheet pile quaywalls (refer to **Fig. 1.4.3**);

iii) Sheet pile quaywalls with raking pile anchorages (refer to **Fig. 1.4.4**);

iv) Sheet pile quaywalls anchored by forward batter piles (refer to **Fig. 1.4.5**);

v) Double sheet pile quaywalls (refer to **Fig. 1.4.6**).

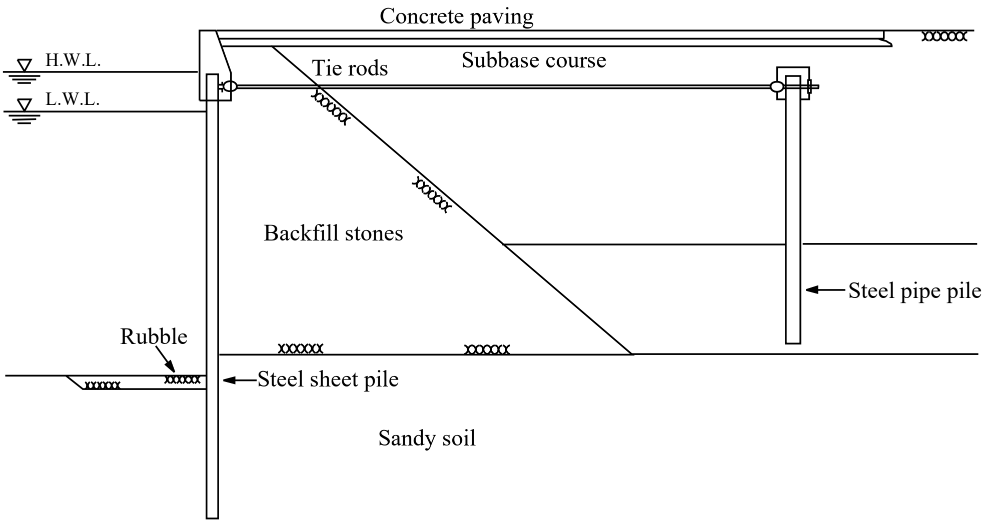


Fig. 1.4.2 Cross Section of a Sheet Pile Quaywall

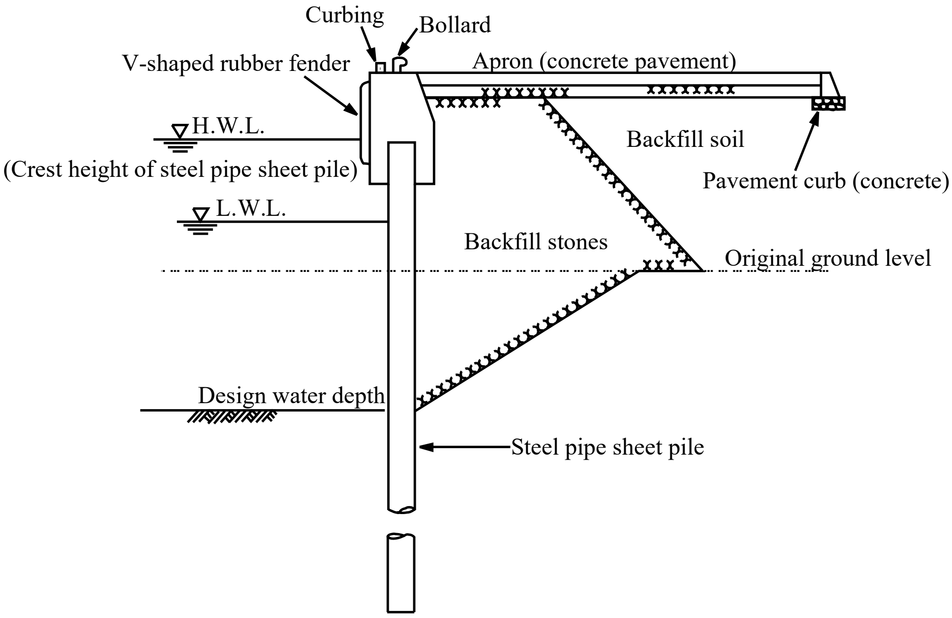


Fig. 1.4.3 Cross Section of a Cantilevered Sheet Pile Quaywall

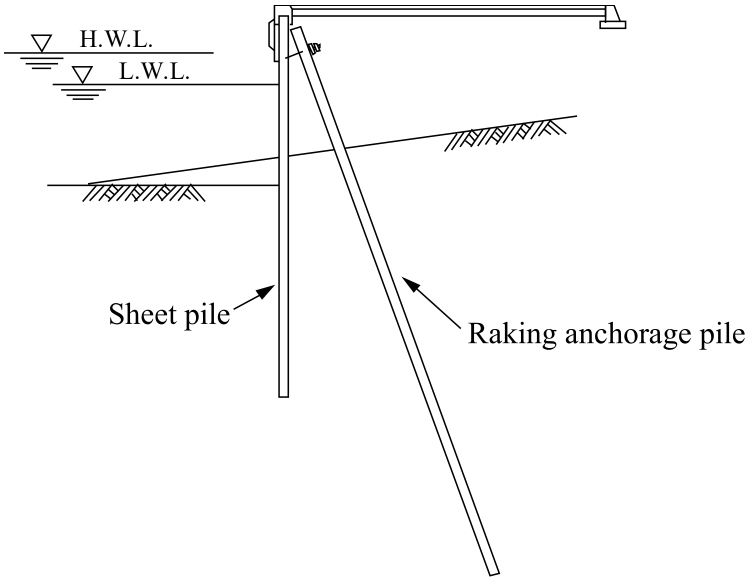


Fig. 1.4.4 Cross Section of a Sheet Pile Quaywall with Raking Pile Anchorage

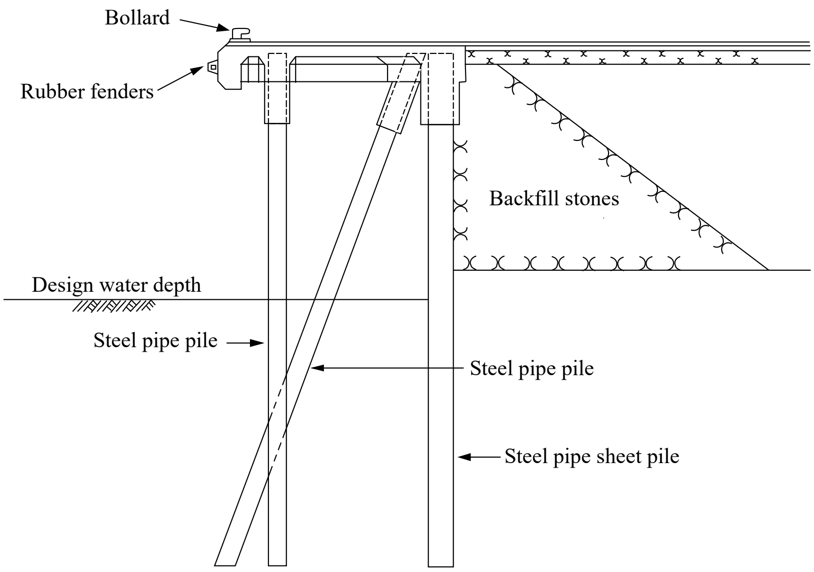


Fig. 1.4.5 Cross Section of a Sheet Pile Quaywall Anchored by Forward Batter Piles

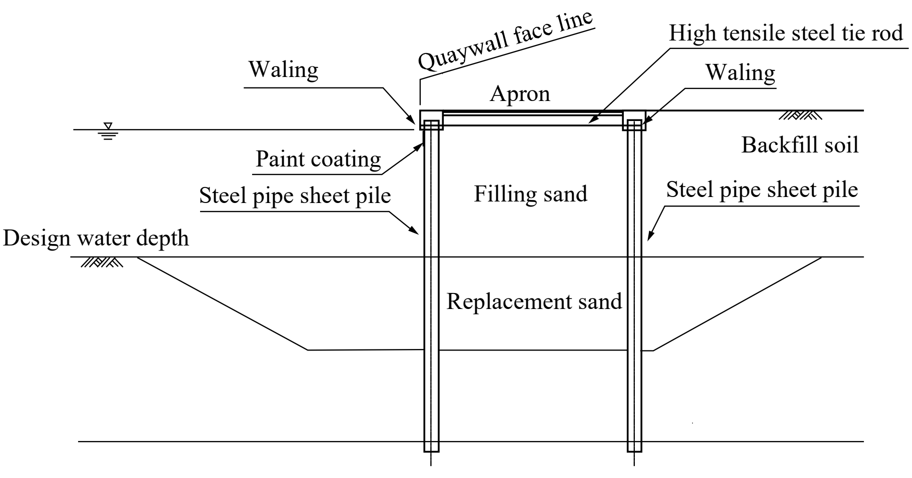


Fig. 1.4.6 Cross Section of a Double Sheet Pile Quaywall

③ **Quaywalls with relieving platforms**

(a) Outline:

Quaywalls with relieving platforms have structures that alleviate earth pressure acting on sheet piles with relieving platforms above sheet pile walls and resist horizontal loads with the active earth pressure at the embedded sections of sheet piles and with the horizontal resistance of relieving platform piles driven behind the sheet pile walls. Some types of quaywalls with relieving platforms have sheet pile walls at the rear face of relieving platforms. **Fig. 1.4.7** shows an example of a cross section of a quaywall with a relieving platform.

(b) Characteristics:

1) Quaywalls with relieving platforms enable piles that resist surcharges and can be constructed on soft ground, to which the structures of sheet pile quaywalls cannot be applied because sufficient passive earth pressure cannot be obtained as resistance at the embedded sections of sheet pile walls.

2) The sheet pile walls for quaywalls with relieving platforms can be smaller than those for sheet pile quaywalls.

3) The construction procedures of quaywalls with relieving platforms are complex compared with those of sheet pile quaywalls.

4) Quaywalls with relieving platforms require long construction periods.

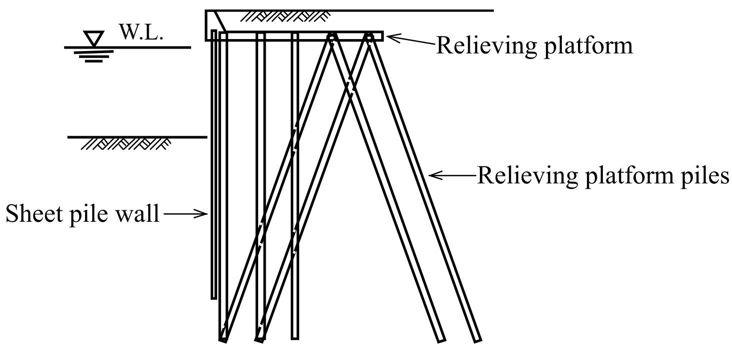


Fig. 1.4.7 Cross Section of a Quaywall with a Relieving Platform (L-Shaped Platform)

④ **Cellular-bulkhead type quaywalls**

(a) Outline:

Cellular-bulkhead type quaywalls have structures that comprise cylindrical cellular-bulkheads that are made of steel plates or steel sheet piles and are installed on the seafloor and infill materials placed inside the cellular bulkhead to ensure quaywall stability. The structures of cellular-bulkhead type quaywalls are largely classified into placement-type cellular bulkhead without embedment and embedded-type cellular bulkhead with embedment. Placement-type cellular-bulkhead quaywalls have structures that resist an external force with the weight and shear resistance of infill. Embedded-type cellular-bulkhead quaywalls can utilize the resistance at the embedded sections to resist external force, in addition to the weight and shear resistance of infill. Cellular bulkheads are normally made of steel, particularly steel plates for placement-type quaywalls and steel plates or linear sheet piles for embedded-type quaywalls. **Fig. 1.4.8** shows an example of a cross section of a cellular-bulkhead quaywall (embedded type).

(b) Characteristics:

1) The structures of cellular-bulkhead quaywalls are relatively simple; therefore, these quaywalls are suitable for rapid construction. The structures are also economical when ground conditions are suitable.

2) Embedded-type cellular-bulkhead quaywalls can eliminate the necessity of constructing foundation mounds; therefore, these quaywalls can reduce the workload associated with undersea construction.

3) Sufficient embedded lengths are required when constructing cellular-bulkhead quaywalls on the ground with small bearing capacity.

4) Cellular-bulkhead quaywalls are classified into the following categories:

   i) Placement-type cellular-bulkhead quaywalls;

   ii) Embedded-type cellular-bulkhead quaywalls.

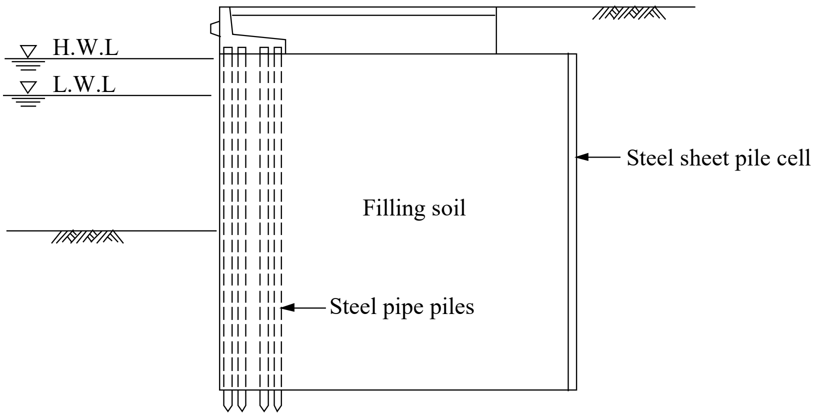


Fig. 1.4.8 Cross Section of a Steel Sheet Pile Cellular-Bulkhead Quaywall

⑤ **Open-type piled wharves and piled wharves**

(a) Outline:

Open-type piled wharves generally comprise earth-retaining revetments and open-type wharves constructed in front of revetments. Earth-retaining revetments have earth-retaining walls with earth slopes behind them. Piled wharves comprise columns such as piles and slabs installed on the columns. **Fig. 1.4.9** shows an example of an open-type wharf on vertical piles.

(b) Characteristics (open-type wharves):

1) Open-type wharves are suitable for soft ground where quaywalls with vertical front faces may cause ground failures.

2) Plural piles are arranged in the direction perpendicular to face lines (perpendicular to berths); therefore, the yield of one pile does not immediately lead to the failures of entire structures.

3) Open-type wharves can utilize existing facilities when constructing new wharves in front of existing revetments or extending the depths of existing shallow berths.

4) In cases of open-type wharves constructed in the areas with high waves, they may be at risk of failures with slabs and access bridges subjected to upward wave force.

5) The structures of open-type wharves comprising the combination of earth-retaining and wharf sections require complex construction procedures.

(c) Characteristics (piled wharves):

1) The structures of piled wharves where superstructures are supported by piles are light in weight compared with other structural types; therefore, these structures are suitable for soft ground to which gravity-type or sheet pile quaywalls cannot be applied.

2) Considering that piled wharves barely disturb the flow of seawater, they can be constructed even in areas with large influences of littoral drift and currents without disturbing the balance of natural conditions.

3) Piled wharves do not require soil for reclamation.

4) Piled wharves may interfere with ship berthing because they do not disturb the flow of seawater.

(d) Characteristics (common to both open-type piled wharves and piled wharves):

1) Both types have a disadvantage for large concentrated loads.

2) Both types are relatively vulnerable to horizontal force.

3) Open-type piled wharves and piled wharves are classified into the following categories depending on the structures of studs supporting slabs:

   i) Piled wharf;

   ii) Cylindrical or square-tube-type wharf;

   iii) Bridge-pier-type wharf

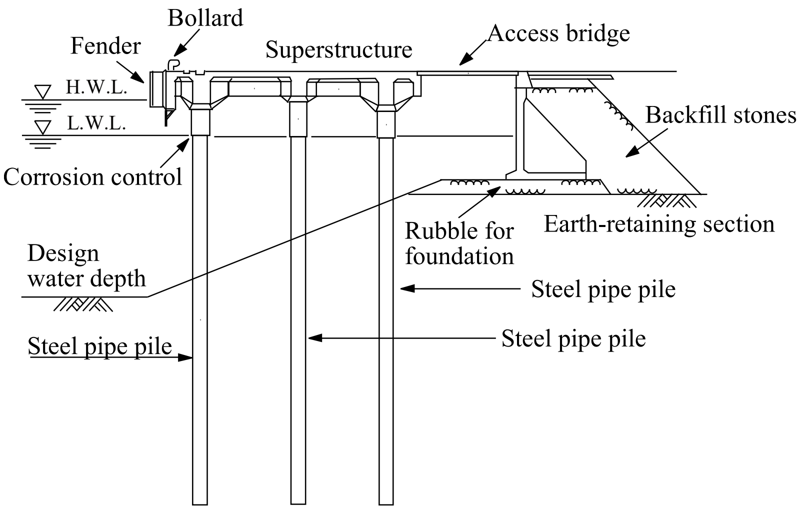


Fig. 1.4.9 Cross-section of an Open Type Wharf on Vertical Piles

⑥ **Detached piers**

(a) Outline:

Detached piers are mooring facilities that are used for handling bulk cargoes, such as coal and iron ore, in large quantities and have rail-mounted portal bridge cranes or loaders with their foundations installed at appropriate depths. Generally, detached piers do not require floor structures and comprise column sections and beam sections installed between column sections. Detached piers are categorized as special open-type piled wharves with no slabs and access bridges; therefore, these piers have characteristics that are similar to open-type piled wharves. **Fig. 1.4.10** shows an example of a cross section of a detached pier.

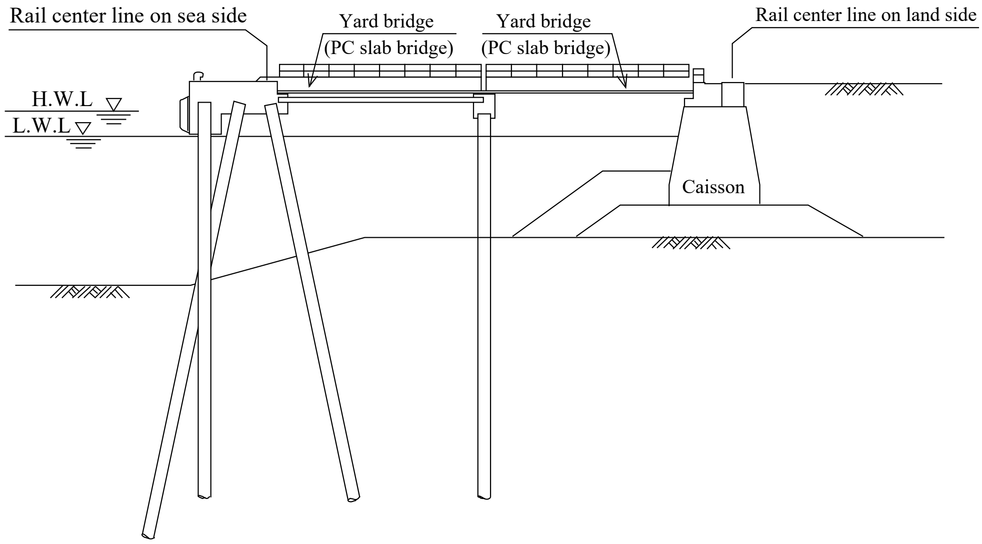


Fig. 1.4.10 Cross Section of Detached Pier

⑦ **Floating piers**

(a) Outline:

Floating piers are mooring facilities that comprise pontoons connected to land and to other pontoons by access bridges. **Fig. 1.4.11** shows an example of a cross sections and plan of a floating pier.

(b) Characteristics:

1) Pontoons maintain fixed distances between the top surfaces of floating piers and water surfaces in a manner that moves up and down in accordance with water levels. Therefore, they are suitable for mooring small vessels and ferry boats mainly for passenger use.

2) Floating piers allow seawater to flow more freely than piled wharves; therefore, these piers have little influence on littoral drift.

3) Floating piers can be easily constructed and relocated.

4) Floating piers are suitable for relatively soft ground.

5) Floating piers have small cargo-handling capacities because of difficulties in mounting cargo-handling equipment on them.

6) Floating piers are unsuitable for areas subjected to large influences of waves and currents.

7) Considering that steel materials are generally used for mooring wires and anchors, it is necessary to maintain respective sections that are made of steel materials with particular focus not only on corrosion but also mechanical wear.

8) Depending on the materials used for fabrication, pontoons are classified into reinforced concrete, steel, prestressed concrete, wood, and FRP types.

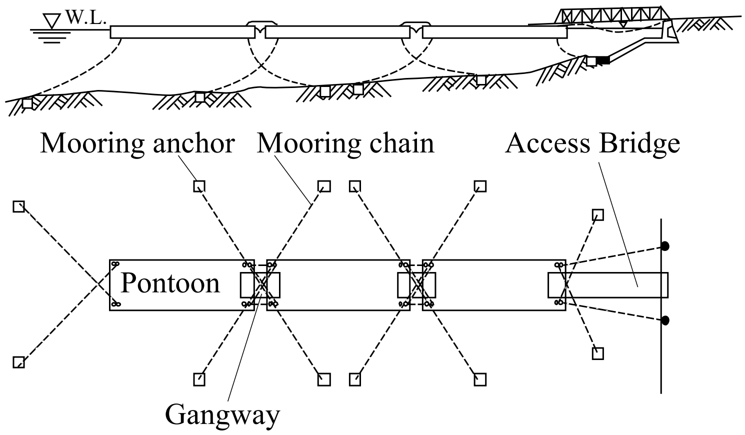


Fig. 1.4.11 Cross Section and Plan of a Floating Pier

⑧ **Dolphins**

(a) Outline:

Dolphins are mooring facilities that comprise a group of columnar structures installed offshore. **Fig. 1.4.12** shows an example of a cross section of a dolphin.

(b) Characteristics:

1) In coastal areas with predetermined depths, dolphins can be easily constructed at low cost and in a short period of time without involving dredging and reclamation work.
2) Dolphins have not been used for handling general cargoes but for handling large quantities of oil, cement, grains, and powdery bulk cargoes with dedicated cargo-handling machines installed on them.
3) Dolphins can be constructed as parts of other main mooring facilities at their bow and stern ends to reduce the lengths of main mooring facilities. When attached to existing mooring facilities, dolphins can extend the effective lengths of existing facilities to be used for mooring.
4) Dolphins are classified into the following categories depending on their structures:
   i) Pile-type dolphin;
   ii) Steel cellular-bulkhead-type dolphin;
   iii) Caisson-type dolphin.

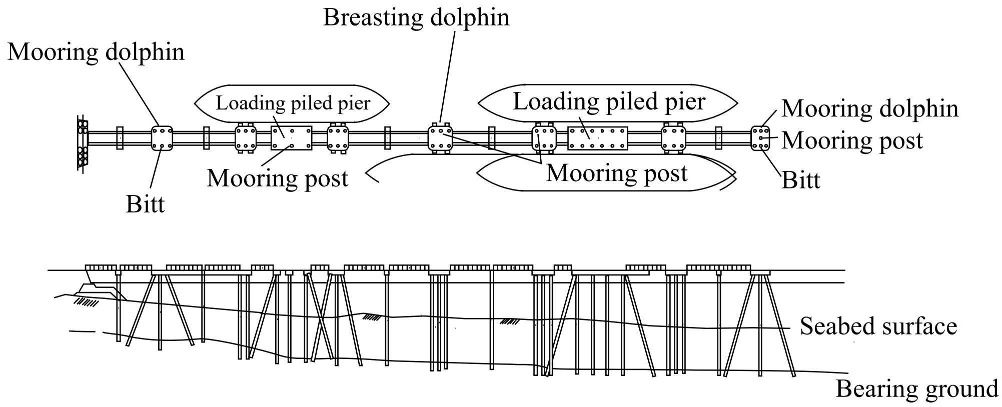


Fig. 1.4.12 Cross Section of a Dolphin

⑨ **Docks**

(a) Outline:

Docks are facilities provided with slip ways to bring ships onshore. **Fig. 1.4.13** shows an example of a cross section of a dock.

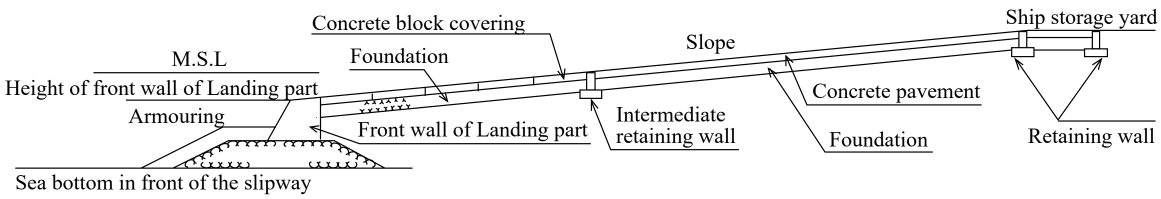


Fig. 1.4.13 Cross Section of a Dock

⑩ **Air-cushion-craft landing facilities**

(a) Outline:

Air cushion crafts are high-speed crafts that navigate above sea surfaces with craft bodies levitated by strong downward airflow. The mooring facilities for air cushion crafts include slip ways, aprons, pontoons, and open-type wharves. Considering that craft bodies have special structures, air cushion crafts require ancillary facilities that are different from normal mooring facilities. Furthermore, the locations of facilities for air cushion crafts should be carefully selected with due consideration to the fact that the navigation of air cushion crafts is largely affected by meteorological and oceanographic conditions and that air cushion crafts generate noise and ship wake waves.

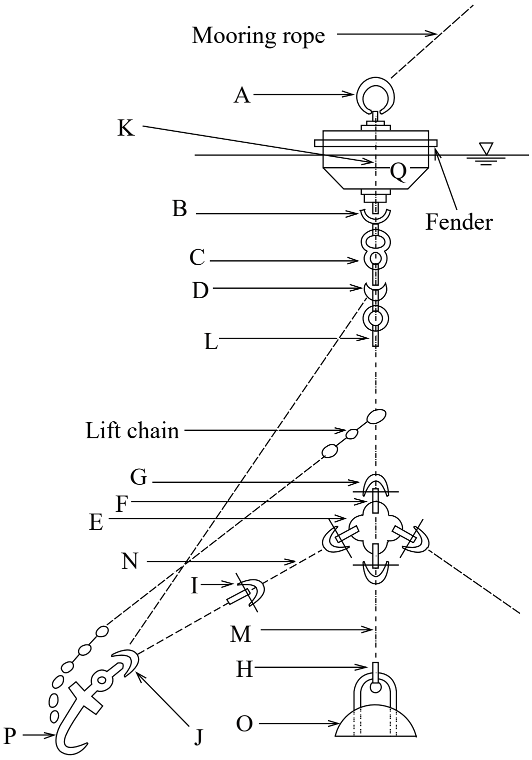
⑪ **Mooring buoys**

(a) Outline:

Mooring buoys are used mainly for mooring ships in basins. Mooring buoys generally comprise floating bodies, mooring rings, anchoring chains, sinkers, and mooring anchors. **Fig. 1.4.14** shows an example of a cross section of a mooring buoy. Mooring buoys have been used for handling petroleum products and timber, cargo handling with barges, and mooring ships without cargo handling operations.

(b) Characteristics:

1) Mooring buoys enable ships to be moored in narrower basin areas than anchoring.
2) Mooring buoys enable ports to accommodate ships that cannot anchor because of exposed bedrock on seafloor surfaces.
3) In coastal areas with predetermined depths, mooring buoys can be easily constructed at low costs and in a short period of time without involving dredging and reclamation work.
4) Mooring buoys can be easily relocated.
5) The operational wave heights of mooring buoys are higher than other types of mooring facilities.
6) It is generally difficult to mechanize cargo-handling operations with mooring buoys. Therefore, in many cases, mooring buoys have lower cargo-handling efficiency than other mooring facilities.
7) Generally, mooring buoys require large basin areas than other mooring facilities.



A; Harp shackle (mooring ring) or quick release hook	1
B; Anchor shackle	1
C; Swivel piece	1
D; Joining shackle	1
E; Mooring piece	1
F; Long ring	2
G; Joining shackle	2
H; Anchor shackle	1
I; Joining shackle	2
J; Anchor shackle	2
K; Chain	1
L; Main chain (Anchoring chain of floating body)	1
M; Sinker chain	1
N; Chain or ground chain	4
O; Sinker	1
P; Anchor or screw	1
Q; Buoy	1

Fig. 1.4.14 Example Cross Section of a Mooring Buoy

1.5 Points of Caution Regarding High-Earthquake-Resistance Facilities

(1) Mooring facilities that are categorized as high-earthquake-resistance facilities shall ensure restorability under the accidental situation with respect to Level 2 earthquake ground motions. Facilities that require additional enhancement in earthquake resistance in accordance with their natural and social conditions shall also ensure usability under this accidental situation. Restorability and usability are defined as the required performances for the functions that need to be operative after the actions of Level 2 earthquake ground motions and not for the functions that need to be fulfilled in normal time.

(2) Classifications of high-earthquake-resistance facilities

Depending on the functions that need to be maintained after the actions of Level 2 earthquake ground motions, high-earthquake-resistance facilities are classified into the following: high-earthquake-resistance facilities (specially designated emergency supply transport), high-earthquake-resistance facilities (specially designated trunk line cargo transport), and high-earthquake-resistance facilities (standard emergency supply transport). The performance requirements and the contents of the design situations are set for the respective facilities according to this classification. Refer to **Table 1.5.1** for the details of the classification of high-earthquake-resistance facilities.

Table 1.5.1 Classification of High-Earthquake-Resistance Facilities

	High-earthquake-resistance facility		
	Specially designated		Standard
	Emergency supply transport	Trunk line cargo transport	Emergency supply transport
Functions required after the actions of Level 2 earthquake ground motions	Facilities need to maintain structural stability after earthquakes so that they can promptly be used for the mooring and landing operation of ships, the embarkation and disembarkation of passengers, and the loading and unloading	Facilities need to maintain structural stability after earthquakes so that they can promptly (in a short period of time) be used for the mooring and landing of ships and the loading and unloading of trunk line cargoes.	Facilities need to maintain structural stability after earthquakes so that they can be used for the loading and unloading of emergency relief supplies after a lapse of a certain period.

	High-earthquake-resistance facility		
	Specially designated		Standard
	Emergency supply transport	Trunk line cargo transport	Emergency supply transport
	of cargoes, including emergency relief supplies.		
	Functions required after earthquakes (Primary functions are not required)	Primary functions	Functions required after earthquakes (Primary functions are not required)
Required performance	Usability*)	Restorability	Restorability*)
Allowable degree of restoration	Minor repairs	Minor repairs	A certain level of repairs

*): The required performance is for the functions to be fulfilled after earthquakes (to transport emergency relief supplies) and not for the primary functions of respective facilities.

① High-earthquake-resistance facilities for emergency supply transport

(a) Specially designated (emergency supply transport)

High-earthquake-resistance facilities (specially designated emergency supply transport) shall ensure usability under the accidental situation with respect to Level 2 earthquake ground motions. Here, usability does not always mean that facilities need to resist the actions of Level 2 earthquake ground motions without being damaged but indicates that facilities need to protect against damage up to a level that retains their function for the loading and unloading of emergency relief supplies.

(b) Standard (emergency supply transport)

High-earthquake-resistance facilities (standard emergency supply transport) shall ensure restorability under the accidental situation with respect to Level 2 earthquake ground motions. Here, restorability means that the facilities need to protect against damage due to the actions of Level 2 earthquake ground motions up to a level wherein emergency restoration can allow the loading and unloading of emergency relief supplies after a lapse of a certain period, i.e., approximately one week after the facilities were subjected to the actions of Level 2 earthquake ground motions.

(c) Difference between specially designated and standard emergency supply transports

The major difference between high-earthquake-resistance facilities (specially designated emergency supply transport) and high-earthquake-resistance facilities (standard emergency supply transport) is the time allowed for these facilities to become available for the loading and unloading of emergency relief supplies after they were subjected to the actions of Level 2 earthquake ground motions. In this regard, it is necessary to plan high-earthquake-resistance facilities for emergency supply transport by taking into consideration this difference.

Specifically, the Basic Disaster Prevention Plan (Article 34 of the Disaster Countermeasure Basic Act) can consider a phased emergency supply transportation operation according to the degrees of emergency of relief supplies (**Table 1.5.2**). According to the table, the planning of high-earthquake-resistance facilities can incorporate such phased operations in a manner that enables high-earthquake-resistance facilities (specially designated emergency supply transport) to serve emergency supply transports from the early phase in region I and high-earthquake-resistance facilities (standard emergency supply transport) to additionally serve emergency supply transports from the intermediary phase in region II.

Table 1.5.2 Phased Emergency Supply Transportation Operation in the Basic Disaster Prevention Plan

Region	Phase	Subject of emergency transportation	
I	Phase 1	(1)	Manpower and supplies for lifesaving operations such as rescue, first aid, and medical activities
		(2)	Manpower and supplies for operations to prevent disasters from expanding, such as firefighting and flood control
		(3)	Manpower and supplies for governmental and municipal disaster countermeasures, as well as emergency measures, to restore and protect communication, power, gas and water
		(4)	Injured persons to be transferred to backup medical institutions
		(5)	Manpower and supplies for the urgent restoration of transportation facilities, bases, and traffic control
	Phase 2	(1)	Continuation of phase 1 activities
		(2)	Supplies such as food and water necessary for the maintenance of life
		(3)	Patients, injured persons, and disaster victims to be transferred out of the disaster areas
		(4)	Manpower and supplies necessary for the urgent restoration of transportation facilities
II	Phase 3	(1)	Continuation of phase 2 activities
		(2)	Manpower and supplies necessary for disaster restoration
		(3)	Daily essentials

② **High-earthquake-resistance facilities for trunk line cargo transport**

High-earthquake-resistance facilities (specially designated trunk line cargo transport) shall ensure restorability under the accidental situation with respect to Level 2 earthquake ground motions. Here, restorability means that these facilities need to protect against damage due to the actions of Level 2 earthquake ground motions within predetermined levels (e.g., displacement of the facilities in the ranges allowed for the operation of respective cargo handling machines) to enable these facilities to remain functional for trunk line cargo transport again after minor repairs following the lapse of short periods. The short periods need to be appropriately defined in accordance with the required functions, which differ for each facility.

(3) Standard concept of the limit values for the deformation of high-earthquake-resistance facilities with respect to Level 2 earthquake ground motions

The standard limit values for the deformation of high-earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions can be set in accordance with the required performance of respective facilities, as shown below. However, this provision shall not be applied to cases wherein the limit values for deformation are set on the basis of comprehensive determination by taking into consideration the situations of areas where facilities are constructed, the required performance, and the structural types of the facilities.

① **High-earthquake-resistance facilities for emergency supply transport**

(a) Specially designated (emergency supply transport)

The standard limited values for the residual horizontal deformation and residual inclination angles of high-earthquake-resistance facilities (specially designated emergency supply transport) can be set from a functional viewpoint at approximately 30 to 100 cm and 3°, respectively. For example, the limit value for a residual horizontal deformation of 100 cm can be set for facilities that can ensure the required usability even if the deformation may be large because the materials for emergency repairs are always ready and because emergency response systems are established. When setting the limit values, refer to the performance records of the emergency supply transport that was established immediately after the 1995 South Hyogo Prefecture Earthquake.[1] It has been indicated that the meandering (uneven displacement) of face lines is a more important factor than residual horizontal deformation when evaluating the usability of mooring facilities in terms of ship berthing during emergencies.[2] Accordingly, there is an idea of deriving the limit values for residual deformation by first setting the limit values for uneven displacement and then using the correlation between the uneven displacement and residual deformation. It is also important to curb the level differences between mooring facilities and pavements at their back in terms of the facilitation of cargo handling during emergencies. Curbing horizontal deformation generally curbs the level differences between mooring facilities and pavements at their back.

(b) Standard (emergency supply transport)

The limit values for the residual horizontal deformation of high-earthquake-resistance facilities (standard emergency supply transport) shall be appropriately set at approximately 100 cm or more by taking into consideration the availability of cargo handling operation after a lapse of a certain period following the actions of Level 2 earthquake ground motions.

② **High-earthquake-resistance facilities for trunk line cargo transport**

The limit values for the residual deformation of high-earthquake-resistance facilities (specially designated trunk line cargo transport) shall be set on the basis of the periods necessary for restoring the required functions. Regarding the periods, there are cases wherein it is rational to set shorter periods for earthquakes such as subduction-zone earthquakes, which cause more extensive damage than inland active fault earthquakes, which cause concentrated damage on relatively narrow areas, from the viewpoint of maintaining functions for trunk line cargo transport. In such cases, smaller limit values can be set for subduction-zone earthquakes than for inland active fault earthquakes.

There are many cases of high-earthquake-resistance facilities (specially designated trunk line cargo transport) that are provided with cranes with a seismic isolation or vibration control function to equalize the earthquake resistance of mooring facilities and cranes. In such cases, earthquake response analyses that consider the dynamic interaction between mooring facilities and cranes shall be performed to keep the responses of the structural members of cranes within the elastic limits. For the details of seismic isolation or vibration control cranes, refer to **Part III**, **Chapter 7**, **2.2 Container Cranes**.

[References]

1) Takahashi, H., T. Nakamoto and F. Yoshimura: Analysis of maritime transportation in Kobe Port after the 1995 Hyogoken-Nanbu Earthquake, Technical Note of PHRI No. 861, 1997

2) Kazui, K., H. Takahashi, T. Nakamoto and Y. Akakura: Evaluation of allowable damage deformation of gravity type quaywall during earthquake, Proceedings of 10th Symposium on Earthquake Engineering, K-4, 1998

2 Wharves

2.1 Common Items for Wharves

[Ministerial Ordinance] (Performance Requirements for Quaywalls)

Article 26

1 The performance requirements for quaywalls shall be as prescribed respectively in the following items in consideration of the structural type:

(1) The performance requirements shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied so as to enable the safe and smooth mooring of ships, embarkation and disembarkation of people, and handling of cargoes.

(2) Damage to the quaywall, etc. due to the action of self-weight, earth pressure, Level 1 earthquake ground motion, berthing and traction by ships, surcharge loads, etc. shall not impair the functions of the quaywalls and shall not adversely affect the continuous use of the quaywall.

2 In addition to the provisions of the previous paragraph, the performance requirements for quaywalls provided in the following items shall be as prescribed respectively in those items:

(1) “Performance requirements for quaywalls to protect environment” shall be such that quaywalls shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to contribute to conservation of the environment of ports and harbors without impairing the original functions of the quaywalls.

(2) “Performance requirements for quaywalls classified as high earthquake-resistance facilities” shall be such that damage due to the action of Level 2 earthquake ground motions, etc. shall not affect the restoration through minor repair works of functions required for the quatwalls in the aftermath of the occurrence of Level 2 earthquake ground motion. Provided, however, that for the performance requirements for the quaywall which requires further improvements in earthquake-resistant performance due to environmental conditions, social conditions, etc. to which the quaywalls are subjected, damage due to Level 2 earthquake ground motions, etc. shall not impair the functions required for the quatwalls in the aftermath of the occurrence of Level 2 earthquake ground motion, and shall not adversely affect the continuous uses of the quaywalls.

[Public Notice] (Performance Criteria of Quaywalls)

Article 48

1 The performance criteria common to quaywalls shall be as prescribed respectively in the following items:

(1) Quaywalls shall have the water depth and length necessary for accommodating the design ships in consideration of their dimensions.

(2) Quaywalls shall have a crown height that considers the range of tidal levels, the dimensions of the design ship, and the usage conditions of the facilities.

(3) Quaywalls shall have ancillary equipment as necessary in consideration of the usage conditions.

2 In addition to the provisions of the preceding paragraph, the performance criteria for quaywalls specified in the following items shall be as prescribed respectively in those items:

(1) “Performance criteria for quaywalls for the purpose of environmental conservation” shall be such that quaywalls have the dimensions necessary to contribute to conservation of environments of ports and harbors in consideration of the environmental conditions, etc. to which the quaywalls are subjected, without impairing the original functions of the quaywalls.

(2) “Performance criteria for quaywalls classified as high earthquake-resistance facilities” shall be such that the degree of damage under the accidental situation, in which dominating action is Level 2 earthquake ground motion, shall be equal to or less than the threshold level in consideration of the performance requirements.

[Interpretation]

11. Mooring Facilities

(1) Green Quaywalls (Article 26, Paragraph 2, Item 1 of the Ministerial Ordinance on Criteria and Interpretation related toArticle 48, Paragraph 2, Item 1of the Public Notice)

① Quaywalls for protecting the environment are classified as green quaywalls to which the subsequent items shall be applied, in addition to the provisions for quaywalls.

② The performance requirement for green quaywalls shall focus on serviceability. The term "protective capability" refers to the performance of quaywalls in protecting port environments for organisms, ecosystems, and others without impairing their essential functions.

③ The dimensions of quaywalls for protecting environments shall indicate the structure, cross-sectional dimensions, and ancillary facilities. When setting the structure and cross-sectional dimensions in the performance verifications of quaywalls to protect environments and installing ancillary facilities, appropriate consideration shall be given to factors that affect the objective to protect port environments for organisms and ecosystems without impairing the essential functions of the quaywalls.

(2) Quaywalls That Are Classified as High Earthquake-resistance Facilities (Article 26, Paragraph 2, Item 2 of the Ministerial Ordinance on Criteria and Interpretation related to Article 48, Paragraph 2, Item 2 of the Public Notice)

① The following classifications are used as standards in provisions stipulating the appropriate performance of high earthquake-resistance facilities corresponding to the functions necessary after the action of Level 2 earthquake ground motions and the allowable period for restoration to demonstrate those functions.

a) Specially designated (emergency supply transport): facilities that can be used by ships and perform embarkation/disembarkation of persons, cargo handling of emergency supplies, etc., immediately after the action of Level 2 earthquake ground motions.

b) Specially designated (trunk line cargo transport): facilities that can be used by ships and perform cargo handling of trunk line cargoes within a short period after the action of Level 2 earthquake ground motions.

c) Standard (emergency supply transport): facilities that can be used by ships and perform the embarkation/disembarkation of persons, cargo handling of emergency supplies, etc., within a certain period after the action of Level 2 earthquake ground motions.

② The performance requirements for high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action are stipulated in the subsequent items corresponding to the classifications of high earthquake-resistance facilities:

(a) The performance requirement for high earthquake-resistance facilities (specially designated (emergency supply transport)) shall focus on serviceability. Serviceability refers to the limited performance requirements for the functions of facilities deemed necessary for transporting emergency supplies after earthquakes and is independent of the serviceability required for normal cargo handling work in facilities.

(b) The performance requirement for high earthquake-resistance facilities (specially designated (trunk line cargo transport)) shall focus on restorability.

(c) The performance requirement for high earthquake-resistance facilities (standard (emergency supply transport)) shall focus on restorability.

③ **Attached Table 11-1** shows the verification items and standard indexes to determine the limit values that are common to quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action. "Damage" has been adopted as the verification item in **Attached Table 11-1** from the viewpoint of comprehensiveness and by considering the fact that verification items will differ depending on the structural type. Furthermore, the indexes for determining limit values shall be appropriately set for performance verification. It may also be noted that settings in connection with the Public Notice Article 22 (Common Performance Criteria of Component Members of Target Facilities Subject to the Technical Standard) may also be applied when necessary, in addition to this code.

Attached Table 11-1 Verification Items and Standard Indexes for Determining the Limit Values that Are Common to Quaywalls Classified as High Earthquake-resistance Facilities

Ministerial Ordinance			Public Notice			Performance requirements*	Design state			Verification item	Standard indexes to determine the limit values
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	2	2	48	2	2	Restorability, serviceability	Accidental	L2 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Damage	–

*) In this table, "serviceability" refers to the "necessary function after earthquake (emergency supply transport)."
*) In this table, "restorability" refers to the "essential function" or "necessary function after earthquake (emergency supply transport)."

④ **Attached Table 11-2** shows the verification items and standard indexes for determining the limit values for gravity-type quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action. The standard indexes for determining the limit values for the face line deformation quantity of a quaywall in the table can be set with reference to the descriptions in subsequent items corresponding to the classification of high earthquake-resistance facilities.

Attached Table 11-2 Verification Items and Standard Indexes for Determining the Limit Values for Gravity-type Quaywalls Classified as High Earthquake-resistance Facilities

Ministerial Ordinance			Public Notice			Performance requirements*	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	2	2	48	2	2	Restorability, serviceability	Accidental	L2 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Deformation of the face line of the quaywall	Limit of residual deformation

*) In this table, "serviceability" refers to the "necessary function after earthquake (emergency supply transport)."
*) In this table, "restorability" refers to the "essential function" or "necessary function after earthquake (emergency supply transport)."

(a) High earthquake-resistance facilities (specially designated (emergency supply transport))

The limit of deformation of high earthquake-resistance facilities (specially designated (emergency supply transport)) shall be the deformation of a degree such that the berthing of ships for the marine transport of emergency supplies, evacuees, construction machinery for removing obstructions, etc., is possible and shall be set appropriately. In general, the residual horizontal displacement of the quaywall can be used as the index of deformation.

(b) High earthquake-resistance facilities (specially designated trunk line supply transport)

The limit of deformation of high earthquake-resistance facilities (specially designated (trunk line cargo transport)) shall be the deformation of a degree such that trunk line cargo transport can be performed after slight restoration, within the permissible displacement set in line with the characteristics of the cargo handling equipment, or similar and shall be set appropriately. In general, the residual horizontal displacement of the quaywall, residual inclination angle of the wall, and relative displacement of the rail span can be used as indexes of deformation. In the case of quaywalls using cargo handling equipment for trunk line cargo transport, appropriate consideration shall be given to the form, type, and characteristics of the cargo handling equipment when setting limit values.

(c) High earthquake-resistance facilities (standard (emergency supply transport))

The limit of deformation of high earthquake-resistance facilities (standard (emergency supply transport)) shall be the deformation of a degree such that cargo handling of emergency supplies can be performed after emergency restoration within a given period of time and shall be set appropriately. In general, the residual horizontal displacement of the quaywall can be used as the index of deformation.

⑤ **Attached Table 11-3** shows the verification items and standard indexes to determine the limit values for sheet pile quaywalls classified as high earthquake-resistance facilities (specially designated (emergency supply transport) and specially designated (trunk line supply transport)) under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action. The structural types of anchorages are broadly classified as vertical pile anchorage, coupled-pile anchorage, sheet pile anchorage, and concrete wall anchorage. In the performance verification of anchorages, appropriate verification items shall be set corresponding to the structural type. The standard indexes for determining the limit values for the face line deformation quantity in the table shall be equivalent to the performance criteria of gravity-type quaywalls classified as high earthquake-resistance facilities (specially designated (emergency supply transport) and specially designated (trunk line supply transport)).

Attached Table 11-3 Verification Items and Standard Indexes to Determine the Limit Values for Sheet Pile Quaywalls Classified as High Earthquake-resistance Facilities (Specially Designated (Emergency Supply Transport) and Specially Designated (Trunk Line Supply Transport)) with respect to the Accidental Situation

Ministerial Ordinance			Public Notice			Performance requirements*	Design state			Verification item	Standard index to determine limit values
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	2	2	48	2	2	Serviceability, restorability	Accidental	L2 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Deformation of the face line of the quaywall	Limit of the residual deformation
										Yielding of sheet piles	Design yield stress
										Rupture of the tie member	Design rupture strength
										Damage to anchorage*1)	Limit curvature
										Axial force acting on anchorage*2)	Action-resistance ratio with respect to the bearing force of anchorage (pushing and pulling)
										Stability of the anchorage*3)	Design ultimate capacity of the section
										Cross-sectional failure of the superstructure	Design ultimate capacity of the section

*1): The structural types of anchorages are limited to cases of vertical pile anchorage, coupled-pile anchorage, and sheet pile anchorage.
*2): The structural types of anchorages are limited to the case of coupled-pile anchorage.
*3): The structural types of anchorages are limited to the case of concrete wall anchorage.
*) In this table, "serviceability" refers to the "necessary function after earthquake (emergency supply transport)" and indicates the required capacity for specially designated (emergency supply transport).
*) In this table, "restorability" refers to the "essential function" and indicates the required capacity for specially designated (trunk line cargo transport).

⑥ **Attached Table 11-4** shows the verification items and standard indexes for determining the limit values for sheet pile quaywalls classified as high earthquake-resistance facilities (standard (emergency supply transport)) under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action. The standard indexes for determining the limit values for the face line deformation quantity in the table shall be equivalent to the performance criteria of gravity-type quaywalls classified as high earthquake-resistance facilities (standard (emergency supply transport)).

Attached Table 11-4 Verification Items and Standard Indexes for Determining the Limit Values for Sheet Pile Quaywalls Classified as High Earthquake-resistance Facilities (Standard (Emergency Supply Transport)) with respect to the Accidental Situation

Ministerial Ordinance			Public Notice			Performance requirements*	Design state			Verification item	Standard index for determining the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	2	2	48	2	2	Restorability	Accidental	L2 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Deformation of the face line of the quaywall	Limit of the residual deformation
										Damage to sheet pile	Limit curvature
										Rupture of the tie member	Design rupture strength
										Damage to anchorage*1)	Limit curvature
										Axial force acting on anchorage*2)	Action-resistance ratio with respect to the bearing force of anchorage (pushing and pulling)
										Stability of the anchorage*3)	Design ultimate capacity of the section
										Cross-sectional failure of the superstructure	Design ultimate capacity of the section

*1): The structural types of anchorages are limited to the cases of vertical pile anchorage, coupled-pile anchorage, and sheet pile anchorage.
*2): The structural types of anchorages are limited to the case of coupled-pile anchorage.
*3): The structural types of anchorages are limited to the case of concrete wall anchorage.
*) In this table, “restorability” refers to the “necessary function after earthquake (emergency supply transport),”

⑦ The verification items and standard indexes for determining the limit values for cantilevered sheet pile quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action shall be equivalent to the provisions for sheet pile quaywalls classified as high earthquake-resistance facilities with the exception of the verification items for tie rods and anchorages.

⑧ The verification items and standard indexes for determining the limit values for double sheet pile quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action shall be equivalent to the provisions for sheet pile quaywalls classified as high earthquake-resistance facilities.

⑨ The verification items and standard indexes for determining the limit values for quaywalls with relieving platforms classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action shall be equivalent to the provisions for gravity-type quaywalls and sheet pile quaywalls classified as high earthquake-resistance facilities corresponding to the structural characteristics of respective members.

⑩ The verification items and standard indexes for determining the limit values for cellular-bulkhead quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to

Level 2 earthquake ground motions as the dominant action shall be equivalent to the provisions for gravity-type quaywalls classified as high earthquake-resistance facilities.

2.1.1 Dimensions of Wharves

(1) Dimensions of Wharves

① **Length**

The length of a wharf used in the performance verification of a wharf shall be set as the value obtained by adding the necessary lengths of the bow and stern mooring lines to the length overall of the design ship on the premise that the wharf is used exclusively by the design ship.

② **Water depth**

The water depth used in the performance verification of a wharf shall be set as the value obtained by adding the keel clearance corresponding to the design ship to the maximum draft such as the full load draft of the design ship to obtain a value that will not interfere with the use of the design ship.

③ **Crown height**

For the performance verification of the crown height of a wharf, the assumed use conditions of the facilities shall be considered for the safe and smooth use of the wharf.

④ **Ancillary equipment**

For the performance verification of a wharf, the ancillary equipment shall be considered for the safe and smooth use of the wharf. The performance requirements for the ancillary equipment of mooring facilities are stipulated in Article 33 of the Ministerial Ordinance on Criteria (Performance Requirements for the Ancillary Facilities of Mooring Facilities), and the performance criteria for the ancillary equipment are stipulated in Articles 60 to 74 of the Public Notice corresponding to the types of ancillary equipment.

⑤ **Shape of the wall and front toe**

In addition to the items provided here, for the performance verification of a wharf, the shape of the wall and front toe of the wharf (clearance limits of structure) shall be appropriately set so as to prevent them from coming into contact with the wharf during berthing.

(2) Length, Water Depth, and Layout of Berths

① The length and water depth of berths should be appropriately set based on the ship dimensions.

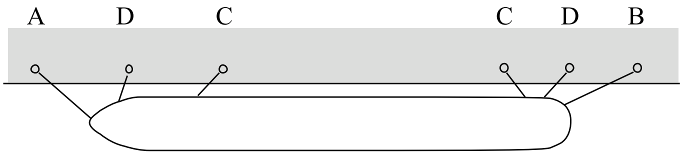
② The mooring lines shown in **Fig. 2.1.1** are desirable when a vessel is moored parallel to a wharf. The bow and stern lines are usually set at an angle of 30° to 45° with the quay face because these lines are used to prevent both the longitudinal movement (in the bow and stern directions) and lateral movement (in the onshore and offshore directions) of the vessel.

③ The water depth of berths can be calculated using **equation (2.1.1)**. The maximum draft represents the maximum draft in a calm water condition such as the condition when the ship is moored, e.g., full-load draft of the design ship. For keel clearance, a value of approximately 10% of the maximum draft or more is desirable. However, in mooring facilities where ships may moor for harborage in abnormal weather or similar conditions, the addition of a keel clearance that considers wind and wave factors is necessary.

Berth water depth = Maximum draft + Under keel clearance **(2.1.1)**

④ In the case of a berth where flammable dangerous cargoes are handled, it is necessary to keep a distance of 30 m or more between oil tanks, boilers, and working areas that use open fire in the cargo handling area and the mooring vessel at the berth. However, the distance may be shortened to approximately 15 m when there is no risk that the cargo may catch fire in the event of leakage because of the surrounding topography or structure of the facilities of the berth.

⑤ In the case of a berth where flammable dangerous cargoes are handled by tankers and so on, a distance of 30 m or more should be kept between tankers and other anchored vessels and the space for the vessels navigating nearby to navigate 30 m or more away from those tankers should be secured. However, this distance may be increased or decreased as necessary in consideration of the size of the cargo carrying vessel, the type and size of the vessels anchored or navigating nearby, and the condition of ship congestion.



A: bow line
B: stern line
C: spring lines
D: breast lines

Fig. 2.1.1 Arrangement of Mooring Ropes

⑥ **Standard values of wharf dimensions**

For setting the length and water depth of a wharf in cases where the design ship cannot be identified, the standard values of the main dimensions of wharves by ship type shown in **Table 2.1.1** can be used. These standard values have been set on the basis of the standard values of the main dimensions of the design ships shown in **Part II, Chapter 8 Ships, Table 1.1.1**. In principle, the standard values shown in **Table 2.1.1** have been set assuming that the design ship moors parallel to the wharf; however, the standard values for ferries have been also set by assuming cases of bow and stern-side berthing-type wharves in addition to that case. With regard to the standard values for small cargo ships, given that there are large deviations in comparison with other ship types, due consideration of this point is necessary when applying the standard values shown in **Table 2.1.1** in setting the length and water depth of wharves for small cargo ships, as well as applying the standard values of main dimensions of small cargo ships. The gross tonnage GT in **Table 2.1.1, 4** to **7** is either international gross tonnage or Japanese gross tonnage, and the gross tonnage used is specified at the table.

Table 2.1.1 Standard Values of the Main Dimensions of Berths in Cases Where the Design Ship Cannot Be Identified

1. Cargo Ships

Deadweight tonnage DWT (t)	Length of the berth (m)	Water depth of the berth (m)
1,000	80	4.5
2,000	100	5.5
3,000	110	6.0
5,000	130	7.0
6,000	140	7.5
10,000	160	9.0
12,000	170	9.0
15,000	180	10.0
18,000	190	11.0
30,000	230	12.0
40,000	250	13.0
50,000	260	14.0
55,000	270	15.0
70,000	280	16.0
90,000	310	17.0
120,000	340	19.0
150,000	360	20.0
200,000	390	22.0
250,000	420	23.0
300,000	430	25.0
400,000	470	26.0

2. Container Ships

Deadweight tonnage DWT (t)	Length of the berth (m)	Water depth of the berth (m)
10,000	170	9.0
20,000	220	11.0
23,000	230	12.0
27,000	240	13.0
30,000	250	13.0
40,000	290	13.0
50,000	330	14.0
60,000	350	15.0
100,000	410	16.0
140,000	440	17.0
165,000	470	18.0
185,000	500	18.0
200,000	500	18.0

3. Tankers

Deadweight tonnage DWT (t)	Length of the berth (m)	Water depth of the berth (m)
1,000	80	4.5
2,000	100	5.5
3,000	110	6.5
5,000	130	7.5
10,000	170	9.0
15,000	190	10.0
20,000	210	11.0
30,000	230	12.0
50,000	260	14.0
70,000	280	15.0
90,000	310	16.0
100,000	320	17.0
150,000	360	19.0
300,000	440	25.0

4. Roll-On Roll-Off (RORO) Ships

4.1 RORO Ships (GT: Japanese gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
3,000	150	6.5
5,000	180	7.5
10,000	220	9.0
15,000	220	9.0

4.2 RORO Ships (GT: international gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
20,000	240	10.0
40,000	250	11.0
60,000	270	12.0

5. Pure Car Carriers (PCC)

5.1 PCC (GT: Japanese gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
3,000	150	5.5
5,000	170	7.0
40,000	260	12.0

5.2 PCC (GT: international gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
12,000	180	7.5
20,000	200	8.0
30,000	230	9.0
40,000	240	11.0
60,000	260	12.0
70,000	290	12.0

6. Passenger Ships (GT: international gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
3,000	130	5.0
5,000	150	5.5
10,000	180	7.0
20,000	220	8.0
30,000	260	8.0
50,000	310	9.0
70,000	340	9.0
100,000	360	10.0
130,000	390	10.0
160,000	410	10.0

* In setting the berth lengths and water depths for passenger ships with the maximum dimensions, it is desirable to set them after acquiring ship dimensions by using the latest Lloyd's data and Clarkson data and so on. The maximum dimensions of the existing passenger ships and those planned to be built are length of 362.1 m, molded breadth of 47.0 m, and full-load draft of 10.3 m, as shown in **Part II, Chapter 8, 1.1 Standard Values, Table 1.1.5**.

7. Ferries (GT: Japanese gross tonnage)

7-1 Intermediate- and Short-Distance Ferries (sailing distance less than 300 km in Japan)

Gross tonnage GT (t)	Case of the bow and stern side berthing -type		
	Length of the berth (m)	Length of the bow and stern-side berthing-type quaywall (m)	Water depth of the berth (m)
400	60	20	3.5
700	80	20	4.0
1,000	90	25	4.5
3,000	130	25	5.5
7,000	170	30	7.0
10,000	200	30	7.5
13,000	220	35	8.0

7-2 Long-Distance Ferries (sailing distance of the 300 km or more in Japan)

Gross tonnage GT (t)	Case of no bow- and stern side–type berthing	Case of the bow and stern side type berthing		Water depth of the berth (m)
	Length of the berth (m)	Length of the berth (m)	Length of the bow and stern-side berthing-type quaywall (m)	
6,000	190	170	30	7.5
10,000	220	200	30	7.5
15,000	250	230	40	8.0
20,000	260	250	40	9.0

8. Small Cargo Ships

Deadweight tonnage DWT (t)	Length of the berth (m)	Water depth of the berth (m)
700	70	4.0

(3) Crown Height of Wharves

① The crown height of wharves shall be appropriately set in consideration of the subsequent items:

- Safe and smooth cargo handling and embarkation and disembarkation of passengers
- Relations between the freeboards and respective full-load and unloaded draft of design ships
- Uplift force on piled pier
- Possibility of inundation due to storm surge
- Possibility of inundation due to waves
- Possibility of inundation due to tsunamis
- Possibility of the consolidation settlement of the ground and predicted consolidation settlement
- Ease of inspections, diagnoses, and repair work in the maintenance phase (particularly for piled pier)
- Possibility of ground subsidence due to crustal movement after large-scale earthquakes
- Others

② The mean monthly-highest water level can be used as the tidal level used for the datum level of the crown height of wharves.

③ In cases where the design ship cannot be identified, in general, the values shown in **Table 2.1.2** are widely used as the crown height of wharves. It should be noted that the values in this table are expressed using the mean monthly-highest water level as a datum level.

Table 2.1.2 Standard Crown Heights of Wharves

	Tidal range 3.0 m or more	Tidal range less than 3.0 m
Wharf for large vessels (water depth of 4.5 m or more)	+0.5 to 1.5 m	+1.0 to 2.0 m
Wharf for small vessels (water depth of less than 4.5 m)	+0.3 to 1.0 m	+0.5 to 1.5 m

(4) Clearance Limits of Wharves

① The shape of the wall and front toe of the wharves shall be appropriately set so as to prevent them from coming into contact with ships during berthing.

② The clearance limits of wharves shall be set so that they comply with the Sounding Procedure based on the **Implementation Procedure[1] of the Memorandum of Understanding on Hydrographic Survey Associated with Port Construction** between the Ports and Harbours Bureau and the Japan Coast Guard (March 31, 1972).

③ In the cross sections of a vessel, the bottom-corner sections are slightly rounded and have projecting bilge keels. In many cases, the radius of curvature of the corner sections and the height of the bilge keels are 1.0 to 1.5 m and 30 to 40 cm, respectively and the shape of corner sections may be assumed to be nearly 90°. The planned water depths of berths are generally deeper than the maximum draft of the design vessel by 0.3 m or more.

④ According to the **Implementation Procedure of the Memorandum of Understanding on Hydrographic Survey Associated with Port Construction**, water depth in the vicinity of wharves shall be surveyed as far as 1 m from fenders.

⑤ **Fig. 2.1.2** shows the clearance limit for wharves set in consideration of the above items and past examples.[2), 3)] The clearance limit of wharves may be determined using this figure as reference. However, care should be exercised in using the clearance limit shown in the figure because the rolling, pitching, and heaving motions of vessel at berth have not been taken into consideration in the figure.

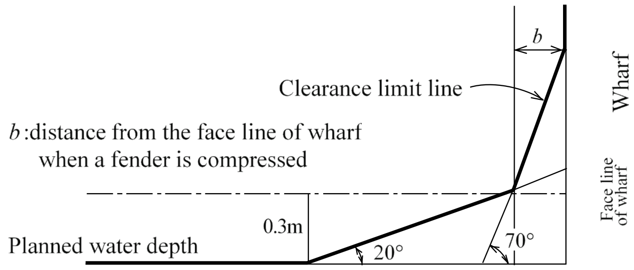


Fig. 2.1.2 Clearance Limit for Mooring Facilities

(5) Design Water Depth

① In general, the design water depth is not equal to the planned water depth. The design water depth is ordinarily obtained by adding a margin to the planned water depth to guarantee the required stability of the mooring facility. Considering that this margin will vary according to the structural type, the water depth of the site, the construction method and accuracy, and the result of scouring, it is desirable that the design water depth is carefully determined with due consideration given to these factors.

② It may be difficult to determine the depth of scouring owing to current or the berthing of specific vessels when setting the design water depth even with the risk of large scouring in front of mooring facilities. In such a case, it is advisable to provide scour prevention measures as described in **Part III, Chapter 5, 2.1.2 Protection against Scouring** instead of considering the depth of scouring as margins.

2.1.2 Protection against Scouring

(1) In cases wherein large scouring is anticipated at the front side of a wharf owing to the currents or turbulence generated by ship propellers, the front of the mooring facility shall be protected with armor stones, concrete blocks, or other materials as a measure against scouring.

(2) In the case that ships may drop anchors at the front side of a wharf, it is necessary to focus on protection area against scouring and on the appropriate selection of scouring protection materials to prevent anchors from dragging.

(3) Examples of overseas test models of the measures against scouring include armor rocks, gabions, a layer of concrete blocks on a geotextile filter, a layer of pillow-like geotextile bags filled with concrete, etc. Furthermore, there has been an inventive plan to install a deflector (curved plate) on a sea bottom to deflect the water current due to propellers toward a sea surface.[4)]

(4) When deliberating measures against scouring, it is advisable to refer to the current velocity calculation formula[5)] and the calculation formula of the required mass of foundation materials[6)] based on the Isbash formula (**Part II, Chapter 2, 6.6.3 Required Mass of Armor Stones and Blocks to Resist Currents**).

2.1.3 Green Quaywalls

(1) Green quaywalls[7)] are a type of quaywalls that contribute to the development of a favorable port environment and hospitable habitats such as tidal flats and rocky shores for organisms in accordance with natural situations wherein quaywalls are located (**Reference (Part I), Chapter 3, 2 Green Port Structures**). Green quaywalls can also be developed by adding the habitat functions to existing quaywalls concurrently with their improvement works.

(2) Environmental research and numerical model analyses shall be employed when identifying the influences on the goal of creating habitats for organisms (**Reference (Part I), Chapter 3, 2 Green Port Structures**). When verifying the performance of green quaywalls, it is necessary to confirm whether structures, cross sections, and ancillary facilities are expected to achieve the goal.

(3) The performance requirement for green quaywalls is that the quaywalls should have the habitat function, and its influences are the presence or absence of foundations related to the inhabitation of organisms; external forces such as waves, currents, etc.; and environments that are necessary for the inhabitation of organisms. The environments necessary for the inhabitation of organisms refer to water depth and water transparency, which affect the light intensity necessary for photosynthesis and water temperature, which affects the activities of organisms. Specifically,

when aiming at the cultivation of a seaweed bed, the structure and cross section of a quaywall and the texture and gradient of ancillary facilities need to allow objective seaweed and sea grass to root and maintain the structural sun light shading effects to a level that does not affect the light intensity necessary for seaweed and sea grass to grow.

(4) The performance verification of green quaywalls shall be implemented in a manner that confirms whether the environments of the locations are within the inhabitable range of objective organisms on the basis of existing knowledge. For example, in the performance verification of a quaywall with the inhabitation of seaweed, the performance verification shall be implemented in a manner that confirms whether such environments are within the inhabitable range of objective seaweed because seaweed is subject to the environments represented by light intensity affecting photosynthesis and respiration, as well as water temperature. Furthermore, in the case where changes in the environmental conditions after the installation of green quaywalls and environmental variations in the future are predictable, the performance of a quaywall can be verified by confirming whether such changes or variations in the future are within the inhabitable range of living organisms on the basis of a numerical model related to the growth of organisms.

(5) The performance verification of green quaywalls shall be implemented by referring to **Part III, Chapter 4, 4 Green Breakwaters** and **Reference (Part I), Chapter 3, 2 Green Port Structures** and the **Guidelines for the Development, Maintenance and Management of Green Port Structures**.[7)]

2.2 Gravity-type Quaywalls

[Public Notice] (Performance Criteria for Gravity-type Quaywalls)

Article 49

The performance criteria for gravity-type quaywalls shall be as prescribed respectively in the following items:

(1) The risk of sliding failure of the ground under the permanent state, in which the dominating action is self-weight, shall be equal to or less than the threshold level.

(2) The risk of failure due to the sliding or overturning of the quaywall body and the insufficient bearing capacity of the foundation ground under the permanent state, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion, shall be equal to or less than the threshold level.

[Interpritation]

11. Mooring Facilities

(3) Performance Criteria of Gravity-type Quaywalls (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance on Criteria and the interpretation related to Article 49 of the Standard Public Notice)

① The required performance of gravity-type quaywalls under the permanent state in which the dominant actions are self-weight and earth pressure and under the variable state in which the dominant actions are Level 1 earthquake ground motions shall focus on serviceability. **Attached Table 11-5** shows the performance verification items and standard indexes for determining limit values with respect to the actions.

Attached Table 11-5 Performance Verification Items and Standard Indexes to Determine the Limit Values under the Respective Design Situations of Gravity-type Quaywalls except Accidental Situations

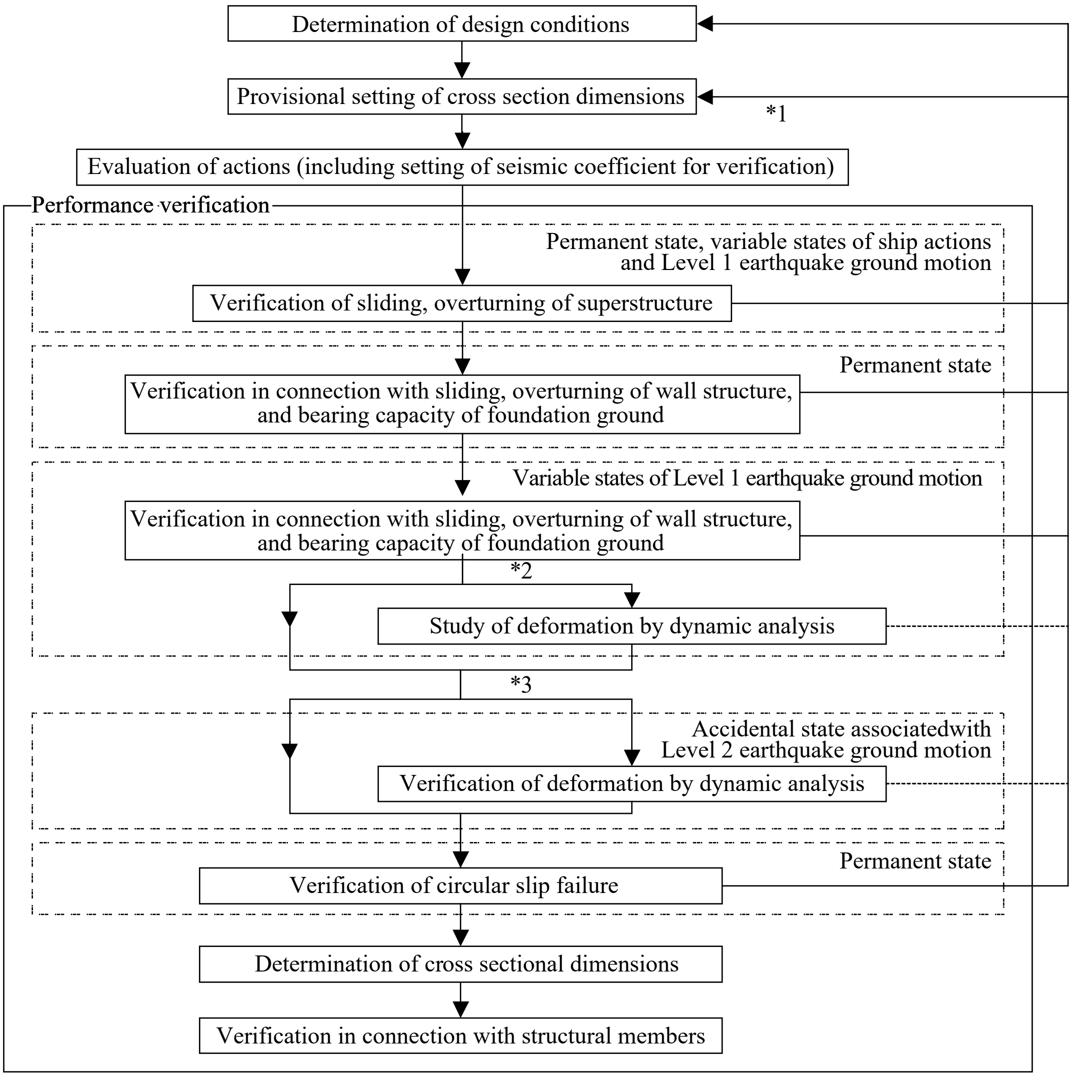
<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index to determine the limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="3">26</td><td rowspan="3">1</td><td rowspan="3">2</td><td rowspan="3">49</td><td rowspan="3">–</td><td>1</td><td rowspan="3">Serviceability</td><td rowspan="2">Permanent</td><td>Self-weight</td><td>Water pressure, surcharges</td><td>Circular slip failure of the ground</td><td>Action–resistance ratio with respect to circular slip failure</td></tr>
<tr><td rowspan="2">2</td><td>Earth pressure</td><td>Self-weight, water pressure, surcharges</td><td>Sliding, overturning of the quaywall, bearing capacity of the foundation ground</td><td>Action–resistance ratios with respect to sliding, overturning, and bearing capacity</td></tr>
<tr><td>Variable</td><td>Level 1 earthquake ground motion</td><td>Self-weight, earth pressure, water pressure, surcharges</td><td>Sliding, overturning of the quaywall, bearing capacity of the foundation ground</td><td>Action–resistance ratios with respect to sliding, overturning, and bearing capacity</td></tr>
</table>

② In addition to this code, the provisions and commentaries in connection with Paragraph 3 (Scouring and Wash Out), Article 22 and Article 28 (Performance Criteria of Armor Stones and Blocks) of the Standard Public Notice shall be applied as necessary, and the provisions and commentaries in connection with Article 23 and Article 27 of the Standard Public Notice shall be applied depending on the type of members comprising the objective gravity-type quaywalls.

2.2.1 General

(1) Depending on the types of wall structures, gravity-type quaywalls are classified as caisson-type quaywalls, L-shaped block-type quaywalls, mass-concrete block-type quaywalls, cellular concrete block-type quaywalls, cast-in-place concrete-type quaywalls, upright wave-dissipating-type quaywalls, and others. The description provided herein can be applied to the performance verification of these gravity-type quaywalls. Regarding upright wave-absorbing-type quaywalls, the performance verification method shown in **Part III, Chapter 5, 2.11 Upright Wave-absorbing-type Quaywalls** can be used as a reference.

(2) **Fig. 2.2.1** shows an example of the performance verification procedure for gravity-type quaywalls. However, because the figure does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions, when examining the effects of liquefaction, it is necessary to appropriately deliberate the possibility of and the measures against liquefaction with reference to **Part II, Chapter 7 Ground Liquefaction**. The seismic coefficient method based on the static equation of equilibrium can be used to examine gravity-type quaywalls under a variable state with respect to Level 1 earthquake ground motions. By contrast, for gravity-type quaywalls classified as high earthquake-resistance facilities, the deliberation of deformation amounts is preferably performed by nonlinear earthquake response analysis or other methods by taking into consideration the dynamic interaction between ground and structures. For those gravity-type quaywalls other than high earthquake-resistance facilities, the performance verification for an accidental situation with respect to Level 2 earthquake ground motions can be omitted.



*1 As the effects of liquefaction, subsidence etc. are not included in this procedure, separate consideration is necessary.

*2 When necessary, study of deformation by dynamic analysis for Level 1 earthquake ground motion is possible. In high-earthquake-resistance facilities, study of deformation by dynamic analysis is preferable.

*3 For high-earthquake-resistance facilities, verification for level 2 earthquake ground motion is performed.

Fig. 2.2.1 Example of Performance Verification Procedure for Gravity-type Quaywalls

(3) **Fig. 2.2.2** shows an example of a cross section of a gravity-type quaywall.

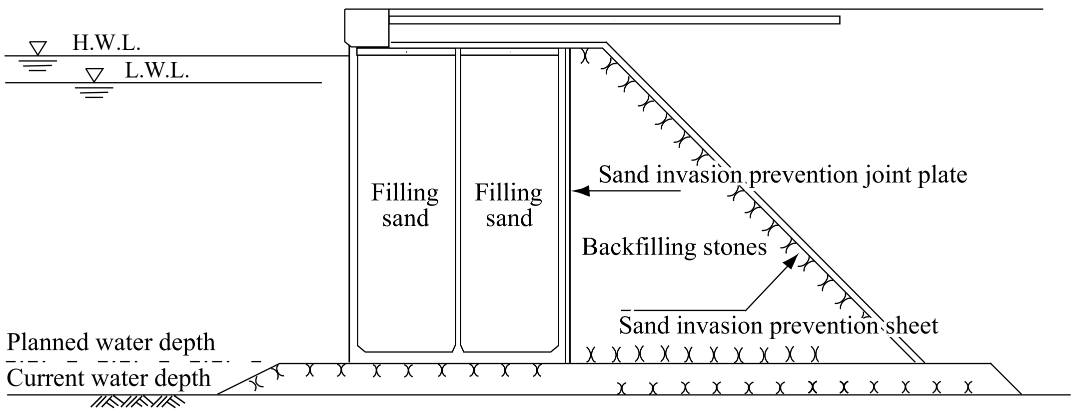


Fig. 2.2.2 Example of a Cross Section of a Gravity-type Quaywall

(4) For the gravity-type quaywalls of special structural types, it is preferable to perform performance verification by using laboratory experiments and numerical analyses.

For example, so-called caisson-type quaywalls with inclined bottom surfaces constructed on seafloors inclined landward may have a large resistance against sliding but require sufficient performance verification with respect to increased bottom reaction force and rocking due to earthquake ground motions via laboratory experiments and numerical analyses. Clearance limits should also be considered. For the performance verification of caisson-type quaywalls with inclined bottom surfaces, refer to **References 8)** and **9)**.

2.2.2 Actions

(1) Types of Actions to Be Considered in Respective Design Situations

The stability verification of gravity-type quaywalls shall consider the following actions in respective design situations. The performance verification for accidental situation can be omitted in the case where the gravity-type quaywalls to be designed are not high-earthquake-resistance facilities.

① **Permanent state**

The dominant actions shall be the self-weight of wall bodies and earth pressure acting on wall bodies.

② **Variable state**

The dominant actions shall be Level 1 earthquake ground motions. For the setting of Level 1 earthquake ground motions, refer to **Part II, Chapter 6, 1.2 Level 1 Earthquake Ground Motions Used for Performance Verification of Facilities**.

③ **Accidental state**

The dominant actions shall be Level 2 earthquake ground motions. For the setting of Level 2 earthquake ground motions, refer to **Part II, Chapter 6, 1.3 Level 2 Earthquake Ground Motions Used for Performance Verification of Facilities**.

(2) Points of Caution When Setting Actions

① Seismic coefficients for verification used in the verification of damage due to sliding and overturning of wall bodies and failures due to insufficient bearing capacity of foundation ground in variable state in respect of Level 1 earthquake ground motion[10), 11)]

(a) For the performance verification with respect to the sliding and overturning of wall bodies, as well as failures due to insufficient bearing capacity under a variable state with respect to Level 1 earthquake ground motions, the seismic coefficient method can also be used in place of the direct evaluation of deformation amounts by using nonlinear response analyses. In such a case, the seismic coefficients for verification to be used in performance verification need to be appropriately set in accordance with the deformation amounts of facilities by taking into consideration the effects of frequency characteristics and duration of earthquake ground motions.

(b) Calculation of the characteristic value of the seismic coefficient for verification

The characteristic values of the seismic coefficients for verification to be used for the performance verification of gravity-type quaywalls installed at depths of -7.5 m or deeper can be calculated with **equation (2.2.1)** by using the maximum corrected acceleration α_c stipulated in **Reference [Part III), Chapter 1, 1 Detailed Items about Seismic Coefficient for Verification** and the allowable deformation amounts D_a of the crowns of quaywalls. The seismic coefficients for verification should be expressed in numerical values rounded off to two decimal places. However, for the calculation of the seismic coefficients for verification in the case of ground improvement through the deep mixing method or the sand compaction pile (SCP) method with a replacement rate of 70% or higher, refer to **Part III, Chapter 2, 5.5 Deep Mixing Method** and **Part III, Chapter 2, 5.10 Sand Compaction Pile Method (for Cohesive Ground)**.

$$k_{h_k} = 1.78\left(\frac{D_a}{D_r}\right)^{-0.55}\frac{\alpha_c}{g} + 0.04 \qquad \textbf{(2.2.1)}$$

where

k_{h_k} : characteristic value of a seismic coefficient for verification;

α_c : maximum corrected acceleration (cm/s^2);

g : gravitational acceleration (980 cm/s^2);

D_a : allowable deformation amount of the crown of a quaywall (10 cm);

D_r : reference deformation amount (10 cm).

By contrast, for the characteristic values of the seismic coefficients for verification k_{h_k} to be used for the performance verification of gravity-type quaywalls installed at depths of less than -7.5 m, refer to the seismic coefficients for the verification of the **Reference for Design of Fishery Ports and Fishing Ground Facilities, Chapter 6, 2.2.3 Design Horizontal Seismic Coefficients That Take into Consideration Frequency Characteristics and Deformation Amounts**.[12)] For those gravity-type quaywalls installed at depths of approximately -7.5 m, the seismic coefficients are preferably set in consideration of the possible discrepancies between those calculated via **equation (2.2.1)** and the methods in **Reference 12)**.

(c) Setting of the allowable amount of deformation

It is necessary to appropriately set the allowable amount of deformation for facilities in accordance with the functions required for the facilities and circumstances in which the facilities are placed. The allowable value of the standard deformation amount of gravity-type quaywalls in Level 1 earthquake ground motion may be taken to be D_a = 10 cm. This allowable value of the standard deformation amount (D_a = 10 cm) is the average value of the amounts of residual deformation of existing gravity-type quaywalls in Level 1 earthquake ground motion, which was calculated by seismic response analyses. The standard deformation amount is determined by taking into consideration the margin of safety necessary to ensure that the seismic performance verification method is accurate enough to prevent the functions of facilities from being impaired by Level 1 earthquake ground motions. Thus, the standard deformation amount is set to be sufficiently smaller than the allowable limit deformation amounts of actual facilities.

(d) The calculation method for the characteristic value of the seismic coefficient for verification in (b) above is based on the condition allowing for no liquefaction. When applying the method to other conditions, the applicability needs to be deliberated via 2D seismic response analyses or model experiments.

(e) The calculation method for the characteristic value of the seismic coefficient for verification in (b) above is based on the allowable deformation amounts D_a in the range of 5 to 20 cm. Therefore, attention is required when applying the method to the allowable deformation amounts in other ranges.

(f) In some areas where small values have been set for Level 1 earthquake ground motions, this calculation method may produce significantly small seismic coefficients for verification. Even in those areas, the seismic coefficients for verification shall be set at the lower limit value of 0.05 by taking into consideration the uncertainty of the hazard analyses used for obtaining Level 1 earthquake ground motions, the accuracy of the calculation method for the seismic coefficients for verification, and the setting method for allowable deformation amounts.

(g) Considering that the above calculation method may result in excessively large seismic coefficients for verification, any of the following measures can be taken when the calculation produces values larger than 0.25 provided that deformation is preferably confirmed directly by dynamic analyses or other methods even when any of the measures in 2) to 4) is taken:

1) Setting of a cross section with a seismic coefficient for verification of 0.25 and evaluation of the cross section by dynamic analysis that is capable of dealing with a dynamic mutual interaction between the ground and a structure

2) Deliberation of ground improvement

3) Adoption of another structural type

4) Deliberation of the changing allowable deformation D_a of a facility without excessively enlarging it while satisfying the requirement to keep the damage due to Level 1 earthquake ground motions to a

gravity-type quaywall within a level that enables the function of the facility and prevents the impairment of its continuous use.

(h) When implementing the deep mixing method or the SCP method with a replacement rate of 70% or higher, the calculation method for the characteristic value of the seismic coefficient for verification can be applied to the performance verification of gravity-type quaywalls provided that an appropriately set reduction coefficients are used. The reduction coefficients are determined on the basis of the comparison of the 2D effective stress analysis results between unimproved and improved ground. For the details of the reduction coefficients, refer to **Part III, Chapter 2, 5.5 Deep Mixing Method** and **Part III, Chapter 2, 5.10 Sand Compaction Pile Method (for Cohesive Ground)**.

(i) When constructing quaywalls without improving very soft, normally consolidated, cohesive soil layers, the calculation method for the characteristic value of the seismic coefficient for verification may underestimate the seismic coefficients for verification from the viewpoint of deformation amounts. Therefore, in such cases, the deformation amounts should be evaluated directly by using detailed methods, such as nonlinear effective stress analyses.

(j) Soft ground is subjected to large shear strain when oscillated by strong earthquake ground motions, thus aggravating damage to quaywalls. However, in some cases, analyses do not produce large earthquake ground motions on ground surfaces and underestimate the seismic coefficients for verification. Therefore, the 1D earthquake response analysis codes to be used for verification should be able to appropriately evaluate the amplification of earthquake ground motions in soft ground, particularly the amplification of the acceleration in the frequency ranges critical for calculating the seismic coefficients for verification.

(k) When applying the seismic coefficient method to performance verification by using seismic coefficients for verification in a vertical direction, such coefficients shall be appropriately set in accordance with the characteristics of the facilities and ground.

(l) The seismic coefficients should be calculated for verification on the basis of appropriate deliberation before their use in the performance verification of structural members under an accidental situation with respect to Level 2 earthquake ground motions. Regarding the damage to caisson quaywalls in Kobe Port due to the 1995 South Hyogo Prefecture Earthquake, the failure of bottom slabs of caissons has not been reported even though they underwent large deformation. Furthermore, there is little knowledge on the seismic coefficients for verification with respect to Level 2 earthquake ground motions. Therefore, the seismic coefficients for verification to be used for the performance verification of structural members under an accidental situation with respect to Level 2 earthquake ground motions can be calculated by method (b) above by using the acceleration time history of the ground surface of the free ground area for convenience. In such a case, the allowable deformation amount D_a can be set to 50 cm. However, the seismic coefficients for verification shall be up to 0.25 and equal to or larger than the seismic coefficients for verification for Level 1 earthquake ground motions. When the seismic coefficients for verification for Level 1 earthquake ground motion are higher than 0.25, the seismic coefficients for verification for Level 2 earthquake ground motions can also be higher than 0.25.

② **Determination of wall body portions**

(a) In cases wherein stability needs to be verified by substituting inertia force for the actions of earthquake ground motions, it is necessary to assess the inertia force on the basis of the appropriate determination of the quaywall bodies. In such cases, the quaywall bodies can be set as shown below depending on the types of structures. This setting of quaywall bodies shall not be applied to cases wherein deformation amounts are assessed directly by a detailed method such as nonlinear effective stress analysis or similar methods.

(b) **Fig. 2.2.3** shows that the wall bodies of gravity-type quaywalls can be defined as the portions between the face lines of the quaywalls and the vertical planes passing through the rear toes of the quaywalls. Normally, wall bodies are provided with backfills behind them. In many gravity-type quaywalls, some parts of the backfill are positioned above the wall bodies and act as parts of the quaywall bodies. However, it is difficult to apply this concept to all cases unconditionally because the extent of backfill that is considered part of the quaywall bodies varies depending on the shapes of the quaywall bodies and the modes of failures. In general, the extent of backfill that is considered part of the quaywall bodies can be defined by the shaded area in **Fig. 2.2.3** to simplify the design calculation because modest changes in the locations of the quaywall body boundary planes do not affect the stability of the quaywall bodies significantly.

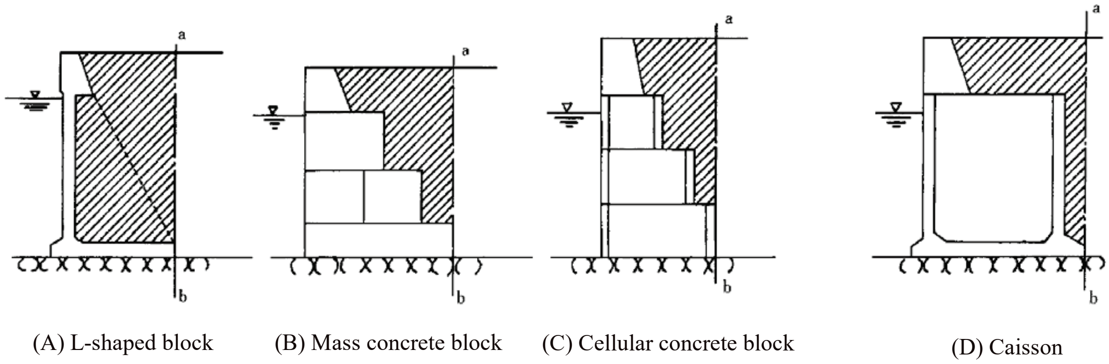


Fig. 2.2.3 Determination of Wall Body Portions

(c) In cases wherein quaywall structures require stability during the examination of respective horizontal strata, similar to the case for block-type quaywalls, the determination of virtual wall bodies may be performed as follows. Normally, tenons are provided between blocks for better interlocking; however, the interlocking effects of the tenons are preferably ignored in the examination of the following virtual wall bodies.

1) Examination of sliding

Fig. 2.2.4 shows that the portion in front of the vertical plane passing through the rear toe at the level under examination can be considered a wall body.

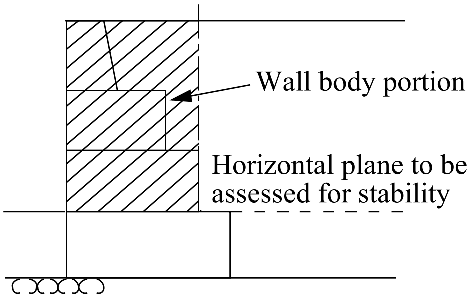


Fig. 2.2.4 Determination of Wall Body Portion for the Stability of Sliding at a Horizontal Joint

2) Examination of Overturning

The backfill in front of a vertical plane passing through the most landward side edge among blocks stacked on a block placed on the seaward side above the plane subject to stability examination may be regarded as a part of the wall body. For example, in the case of a block-type quaywall (**Fig. 2.2.5**), the weight of the portion in front of the vertical plane (shown by hatched lines) through the block placed on block Ⓒ on the seaward side can be considered to resist overturning; however, the weights of block Ⓑ and soil Ⓐ are not considered to contribute to overturning resistance.

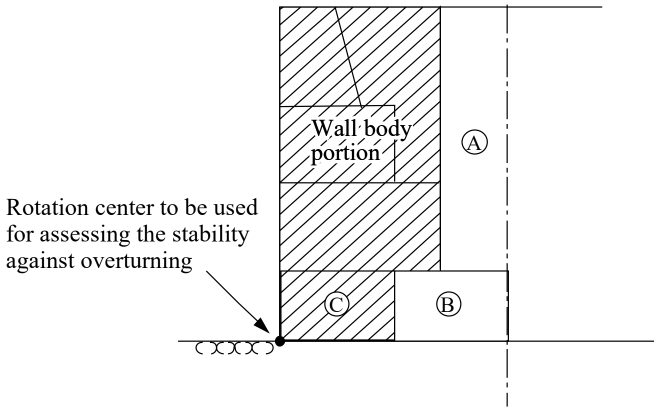


Fig. 2.2.5 Determination of Wall Body Portion for Stability against Overturning

3) Examination of Failure due to the Inadequate Bearing Capacity of Foundation Ground

In examining failures, the portion in front of the vertical plane passing through the rear toe of a wall body can be considered a virtual wall body. In the case of a cellular block wherein the wall body and a filling section appear to have different bottom reactions, the lowermost block shall preferably be constructed as an integrated block.

③ The residual water level should be set at a level one-third of the tidal range above the mean monthly lowest water level (LWL). For the design tidal levels, refer to **Part II, Chapter 2, 3.6 Design Tidal Level Conditions**. In general, the range of the residual water level difference increases as the tidal range increases and as the permeability of the wall body material decreases. Water behind the wall body permeates through voids in the wall joints, foundation mound, and backfill. The residual water level difference can be reduced by improving the permeability of these materials. On the contrary, care is necessary because this approach may result in the leakage of the backfill material.

The abovementioned value of the residual water level is applicable to cases in which long-term permeability can be secured. In cases wherein permeability is low from the initial stage or permeability reduction is expected to be reduced over a long-term, it is preferable to assume a large residual water level difference in consideration of these conditions.

A residual water level difference may occur when wave troughs hit the front face of a wall body in general; however, it is not necessary to consider the increase in the residual water level difference due to wave attacks in the performance verification of quaywalls.[13)]

④ For the wall friction angle, δ = 15° can be used. For L-shaped blocks, the shear resistance angle of the backfilling material at the virtual back plane can be used. For details, the **Technical Manual for L-Shaped Block Quaywalls**[14)] may be used as a reference.

⑤ The surcharge may be determined in accordance with **Part II, Chapter 10, 3 Surcharge**.

⑥ Buoyancy is affected by numerous indeterminate factors. Therefore, it is preferable to set buoyancy by considering the worst-case scenario for the facilities concerned. For example, as shown in **Fig. 2.2.6**, buoyancy may be calculated for the submerged portion of the wall body below the residual water level. This approach is applicable to cases in which the difference between the front water level and residual water level is within a normal level; in cases wherein the difference in water levels is remarkable, buoyancy must be set appropriately on the basis of the natural conditions of the objective facilities concerned and other relevant factors.

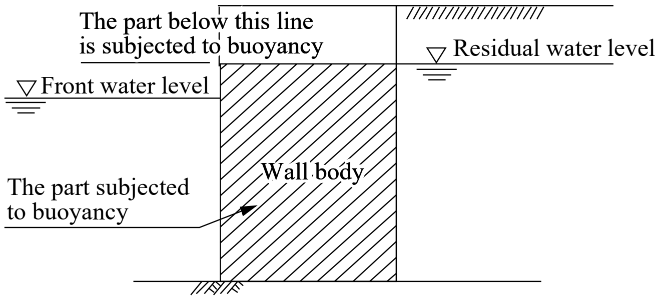


Fig. 2.2.6 Assumption for Calculating Buoyancy

⑦ To obtain earth pressure during earthquake ground motions, it is normal practice to use the equations for the calculation of earth pressure proposed by Mononobe and Okabe, which is shown in **Part II, Chapter 4, 2.3 Earth Pressure during Earthquake**. However, this is based on the concept of the seismic coefficient method, and the calculation results differ from the actual earth pressures resulting from the dynamic interaction of structures, soil, and water. Shaking table tests have shown that the inertial force of wall bodies and the earth pressure during earthquake ground motions have phase differences because they oscillate in the opposite phase when the ground is dense and in the same phase when the ground is loose, e.g., due to liquefaction. In principle, liquefaction is not considered in the performance verification under variable situation with respect to Level 1 earthquake ground motions. Therefore, it is necessary to consider that the inertial force of wall bodies and the earth pressure during earthquake ground motions have opposite phases.[15)] The seismic coefficients for verification explained above allow performance verification to be made in accordance with the deformation of quaywalls by taking into consideration the differences in phases.

⑧ **Dynamic water pressure acting on wall bodies during earthquake ground motions**

For the dynamic water pressure acting during an earthquake, refer to **Part II, Chapter 4, 3.2 Dynamic Water Pressure**.

⑨ **Earth pressure reduction effect by backfill**

In cases wherein high-quality backfill is placed (e.g., a backfill material with a shear resistance angle of 40° is used for rubble), the earth pressure reduction effect by the backfill can be obtained using an analytical method (calculation of earth pressure by discrete method) that takes into consideration the composition of the soil behind the wall body and the strength of each layer behind the quaywall.[16)] In ordinary gravity-type quaywalls, rubble or cobble stones are used as the backfill material. In this case, the earth pressure reduction effect may be assessed using the following simplified method[17)]:

(a) When the cross section of a backfill is triangular

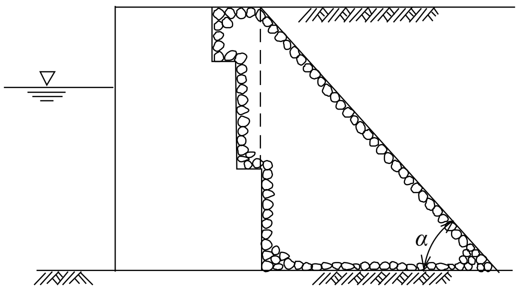
When a backfill is laid in a triangular shape from the point of intersection of the vertical line passing through the rear toe of a quaywall and the ground surface with an angle of slope less than the angle of repose α of the backfill material (**Fig. 2.2.7(a)**), the rear side of wall is entirely filled with backfill material. When the reclaiming material is slurry like cohesive soil, the application of filling-up work or the installation of sand invasion prevention sheets to the surface of the backfill shall be used to prevent the slurry cohesive soil from passing through the voids in the backfill and from reaching the quaywall.

(b) When the cross section of the backfill is rectangular

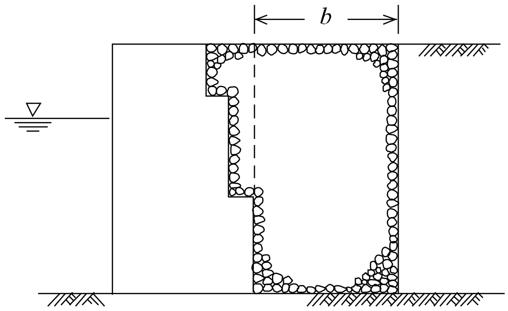
In the case of a triangular-shaped backfill with a slope steeper than the angle of repose of the backfill material or any other irregular-shaped backfill, the effect may be considered similar to a case with a rectangular-shaped backfill that has an area equivalent to the backfill in question. The effect of the rectangular backfill shown in **Fig. 2.2.7(b)** may be considered as follows.

1) When width b of the rectangular-shaped backfill is larger than the height of the wall, this case should be considered in the same manner as the case with a triangular backfill in **Fig. 2.2.7(a)**. When width b is equal to 1/2 of the height, it shall be assumed that the earth pressure is equivalent to the mean of the earth pressure due to the backfill and the reclaimed soil.

2) If the width b is 1/5 or less of the height of the wall, the earth pressure reduction effect due to the backfill shall not be considered.



(a) Triangular shape backfill



(b) Rectangular shape backfill

Fig. 2.2.7 Shapes of Backfill

2.2.3 Performance Verification

(1) Performance Verification Items

When conducting performance verification for the overall stability of structures on the basis of the static equation of equilibrium under a permanent state with respect to self-weight and under a variable state with respect to earth pressure and Level 1 earthquake ground motions as well as performance verification of structures under an accidental situation with respect to Level 2 earthquake ground motions, the necessary performance verification items shall be appropriately set with reference to **Part III, Chapter 5, 2.2 Gravity-type Quaywalls, [Interpretation], Attached Table 11-5** and to **Part III Chapter 5, 2.1 Common Items for Wharves, [Interpretation], Attached Tables 11-1 and 11-2**. When conducting the performance verification of structures under a variable state with respect to Level 1 earthquake ground motions by using nonlinear effective stress analyses, the performance verification items shall be set similar to the performance verification of structures under an accidental situation with respect to Level 2 earthquake ground motions. Furthermore, when gravity-type quaywalls to be designed are not high earthquake-resistance facilities, the performance verification for accidental situation can be omitted.

(2) Performance Verification for the Overall Stability of Structures under a Permanent State with respect to Self-weight

① Examination of the sliding failure of the ground

(a) In cases wherein the foundation ground is weak, circular slip failure from an arbitrary point behind the intersection of the vertical plane through the rear toe of the wall and the bottom plane of the rubble may be examined.

(b) The verification of the circular slip failure of the foundation ground under the permanent situation with respect to self-weight can be performed using **equation (2.2.2)**. In this equation, subscripts *k* and *d* indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.2.1**, in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$
\begin{aligned}
& m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\
& R_k = \sum \left[\{ c'_k\, s + (w'_k + q_k)\cos^2\theta \tan\phi'_k \} \sec\theta \right] \\
& S_k = \sum \left\{ (w'_k + q_k + q_{RWL_k}) \sin\theta \right\}
\end{aligned}
\tag{2.2.2}
$$

where

c' : undrained shear strength for cohesive soil ground or apparent adhesion under a drained condition for sandy soil ground (kN/m^2);

s : width of a segment (m);

w' : effective weight of a segment (kN/m) (atmospheric weight when above the water surface or underwater weight when below the water surface);

q : surcharge acting on a segment (kN/m);

q_{RWL} : weight of water, i.e., $\rho_w g(\mathrm{RWL} - \mathrm{LWL})s$, in a segment corresponding to the difference in water levels between the residual water level (RWL) at the back of a facility and a tidal level (LWL) in front of a facility in a case where RWL is higher than LWL (kN/m);

ϕ' : apparent shear resistance angle based on effective stress (°);

θ : angle between the bottom face of a segment and a horizontal plane (°) (Refer to **Part III, Chapter 2, 4 Slope Stability**);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.1 Partial Factors Used for the Performance Verification of the Circular Slip Failure of Foundation Ground

Verification object	Coefficient of variation of cohesive soil in a representative soil layer *CV*	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor *m*
Circular slip failure of foundation ground (Permanent state)	Case of no cohesive soil in the layer where a circle passes through	0.83	1.01	– (1.00)
	Less than 0.10	0.86	1.05	– (1.00)
	Not less than 0.10 and less than 0.15	0.85	1.04	– (1.00)
	Not less than 0.15 and less than 0.25	0.80	1.02	– (1.00)
	Not less than 0.25	– (1.00)	– (1.00)	1.30

(c) The partial factors shown in **Table 2.2.1** have been set with reference to the safety levels in the past standards.[18] Furthermore, the *CV*s of cohesive soil in the table can be determined using the *CV*s corresponding to correction factor b_1 obtainable in the process of calculating the characteristic values of adhesion in **Part II, Chapter 3, 2.1 Estimation of the Physical Property of the Ground**. In such a case, among the soil layers (excluding thin ones) where circles can pass through, the soil layer that has the largest *CV* can be the representative soil layer.

(d) Regarding the partial factors for circular slip failure, when the objective ground is subjected to soil improvement using SCP with a replacement rate of 30–80% under the wall body, those partial factors shown in **Part III, Chapter 2, 5.10 Sand Compaction Pile Method (for Cohesive Ground)** can be used.

(3) Performance Verification of the Overall Stability of Structures under a Permanent State with respect to Earth Pressure and Variable State in respect of Level 1 Earthquake Ground Motions

① Examination of sliding of wall bodies

The examination of the stability of wall bodies against sliding can be performed using **equation (2.2.3)**. In this equation, subscripts *k* and *d* indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.2.2** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience. The partial factors (permanent state) shown in the table have been set with reference to the safety levels in the past standards.[19]

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$R_k = f_k (W_k + P_{Vk} - P_{Bk}) \tag{2.2.3}$$

$$S_k = P_{Hk} + P_{wk} + P_{dwk} + P_{Fk}$$

where

f : *a* friction coefficient between the bottom face of a wall body and a foundation;

W : weight of the materials constituting a wall body (kN/m);

P_V : resultant vertical earth pressure acting on a wall body (kN/m);

P_B : buoyancy acting on wall (kN/m);

P_H : resultant horizontal earth pressure acting on a wall body (kN/m);

P_w : resultant residual water pressure acting on a wall body (kN/m)

P_{dw} : resultant dynamic water pressure acting on a wall body (kN/m) (only during earthquakes);

P_F : inertia force acting on a wall body (kN/m) (only during earthquakes);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.2 Partial Factors Used for the Performance Verification of the Sliding of Wall Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of a wall body (Permanent state)	0.87	1.06	– (1.00)
Sliding of a wall body (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.00

The characteristic values of the resultant dynamic water pressure P_{dw} in **equation (2.2.3)** can be calculated with the following equations:

$$P_{dw_k} = \frac{7}{12} k_{h_k} \rho_w g h^2 \tag{2.2.4}$$

$$P_{F_k} = k_{h_k} W_k \tag{2.2.5}$$

where

ρ_w : density of seawater (t/m^3);

g : gravitational acceleration (m/s^2);

h : water depth in front of a wall body (depth from the bottom face of a wall body to the water level in front of a wall body) (m);

k_h : seismic coefficient for verification.

Furthermore, when a caisson has footings with rectangular cross sections on both sea and land sides, the characteristic value of buoyancy can be calculated using the following equation.

$$P_{B_k} = \rho_w g \left[\left(wl_k + h' \right) B + 2 h_f B_f \right] \tag{2.2.6}$$

where

ρ_w : density of seawater (t/m^3);

g : gravitational acceleration (m/s^2);

wl : residual water level (m);

h' : installation depth of a wall body (m);

B : width of a wall body (m);

h_f : height of a footing (m);

B_f : width of a footing (m).

(a) When using friction enhancement mats under the bottom faces of wall bodies to enhance the stability against sliding during the action of earthquake ground motions, particular attention is required for the inconsistency between a verification equation based on the static equation of equilibrium and deformation mechanisms.[20), 21)] Therefore, the earthquake-resistance performance of wall bodies shall be performed in accordance with **Part III, Chapter 5, 2.2.4 Performance Verification for the Deformation Amounts of Facilities during Earthquakes**. All calculation methods for the seismic coefficients for verification to be used for the verification of failures due to the sliding, overturning of the quaywall, and insufficient bearing capacity of foundation ground under variable state with respect to Level 1 earthquake ground motions have been established for the condition to obtain the widths of wall bodies without the use of a friction enhancement mat.

(b) The following vertical force acting on a wall body is normally considered in examining the sliding of a wall body:

1) The value obtained by subtracting buoyancy from the weight of the wall body excluding surcharges (e.g., load of bulk cargoes) anterior to the virtual boundary plane of the wall body

2) The vertical component of earth pressure acting on virtual boundary plane

(c) The following horizontal force acting on a wall body is normally considered in examining the sliding of a wall body:

1) The horizontal component of the earth pressure acting on the virtual boundary plane of a wall body with a surcharge applied to the surface of backfilling soil.

2) Residual water pressure

3) In addition to the above, for the performance verification during the actions of earthquake ground motions, the inertia force and dynamic water pressure acting on the wall body, the horizontal component of the earth pressure during earthquakes, and the horizontal force of cargo handling equipment acting on the wall body through the legs of the equipment

(d) The coefficient of friction can be set in accordance with **Part II, Chapter 11, 9 Friction Coefficient**.

(e) In cases of quaywalls with horizontal joints, similar to the case with block-type quaywalls, it is preferable that the quaywalls are provided with tenons that enable the respective joints to exert sufficient interlocking effects and have strength to resist the horizontal force applied to them. The structures of tenons can follow **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

(f) Even when rubble foundations or foot protection blocks are installed in front of wall bodies for the purpose of scour prevention or the protection of the foots of slopes, it is advisable that the performance verification with respect to sliding are performed without considering the resistance of the rubble foundations or foot protection blocks to the sliding of the wall bodies.

② **Examination of the stability against overturning**

The examination of the stability of a wall body against overturning can be performed using **equation (2.2.7)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.2.3** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience. The partial factors (permanent state) shown in the table have been set with reference to the safety levels in past standards.[19)]

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$R_k = (aW_k - bP_{B_k} + cP_{V_k}) \tag{2.2.7}$$

$$S_k = dP_{H_k} + eP_{w_k} + hP_{dw_k} + iP_{F_k}$$

where

W : weight of materials comprising a wall body (kN/m);

P_B : buoyancy acting on a wall body (kN/m);

P_V : resultant vertical earth pressure acting on a wall body (kN/m);

P_H : resultant horizontal earth pressure acting on a wall body (kN/m);

P_w : resultant residual water pressure acting on a wall body (kN/m);

P_{dw} : resultant dynamic water pressure acting on a wall body (kN/m) (only during earthquakes);

P_F : inertia force acting on a wall body (kN/m) (only during earthquakes);

a : distance from the action line of resultant weight of wall to the front toe of a wall body (m);

b : distance from the action line of buoyancy to the front toe of a wall body (m);

c : distance from the action line of resultant vertical earth pressure to the front toe of a wall body (m);

d : distance from the action line of resultant horizontal earth pressure to the bottom of a wall body (m);

e : distance from the action line of resultant residual water pressure to the bottom of a wall body (m);

h : distance from the action line of resultant dynamic water pressure to the bottom of a wall body (m) (only during earthquakes);

i : distance from the action line of inertial force to the bottom of a wall body (m) (only during earthquakes);

R : resistance term (kN·m/m);

S : load term (kN·m/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.3 Partial Factors Used for the Performance Verification of the Overturning of Wall Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of a wall body (Permanent state)	0.99	1.23	—— (1.00)
Overturning of a wall body (Variable state with respect to Level 1 earthquake ground motions)	—— (1.00)	—— (1.00)	1.10

The characteristic value of residual water pressure P_{w_k} shall be appropriately calculated by referring to **Part II, Chapter 4, 3.1 Residual Water Pressure**. In cases wherein caissons have footings with a rectangular cross section on both the sea and shore sides, **equation (2.2.6)** can be used for calculating the characteristic value of buoyancy.

③ **Examination of the bearing capacity of foundation ground**

(a) When examining the bearing capacity of shallow foundations, the force acting on the bottom of wall bodies is the resultant force of loads acting in the vertical and horizontal directions; therefore, this force can be examined using **Part II, Chapter 2, 3.2.5 Bearing Capacity for Eccentric and Inclined Actions**. For the standard partial factor used in performance verification, the values shown in **Table 2.2.4** may be used.

(b) The performance verification of the stability of the bottom of a wall body with respect to the bearing capacity of ground may be performed by using **equation (2.2.8)**. The partial factors in the equation can be selected from the values in **Table 2.2.4**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. In **Table 2.2.4**, the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience. When using **equation (2.2.8)**, an auxiliary parameter E_f needs to be determined via repeated calculation so that E_f satisfies $R_k = E_f \times S_k$ (with attention to the fact that R_k is a function of E_f), and the performance verification of bearing capacity can be performed using the R_k and S_k obtained as a result of the repeated calculation.

$$
\begin{aligned}
& m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\
& F_f = \frac{R_k(F_f)}{S_k} \\
& R_k = \sum \left[\frac{\{c'_k\, s + (w'_k + q_k)\tan\phi'_k\}\sec\theta}{1 + \tan\theta \tan\phi'_k / F_f} \right] \\
& S_k = \sum \{(w'_k + q_k)\sin\theta\} + \frac{aP_{H_k}}{r}
\end{aligned}
\qquad \textbf{(2.2.8)}
$$

where

P_H : value of a horizontal action on a soil mass inside a slip failure circle (kN/m);

a : distance from an action position of P_H to the center of a slip failure circle passing through the action position (m);

c' : undrained shear strength for cohesive soil ground or apparent adhesion under drained condition for sandy soil ground (kN/m^2);

s : width of a segment (m);

w' : effective weight of a segment (kN/m) (atmospheric weight when above water surface or underwater weight when below water surface);

q : surcharge acting on a segment (kN/m);

ϕ' : apparent shear resistance angle based on effective stress (°);

θ : angle between the bottom face of a segment and a horizontal plane (°);

F_f : auxiliary parameter representing a ratio of a resistance term to a load term;

r : radius of a slip failure circle (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.4 Partial Factors Used for the Performance Verification of the Failure of Bearing Capacity of Foundation Ground

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Failure of bearing capacity of foundation ground (Permanent state)	—— (1.00)	—— (1.00)	1.20
Failure of bearing capacity of foundation ground (Variable state in respect of Level 1 earthquake ground motions)	—— (1.00)	—— (1.00)	1.00

For the characteristic values and distribution widths of the surcharge loads acting on segments, reference can be made to **Part III, Chapter 2, 3.2.5 Bearing Capacity for Eccentric and Inclined Actions**.

(c) In general, the examination of the bearing capacity of foundation ground is performed with no surcharge applied to a wall body. However, considering that a surcharge causes eccentricity to be decreased but the vertical component force to be increased, the examination may also be performed for the case with a surcharge applied to a wall body as necessary.

(d) The thickness of the foundation mound can be determined by examining the failures due to the insufficient bearing capacity of the foundation ground, the flatness of mound surfaces on which wall bodies are installed, the alleviation of local stress concentration in the ground, etc. The minimum thickness should be determined in accordance with the following guidelines:

1) For a quaywall with a water depth of less than 4.5 m, a thickness of 0.5 m or more; provided, however, that the thickness of the mound shall be at least three times the average diameter of rubbles.

2) For a quaywall with a water depth of 4.5 m or more, a thickness of 1.0 m or more; provided, however, that the thickness of the mound shall be at least three times the average diameter of rubbles.

(e) There have been few cases of gravity-type quaywall structures using foundation piles. In such cases, the performance verification can be performed in accordance with **Part III, Chapter 2, 3.4 Pile Foundations**. In cases of bearing piles driven into the ground susceptible to settlement with the bottom surfaces of wall bodies directly placed on the piles, the structures of wall bodies are destabilized below the bottom surfaces of wall bodies owing to cavities, thus causing the outflow of backfill materials. In such cases, gravity-type quaywalls need to have structures with pile heads covered by rubble mounds.

④ **Examination of settlement**

(a) Gravity-type quaywalls shall ensure their structural stability against settlement due to the consolidation of the ground in accordance with the characteristics of the ground and structures.

(b) For the foundation ground susceptible to settlement, it is important to implement sufficient soil investigation and preliminarily estimate settlement amounts in accordance with **Part II, Chapter 5, 1 Ground Settlement**. It is preferable to take measures for setting higher foundation surfaces or enabling superstructures to be used for final adjustment to achieve predetermined crown heights on the basis of the estimated settlement amounts. It is also necessary to pay attention to the possibility that uneven settlements may cause joint failures and discontinuity or superstructure and apron pavement failures.

(4) Performance Verification for Accidental Situation with respect to Level 2 Earthquake Ground Motions

The performance verification of the seismic resistance of gravity-type quaywalls for Level 2 earthquake ground motions shall be performed by specifically calculating the deformation amounts of facilities via appropriate earthquake response analyses or experiments with reference to **Part III, Chapter 5, 2.2.4 Performance Verification for the Deformation Amounts of Facilities during Earthquakes**. The standard limit values of deformation amounts under accidental situation with respect to Level 2 earthquake ground motions can be appropriately set by referring to Chapter 5, 1.5 Points of Cautions for High Earthquake-resistance Facilities.

2.2.4 Performance Verification for the Deformation Amounts of Facilities during Earthquakes

(1) Performance Verification Methods for the Deformation Amounts of Facilities during Earthquake

Performance verification methods for the deformation amounts of facilities can be broadly classified into two types: methods employing a seismic response analysis and shaking tests using a shaking table or similar apparatus. When performing the performance verification of deformation amounts via seismic response analysis, the appropriateness of the analysis methods should be confirmed by the simulation analyses of damaged cases. For example, FLIP[22)] is one of the methods that have been confirmed applicable to port facilities, such as gravity-type quaywalls. Other methods can also be used properly provided that their applicability has been confirmed by damage simulation analyses and the like.

In cases wherein there have been no damaged cases to confirm the applicability of existing analysis methods to structural types that have been newly adopted, it is necessary to confirm the applicability of the methods through the simulation analyses of appropriate shaking test results.

The general performance verification methods are explained below with the points of caution for their usage. For the details of earthquake response analyses, refer to **Reference (Part III), Chapter 1, 2 Basic Items for Earthquake Response Analyses**.

① **Methods employing seismic response analysis**

Seismic response analyses can be classified as shown in **Table 2.2.5**. In the following, the various types of seismic response analysis methods are explained in accordance with these classifications. Depending on the seismic response analysis methods, these methods may not be suitable in some cases for the purpose of verifying deformation. Therefore, it is necessary to select an analysis method that corresponds to the intended purpose on the basis of the following explanations.

Table 2.2.5 Classification of Seismic Response Analyses

Analysis method (Applicable types of saturated ground)	Effective stress analysis method, total stress analysis method (Combination of solid and liquid layers or solid layer)
Object domain of calculation (Dimensions)	1D, 2D, 3D
General calculation models	Multiple reflection model, point mass model, finite element model
Material characteristics	Linear, equivalent linear, nonlinear
Calculation domain	Time domain analysis method and frequency domain analysis method

(a) Effective stress analysis method and total stress analysis method

From the viewpoint of the prediction and determination of liquefaction and the prediction of soil deformation behavior, seismic response analyses can be classified into analyses based on the effective stress and those based on total stress. In most cases, it is necessary to consider the reduction in effective stress due to the generation of excessive pour pressure in the ground (the extreme reduction in effective stress leads to liquefaction) when predicting the deformation of port facilities during the action of earthquake ground motions. This should be considered because the deformation and response characteristics of the ground are changed as a result of the changes in the stress–strain relation and the attenuation characteristics of soil associated with change in the stress state of the soil such as the reduction in effective stress. The effective stress analysis method can directly calculate the excessive pour pressure generated in the ground. By contrast, the total stress analysis method does not calculate the change in the excessive pour pressure. Therefore, in cases wherein excessive pour pressure exceeds a certain level (approximately 0.5 or more in terms of the excessive pour pressure ratio depending on conditions), the calculation results of the total stress analysis may have large differences from the actual earthquake response of the ground.

There are many cases of using the total stress analysis in practical business because of its simplicity; however, the use of the effective stress analysis is the basic requirement in the performance verification of the deformation of port facilities that are at risk of liquefaction.

(b) Classification based on calculation domain (dimensions)

The seismic response analyses can be classified into 1D, 2D, and 3D methods depending on the calculation of object domains. The 1D method is generally applied to the seismic response analyses of ground with geological stratum structures comprising widely and horizontally accumulated planar strata. However, analysis methods with higher dimensions are required because the facilities used as objects of deformation verification have 3D structures. The 2D method is normally used for the deformation verification of facilities such as quaywalls in the structure–ground system with uniform characteristics in a depth direction. Although the 3D method is required when dealing with regions that include structures such as piles, special elements such as pile–ground cross-interaction springs are normally used to enable such regions to be analyzed by the 2D method. The 3D method is not generally used in practical business because of the necessity to deal with complicated models that require long calculation times; however, this method has been used for the deformation verification of important facilities and for experimental purposes.

(c) Types of general calculation models

1) Multiple reflection model

This calculation model considers that the ground comprises a series of horizontally accumulated soil layers and that the shear waves vertically entering the soil layers from the ground upwardly propagate while repeating transmission and reflection at the boundaries between soil layers. This model is applicable to the earthquake response analyses using linear or equivalent linear methods and is generally inapplicable to the deformation verification of structures.

2) Lumped mass model

This model deals with the ground and structures as a combination of one or more mass, springs, and attenuation mechanisms. This model can be analyzed by relatively simple calculation programs, thus enabling nonlinear deformation–restorative force relation to be incorporated into springs. Although this model enables deformation amounts to be obtained via simple calculation, it is not accurate enough to be a model that can be generally used for detailed deformation verification. This model has been frequently used in dynamic analyses to calculate stress acting on buildings and buried structures (pipes and piles).

3) Finite element model

This model divides ground and structures into a finite number of elements (**Fig. 2.2.8**) and has been used in a wide range of fields. One of the characteristics of this model is its ability to simply represent 2D changes in the layer thicknesses and physical property of the ground. Finite element analysis programs already in practical use include FLUSH[23)] and FLIP[22)] and others. It is necessary to pay attention to the fact that programs like FLUSH, which employ the equivalent linear method, are not suitable for the prediction of residual deformation. By contrast, the applicability of FLIP to the deformation verification of many port facilities has been confirmed by the analyses of damage to port facilities due to the South Hyogo Prefecture Earthquake.[24)] There has been a report that FLAC,[25)] which is one of the finite difference analysis programs based on the explicit method, can also be used for the deformation analyses in the same way as the finite element methods subject to the conformity with constitutive laws.

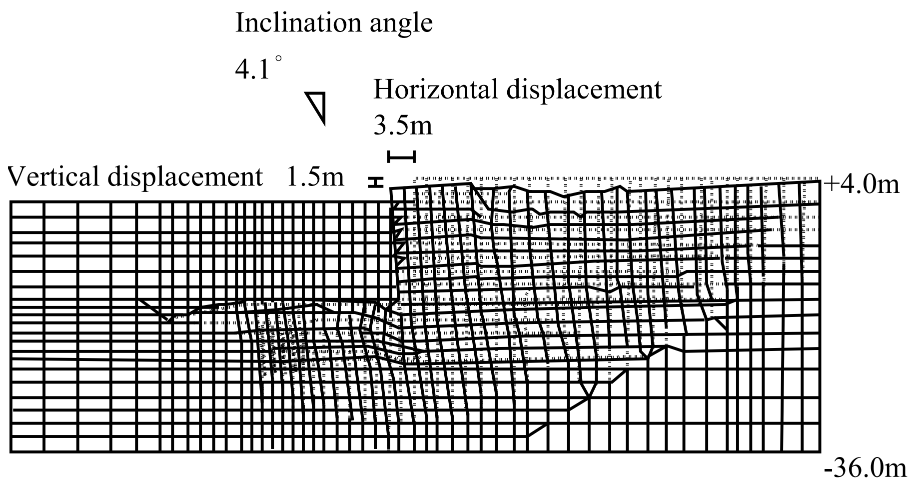


Fig. 2.2.8 Finite Element Model (Gravity-type Quaywall)

4) Individual element model

In the individual element modeling method, soil and respective structures such as wave dissipating blocks are individually modeled as granular objects, and deformation analyses are performed by calculating mutual interaction as a result of the contact among granular objects.[26] This modeling method is particularly suitable for analyzing the deformation associated with the rotation of wave dissipating blocks. There has been a proposal for a hybrid analysis method that combines the individual element method and the finite element method.[27]

(d) Modeling of material characteristics

The modeling of the nonlinear characteristics of soil material constituting the ground is important in the execution of earthquake response analyses. A mathematical model representing behavioral characteristics, such as the stress–strain relation of soil, is called a constitutive law. When shear strain during the action of earthquake ground motions is in a relatively low level, soil shows a linear stress–strain relation; however, when the shear strain is in an intermediate or high level, soil shows significantly nonlinear stress–strain relation. Therefore, depending on the levels of shear strain, it is necessary to use a constitutive law that is capable of dealing with nonlinearity in deformation verification.

Several earthquake response analysis methods have been proposed: a linear analysis method that does not consider the nonlinearity of the materials constituting the ground, an equivalent linear analysis method that performs linear analyses by using material constants depending on the strain levels that the ground receives, and a nonlinear response analysis method that considers the stress–strain relation of the soil subjected to large strain. However, in consideration of the purpose of the deformation verification to examine residual deformation, there are cases wherein linear and equivalent linear analysis methods are not always appropriate and wherein nonlinear response analysis methods based on constitutive laws and capable of dealing with nonlinearity are required.

(e) Classification by calculation domain

From the viewpoint of the calculation domain, the earthquake response analyses can be classified into time domain analysis method and frequency domain analysis method. The effective stress analysis method and calculations with nonlinear material characteristics are generally performed sequentially in the time domain.

② **Methods employing shaking tests**

The methods employing shaking tests, which apply vibrations to structures including the ground by taking into consideration mechanical similarities, are effective in assessing the overall behavior of the structures. However, these methods require high levels of experimental techniques for preparing models that adequately satisfy the condition of similarities. The shaking tests using a shaking table are classified as follows.

(a) Model shaking test in a 1G gravity field

In the model shaking test in a 1G gravity field, models are prepared in a manner that satisfies the similarity ratios by taking into consideration the shapes and mechanical characteristics of the target structures and ground. Assumed earthquake ground motions are applied to the models using a shaking table. The model shaking test generally enables large models to be prepared and is applicable to cases with complex ground and structural configurations. Furthermore, the similarity law considering the dependency of the physical property of soil on confining pressure is normally applied to the model shaking test.[28)]

(b) Model shaking test using a centrifugal loading device

In the test, the assumed earthquake ground motions generated by a shaking test device is applied to models that satisfy the similarity law, and stress states similar to actual situations are reproduced in them by the centrifugal force generated by a centrifugal loading device. The test generally requires models to be small in scale but enables the models to be tested on the basis of the dependency of the physical property of soil on confining pressure without assuming the relation between the physical property of soil and effective confining pressure. However, the test requires attention to the use of the coefficient of permeability conforming to the similarity law and the influences of the particle sizes of ground materials used in the test on test results.

(c) In-situ shaking test

In this type of test, models that are similar to or in substantially the same scale as target structures are prepared either at the location where construction is planned or under similar ground conditions. The responses of the models to artificial ground motions or natural ground motions are then observed. The methods of generating artificial ground motions include methods employing wave vibrators and blasting.[29)]

Although model and in-situ shaking tests are effective tests, they cannot accurately reproduce actual boundary conditions. For example, in the model shaking tests, models are subjected to the input of earthquake motions in rigid ground without the attenuation effect of their downward scattering; therefore, the test results are likely to be strongly influenced by the natural frequency of the ground–structure systems. Furthermore, in-situ shaking tests using blasting cannot reflect the effect of inertia force due to earthquake ground motions in the test results.

2.2.5 Performance Verification of Cellular Blocks

(1) Unlike other gravity-type quaywalls, gravity-type quaywalls that comprise cellular blocks with no bottom slabs have structures that maintain integrity with wall bodies through fillings. Therefore, in addition to the examination of stability similar to the case in other gravity-type quaywalls, overturning should be examined, with due consideration given to the extrusion of the fillings.

(2) Equation for Verifying the Stability of Cellular Blocks against Overturning

The verification of the stability of cellular blocks against overturning in considering of the extrusion of fillings can be performed using **equation (2.2.9)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.2.6** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$R_k = (aW_k - bP_{B_k} + cP_{V_k} + M_{fk})$$
$$S_k = dP_{H_k} + eP_{w_k} + hP_{dw_k} + iP_{F_k} \qquad \textbf{(2.2.9)}$$

where

W : weight of materials comprising a wall body (kN/m);

P_B : buoyancy acting on a wall body (kN/m);

P_V : resultant vertical earth pressure acting on a wall body (kN/m);

M : resistant moment due to friction on wall surfaces with fillings (kN·m/m);

P_H : resultant horizontal earth pressure acting on a wall body (kN/m);

P_w : resultant residual water pressure acting on a wall body (kN/m);

P_{dw} : resultant dynamic water pressure acting on a wall body (kN/m) (only during earthquakes);

P_F : inertia force acting on a wall body (kN/m) (only during earthquakes);

a : distance from the action line of resultant weight of wall to the front toe of a wall body (m);

b : distance from the action line of buoyancy to the front toe of a wall body (m);

c : distance from the action line of resultant vertical earth pressure to the front toe of a wall body (m);

d : distance from the action line of resultant horizontal earth pressure to the bottom of a wall body (m);

e : distance from the action line of resultant residual water pressure to the bottom of a wall body (m);

h : distance from the action line of resultant dynamic water pressure to the bottom of a wall body (m) (only during earthquakes);

i : distance from the action line of inertial force to the bottom of a wall body (m) (only during earthquakes);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.6 Partial Factors Used for the Performance Verification of the Overturning of Wall Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of the cellular block with the extrusion of fillings (Permanent state)	– (1.00)	– (1.00)	1.20
Overturning of the cellular block with the extrusion of fillings (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.10

Furthermore, when a caisson has footings with rectangular cross sections on both the sea and land sides, the characteristic value of buoyancy can be calculated using **equation (2.2.6)**.

(3) When $S_d > R_d$ in **equation (2.2.9)**, the overturning moment due to the action becomes larger than the resistant moment generated by the total vertical force, excluding fillings and the friction on wall surfaces with fillings, thus causing a cellular block to overturn with fillings left behind. In such a case, it is necessary to increase the weight of the cellular block or to provide the cellular block with partition walls.

(4) The characteristic value M_{fk} of the resistant moment generated by the friction force F_1 and F_2 on wall surfaces with fillings can be calculated as follows. The moment around Point A in **Fig. 2.2.9** can be expressed by $l_1F_1 + l_2F_2$ where $F_1 = P_1f$ and $F_2 = P_2f$, and f is a coefficient of friction between a filling material and a wall surface (P_1 and P_2 are the earth pressure of the fillings). For the concept of the earth pressure of fillings acting on wall surfaces, refer to **Part III, Chapter 2, 2.4 Cellular Blocks**. It is desirable to consider the friction resistance generated on the partition walls of a cellular block, in addition to the resistant moment.

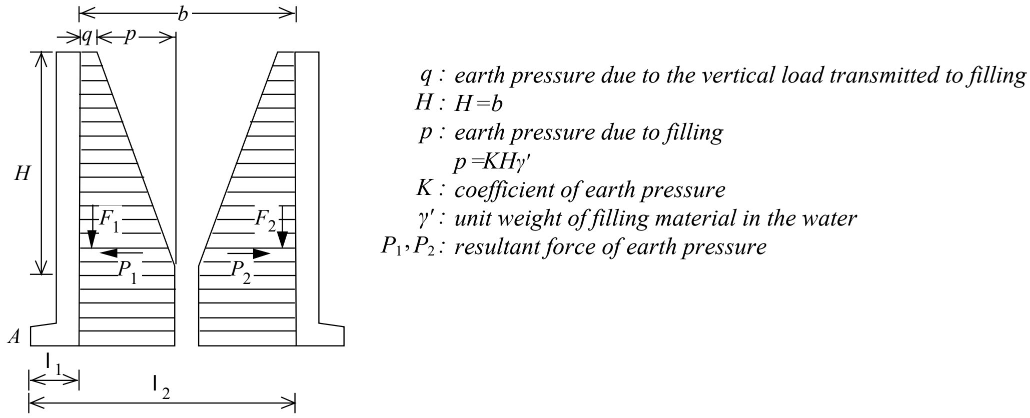


Fig. 2.2.9 Method for Obtaining Wall Surface Friction Resistance

(5) Regarding the characteristic values of the friction coefficient to be used for the performance verification of the sliding of cellular concrete blocks with no bottom slabs, 0.6 and 0.8 shall be used as the reaction force received by the bottoms of reinforced concrete sections and filling stone sections, respectively. However, for convenience, 0.7 can be used for both characteristic values.

2.2.6 Performance Verification of Structural Members

(1) For the performance verification of structural members such as caissons, cellular blocks, L-shaped blocks, refer to **Part III, Chapter 2, 2 Structural Members**. For block-type quaywalls, the blocks that constitute quaywalls should have sufficient strength because they are main sections of wall bodies. For the performance verification of blocks, refer to **Part III, Chapter 2, 2 Structural members**.

(2) The stability of the portion of a superstructure where a mooring post is installed should be examined in a manner that allows the weight of a certain concrete mass of the superstructure to resist mooring force together with the mooring post. In cases wherein the weight of a large concrete mass of a superstructure is required to ensure the stability of a mooring post, the mass shall be reinforced with rebars. In other cases wherein the stability of a mooring post cannot be achieved only by the weight of a superstructure and requires the superstructure to be connected to the main body of a quaywall through rebars, it is necessary that the action allowing the mooring force to be transferred from the superstructure to the main body via the rebars shall be considered in examining the stability of the mooring post.

(3) The performance verification of the portion of a superstructure where a fender is installed can be performed in a manner that focuses only on a certain concrete mass whose weight integrally contributes to resisting fender reaction. In cases wherein a fender is installed at the portion of a superstructure that is connected to a wall body via rebars as reinforcement to support a mooring post, the displacement of the superstructure in a direction that allows passive earth pressure to effectively resist fender reaction cannot be expected; therefore, it is desirable that fender reaction is completely borne by the rebars. In the performance verification of the cross section of a superstructure, fender reaction is assumed to be distributed as a linear load in the range of width b (**Fig. 2.2.10 (a)**) and may be considered to act as shown in **Fig. 2.2.10 (b)**. In many cases, the performance verification of the cross section of the superstructure in the vertical direction is performed by assuming a cantilever beam with the bottom edge of the superstructure as a fulcrum and that in the horizontal direction is performed by assuming either a continuous beam or a simple beam with rigid points in the wall body as fulcrums.

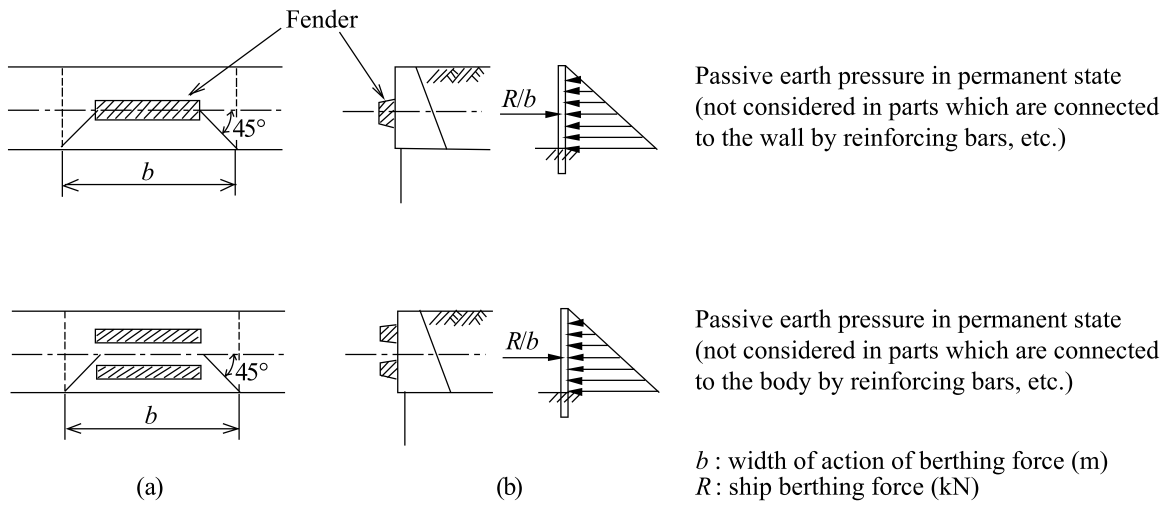


Fig. 2.2.10 Fender Reaction Acting on Superstructures

2.2.7 Structural Details

(1) In cases of gravity-type quaywalls provided with high-quality backfill, the following effects can be expected, in addition to the earth pressure reduction effect:

① Lowering of residual water levels as a result of the increase in permeability

② Protection against backfill soil from being washed out

(2) The fluctuation of residual water levels may cause backfill soil to infiltrate the gaps among backfill materials and cause the settlement of the base courses of apron pavement. Therefore, it is necessary that quaywalls are provided with measures to fill the gaps at the rear of backfilling or sand prevention sheets.

(3) In areas wherein large tidal ranges in front of quaywalls cause severe problems with residual water levels, the shapes of backfill shall be carefully studied so that the residual water levels can be effectively reduced.

(4) In cases wherein land settlement or specific site conditions may cause backfill materials to be washed out, protective measures such as sand washing-out prevention joint plates should be installed on the gaps between the rear faces of wall bodies.

(5) In many cases, the thickness of the cover concrete of caisson-type quaywalls is 20 to 30 cm; however, there may be a case that requires special measures against wave actions, similar to the case with breakwaters depending on construction conditions.

(6) Backfill soil can seep through the gaps between the blocks or rubble stones of gravity-type structures. Therefore, such structures should be provided with appropriate measures, such as sand invasion prevention sheets or plates, to prevent soil from being washed out.

(7) Blocks should be provided with tenons or rebars between them to enhance their integrity as wall bodies via increased interlocking effect. For the methods for interlocking blocks, reference can be made to **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

(8) The portions of the superstructures of quaywalls where ancillary facilities are installed should have appropriate shapes that are suitable for the facilities.

(9) The ancillary facilities generally installed on quaywalls are as follows. For the performance verification of the ancillary facilities, refer to **Part III, Chapter 5, 9 Ancillary Facilities of Mooring Facilities**.

① Fenders
② Mooring posts
③ Curbing
④ Water supply and drainage facilities
⑤ Stairs and ladders
⑥ Others

It is preferable that the joint intervals and the strength of superstructures be examined by taking into consideration the force applied to mooring posts and fenders.

2.3 Sheet Pile Quaywalls

[Public Notice] (Performance Criteria for Sheet Pile Quaywalls)

Article 50

1 The performance criteria for sheet pile quaywalls shall be as prescribed respectively in the following items:

(1) Sheet piles shall have the embedment length necessary for the structural stability and shall contain the degree of risk indicating that the stresses in the sheet piles may exceed the yield stress at the level equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(2) The following criteria shall be satisfied under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating actions are Level 1 earthquake ground motion and traction by ships:

(a) For anchored structures, the anchorage shall be located appropriately in consideration of the structural type, and the risk of losing the structural stability shall be equal to or less than the threshold level.

(b) For structures with ties and waling, the risk that the stresses in the ties and waling may exceed the yield stress shall be equal to or less than the threshold level.

(c) For structures with superstructures, the risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(3) For structures with superstructures, the risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level under the variable situation, in which the dominating action is ship berthing.

(4) Under the permanent situation, in which the dominating action is self-weight, the risk of occurrence of slip failure in the ground below the bottom end of the sheet pile shall be equal to or less than the threshold level.

2 In addition to the provisions in the preceding paragraph, the performance criteria for cantilevered sheet piles shall indicate that the risk in which the amount of deformation of the top of the pile may exceed the allowable limit of deformation is equal to or less than the threshold level under the permanent situation, in which the dominant action is earth pressure, and under the variable situation, in which the dominant actions are Level 1 earthquake ground motion, ship berthing, and traction by ships.

3 In addition to the provisions in the paragraph (1), the performance criteria for double sheet pile structures shall be as prescribed respectively in the following items:

(1) The risk of occurrence of sliding of the structural body shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(2) The risk that the deformation of the top of the front or rear sheet pile may exceed the allowable limit of deformation shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(3) The risk of losing the stability due to the shear deformation of the structural body shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure.

[Interpretation]

11. Mooring Facilities

(4) Performance Criteria of Sheet Pile Quaywalls

① **Sheet pile quaywalls** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance on Criteria and the interpretation related to Article 50, Paragraph 1of the Public Notice)

(a) The required performance of sheet pile quaywalls under a permanent state in which the dominant action is earth pressure and a variable state in which the dominant actions are Level 1 earthquake ground motions shall be serviceability. The performance verification items and standard indexes for determining the limit values with respect to the actions shall be shown in **Attached Table 11-6** provided that those having structures comprising anchorages, those having structures comprising ties and waling, and those having copings shall comply with the provisions in (b), (c), and (d), below respectively.

Attached Table 11-6 Performance Verification Items and Standard Indexes for Determining Limit Values under the Respective Design Situations of Sheet Pile Quaywalls

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index for determining the limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="4">26</td><td rowspan="4">1</td><td rowspan="4">2</td><td rowspan="4">50</td><td rowspan="4">–</td><td rowspan="4">1</td><td rowspan="4">Serviceability</td><td rowspan="2">Permanent</td><td rowspan="2">Earth pressure</td><td rowspan="2">Water pressure, surcharges</td><td>Necessary embedded length</td><td>Embedded length required for structural stability</td></tr>
<tr><td>Yielding of the sheet pile</td><td>Design yield stress of sheet pile</td></tr>
<tr><td rowspan="2">Variable</td><td rowspan="2">L1 earthquake ground motion</td><td rowspan="2">Earth pressure, water pressure, surcharges</td><td>Necessary embedded length</td><td>Embedded length required for structural stability</td></tr>
<tr><td>Yielding of the sheet pile</td><td>Design yield stress of sheet pile</td></tr>
</table>

(b) For the permanent state in which the dominant action is earth pressure and the variable state in which the dominant actions are Level 1 earthquake ground motions and traction by ships, the performance verification items and standard indexes for determining the limit values with respect to anchorages shall be those shown in **Attached Table 11-7**.

Attached Table 11-7 Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to Anchorages under the Respective Design Situations of Sheet Pile Quaywalls

Ministerial Ordinance			Public Notice			Performance requirements	Design sate			Verification item	Standard index for determining the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	50	-	2a	Serviceability	Permanent	Earth pressure	Water pressure, surcharges	Necessary embedded length	Embedded length required for structural stability
										Yielding of the anchorage*1)	Design yield stress
										Axial force on the anchorage*2)	Action–resistance ratio with respect to the bearing force of an anchorage (press force and pullout force)
										Stability of the anchor wall*3)	Design cross-section resistance Passive earth pressure on the front face of anchor plate
							Variable	L1 earthquake ground motion [traction force of ships]	Earth pressure, water pressure, surcharges	Necessary embedded length	Embedded length required for structural stability
										Yielding of the anchorage*1)	Design yield stress
										Axial forces in the anchorage*2)	Action–resistance ratio with respect to the bearing force of an anchorage (press force and pullout force)
										Stability of the anchor wall*3)	Design cross-section resistance Passive earth pressure on the front face of the anchor plate

* [ ] indicates an alternative dominant action to be studied as design situations.
*1): Only when the structural type of the anchorage is a vertical pile anchor, a coupled-pile anchor, or sheet pile anchor
*2): Only when the structural type of the anchorage is a coupled-pile anchor
*3): Only when the structural type of the anchorage is a slab anchor

(c) For the permanent state in which the dominant action is earth pressure and the variable state in which the dominant actions are Level 1 earthquake ground motions and traction by ships, the performance verification items and the standard indexes for determining the limit values with respect to ties and waling shall be those shown in **Attached Table 11-8**.

Attached Table 11-8 Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to Ties and Waling under the Respective Design Situations of Sheet Pile Quaywalls

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for determining the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	50	–	2b	Serviceability	Permanent	Earth pressure	Water pressure, surcharges	Yielding of the tie	Design yield stress
										Yielding of the waling	
							Variable	L1 earthquake ground motion [traction force of ships]	Earth pressure, water pressure, surcharges	Yielding of the tie	
										Yielding of the waling	

* [ ] indicates an alternative dominant action to be studied as design situations.

(d) For the permanent state in which the dominant action is earth pressure and the variable state in which the dominant actions are Level 1 earthquake ground motions and traction by ships, the performance verification items and standard indexes for determining the limit values with respect to the copings of sheet pile quaywalls shall be those shown in **Attached Table 11-9**.

Attached Table 11-9 Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to the Copings of Sheet Pile Quaywalls under their Respective Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for determining the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	50	–	2c	Serviceability	Permanent	Earth pressure	Surcharges	Cross-section stress of the coping	Bending compression stress
							Variable	L1 earthquake ground motion [traction force of ships] [berthing force of ships]	Earth pressure, surcharges	Failure of the cross section of the coping	Design cross-section resistance

* [ ] indicates an alternative dominant action to be studied as design situations.

(e) For the permanent situation in which the dominant action is the self-weight of sheet pile quaywalls, the performance verification items and standard indexes to determine the limit values of sheet pile quaywalls shall be those shown in **Attached Table 11-10**.

Attached Table 11-10 Performance Verification Items and Standard Indexes to Determine the Limit Values under the Permanent Situation in which the Dominant Action Is the Self-weight of Sheet Pile Quaywalls

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	50	–	4	Serviceability	Permanent	Self-weight	Water pressure, surcharge	Circular slip failure of the ground	Action–resistance ratio with respect to circular slip

(f) In the cases of using sheet piles with special joints or large-scale joints, the performance verification items and standard indexes to determine the limit values with respect to the stress on the joints shall be appropriately set as needed.

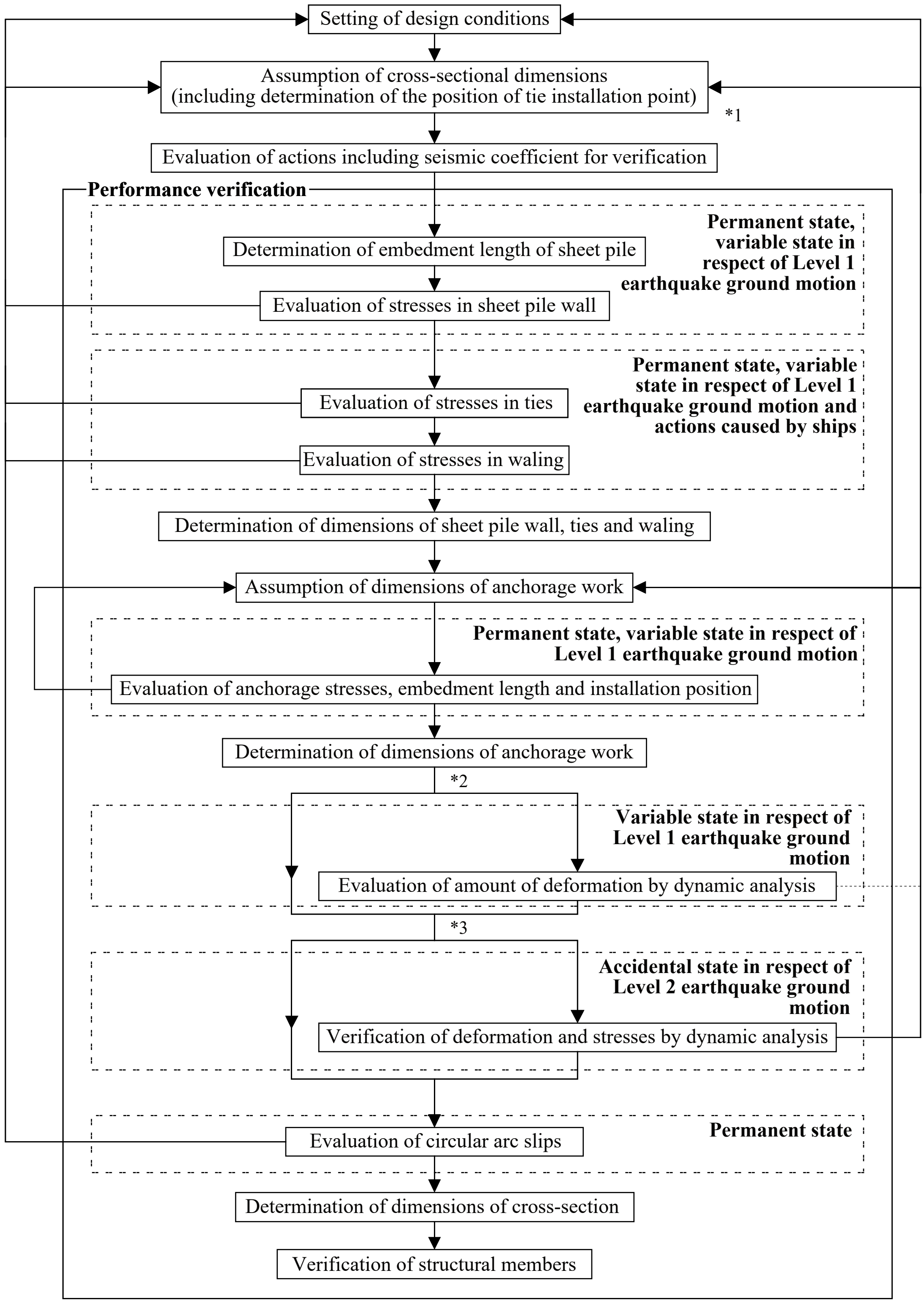
(g) In addition to the provisions in this code, sheet pile quaywalls shall comply with the provisions and commentaries in Paragraph 3 (Scouring and Wash Out), Article 22 and Article 28 (Performance Criteria of Armor Stones and Blocks) of the Standard Public Notice as needed.

2.3.1 General

(1) The provisions in this section can be applied to the performance verification of steel sheet pile quaywalls with anchorages.

(2) **Fig. 2.3.1** shows an example of the sequence of the performance verification of sheet pile quaywalls. However, **Fig. 2.3.1** does not show the evaluation of the effects of the liquefaction and settlement due to earthquakes. Therefore, it is necessary to appropriately deliberate the possibility of and countermeasures against liquefaction with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the variable state with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method on the basis of a static equation of equilibrium. However, for high earthquake-resistance facilities, it is advisable to deliberate deformation by using nonlinear seismic response analysis or other methods that take into consideration the dynamic interaction between the ground and structures. For sheet pile quaywalls other than high earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.

(3) **Fig. 2.3.2** shows an example of the cross section of the sheet pile quaywalls.



*1: The evaluation of liquefaction and settlement are not shown; therefore, it is necessary to consider these separately.
*2: When necessary, an evaluation of the amount of deformation by dynamic analysis can be performed for the Level 1 earthquake ground motion.
For high earthquake-resistance facilities, it is preferable that an examination of the amount of deformation be performed by dynamic analysis.
*3: Verification with respect to Level 2 earthquake ground motion is performed for high earthquake-resistance facilities.

Fig. 2.3.1 Example of the Sequence of the Performance Verification of Sheet Pile Quaywalls

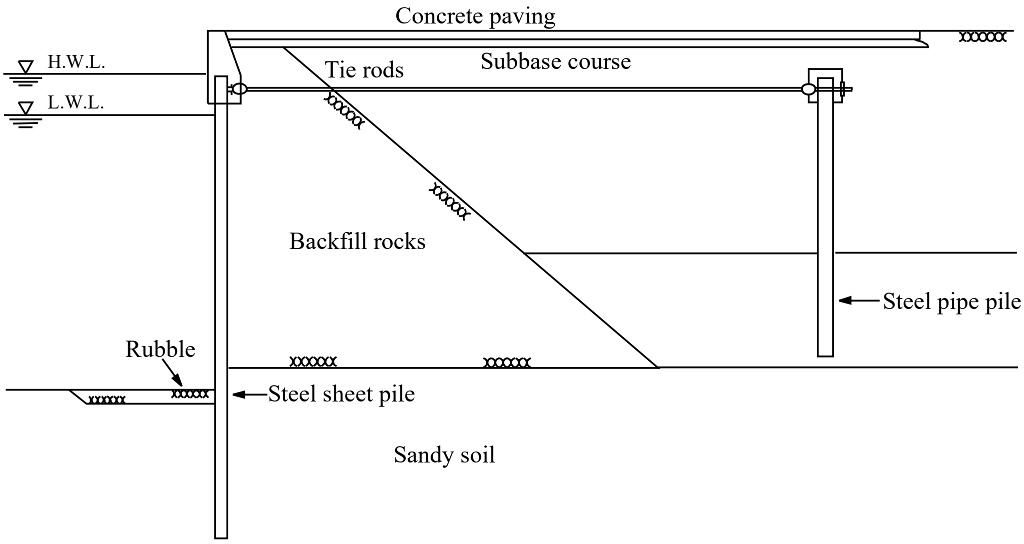


Fig. 2.3.2 Example of the Cross Section of the Sheet Pile Quaywall

2.3.2 Points of Caution When Installing Sheet Pile Quaywalls on Soft Ground

(1) The performance verification of a sheet pile wall on soft ground, such as alluvial cohesive soil on soft seabed, should preferably be conducted via comprehensive examination by using the performance verification methods shown in this section for ties and anchorages and other performance verification methods. An unexpected large deformation may occur in sheet piles constructed on soft ground owing to lateral flows that are caused by the settlement of the ground behind the sheet pile wall. Several methods for lateral flow prediction[30)] have been proposed. These effects should be taken into consideration when conducting the performance verifications.

(2) Care should be exercised in using the performance verification methods for the sheet pile quaywalls described in this section because many of these methods assume that a steel sheet pile wall is driven mainly into sandy soil ground or hard clayey soil ground. For soft ground, it is preferable to perform soil improvement work. When it is not possible to perform soil improvement work because of site conditions, it is preferable to consider using other performance verification methods, in addition to the methods described in this section, such as numerical analysis methods that can accurately evaluate the nonlinear characteristics of soil, so that a comprehensive analysis can be made.

(3) When deliberating the embedded lengths of sheet piles, the deflection curve method,[31)] which is a type of fixed earth support method based on the classical earth pressure theory that deals with the sheet piles with deep embedded lengths, can be used in addition to the method introduced in this section. The deflection curve method obtains an embedded length by solving an equation under the following conditions: the displacement and deflection angle at the lower end of embedment are zero; the displacement at a tie member installation point is zero and is under the loading conditions shown in **Fig. 2.3.3**. The deflection curve method is also applicable to soft ground.

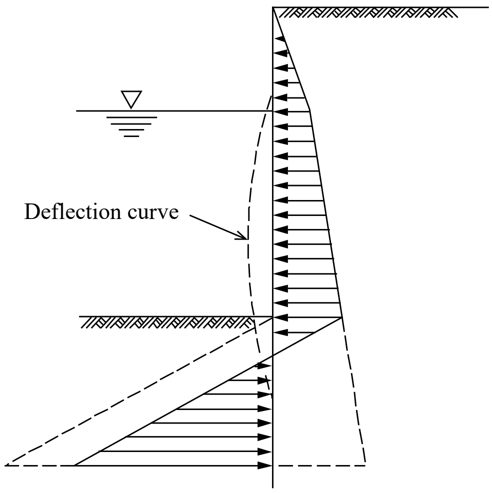


Fig. 2.3.3 Earth Pressure and Deflection Curve

(4) It is advisable to comprehensively deliberate the bending moment in sheet piles and the tensile force in tie members by using the method for bending moment and tie member installation point reaction force, which is explained in this section, and the deflection curve method mentioned above.

(5) Generally, for cohesive soil ground, the stability of embedment cannot be achieved unless **equation (2.3.1)** is satisfied. In this equation, subscript k refers to a characteristic value.

$$4c_k > q_k + \sum w_{i_k} + \rho_w g h_w \tag{2.3.1}$$

where

c : adhesion of sea-bottom soil (kN/m^2);

q : loaded weight (kN/m^2);

w_i : weight of the ith soil layer above the seafloor surface or underwater weight of a soil layer if it is below a residual water level (kN/m^2);

ρ_w : density of seawater (t/m^3);

g : gravitational acceleration (m/s^2);

h_w : difference between a residual water level and a tidal level in front of a quaywall (m).

When soft seabed does not satisfy **equation (2.3.1)**, the seabed needs to be improved by an appropriate method or a sheet pile wall with a relieving platform that needs to be used as a countermeasure.

2.3.3 Setting of Cross-Sectional Dimensions

(1) Installation Positions of Tie Members

① Tie member is a collective term of materials such as tie rods and tie wires connecting sheet piles and anchorages.

② The cross sections of sheet piles and tie members will be largely influenced by the positions of the tie member installation. The positions of tie member installation should be determined by considering the difficulty of the work of tie member attachments and the costs.

③ The bending moment in sheet piles has a tendency to be reduced as the positions for the installation of tie members become lower. Generally, the bending moment is reduced by installing tie members at positions that are approximately half the heights of sheet pile walls. Thus, the cross-sectional areas of sheet piles can be reduced by lowering the tie member installation positions, and the embedded lengths of sheet pile can be reduced accordingly. By contrast, the tensile force acting on tie members has a tendency to increase as the positions to install tie members become lower, thereby increasing the cross-sectional areas of tie members and the sizes of anchorages. Therefore, it is advisable to decide the tie member installation positions to minimize construction costs by balancing the cost reduction and increase the effects above. Generally, construction costs are reduced as the tie member installation positions become lower. However, if the original ground levels before starting construction are high, decreasing the tie member installation positions may increase the construction costs because of increased excavation and backfilling costs.

④ When the wall height of a sheet pile wall is large, tie members may be provided at two levels to support the wall structure at two points to reduce the bending moments in the sheet pile.

⑤ The tie member installation position is generally set at approximately 2/3 of a tidal difference above the lowest water level (LWL).

(2) Selection of the Structural Types of Anchorages

① The structural types of anchorages are generally broadly classified as vertical pile anchorage, coupled-pile anchorage, sheet pile anchorage, and slab anchorage. The economy, construction time, and construction method differ depending on the structural types; therefore, it is necessary to determine the structural types by considering the elevation of the ground before construction and other site conditions.

② In the case where the ground in front of anchorages is saturated sandy soil and subjected to liquefaction owing to earthquake ground motions, anchorages with shallow embedment are likely to be affected by liquefaction because liquefaction occurs close to ground surfaces. Therefore, coupled-pile anchorages with deeper embedment are preferable in areas with such ground conditions. Refer to **Part II, Chapter 7 Ground Liquefaction** when studying liquefaction due to earthquake ground motions.

③ Generally, the displacement of anchorages when tie members are subjected to tensile force is smaller in the case of coupled-pile anchorage and larger in the case of sheet pile anchorage or vertical pile anchorage. Furthermore, the displacement due to earthquake ground motions is particularly increased in the case of anchorages using sheet piles and vertical piles.

④ Vertical pile anchorages or coupled-pile anchorages are generally preferable in the case where original seafloor surfaces are deep before starting construction.

⑤ Generally, a coupled-pile anchorage structure is preferable in the case where facilities at the back of quaywalls impose the constraints on the installation locations of anchorages.

⑥ It is necessary to pay attention to possible bending stress in structures, in addition to axial force in the case of coupled-pile anchorages installed in areas subjected to the settlement of backfill soil.

⑦ Whether concrete work can be executed in a dry condition is a criterion for determining the availability of the slab anchorages of sheet pile walls. In most cases, the slab anchorage for relatively large-scale sheet pile walls requires construction below groundwater level involving the temporary closing and drainage of water with pumps. For small-scale concrete walls, concrete structures that were prefabricated at factories can be transported and installed by cranes at sites similar to the case with concrete walls of dead man anchors.

⑧ The construction of sheet pile anchorages is easy and can be executed in a short period because the construction method is identical to that for sheet pile walls. The sheet pile anchorages are particularly preferable in the case where the ground levels at the back of quaywalls are high enough to enable steel sheet piles to be driven onshore.

(3) Installation Locations of Anchorages

① In principle, the location of the anchorages need to be set at an appropriate distance from the sheet pile wall to ensure the structural stability of the main body of the wall and anchorage depending on the characteristics of the anchorages. Normally, when the position of the installation of the anchorage is further from the surface of the sheet pile wall, the deformation restraint of the sheet pile wall during an earthquake will be more effective.[32)] By contrast, the cross-sectional force on the sheet pile walls increases as the level of constraint on deflection increases. The following method for setting the locations of anchorages has been used in many cases, but the application of the method shall be comprehensively determined after considering the relations explained above.

② The location of an anchorage should be determined appropriately in consideration of the structural type of the anchorage because the stability of the anchorage itself is affected by its position, and the location at which the stability is achieved varies depending on the structural type. Furthermore, the location of anchorages on soft ground shall be determined after the comprehensive deliberation of the behavior of sheet piles, tie members, and anchorages on the occurrence of earthquakes by using the method explained in this section or by using a dynamic analysis method that considers the nonlinear characteristics of the ground.

③ The location of a vertical pile anchorage is preferably determined to ensure that the passive failure plane from the point of $l_{m1}/3$ below the tie member installation point of the anchorage, and the active failure plane from the intersection of the sea bottom and sheet piles do not intersect at the level below the horizontal surface containing the tie member installation point at the anchorage (**Fig. 2.3.4**). The value of l_{m1} is the depth of the first zero point of the bending moment for a free-head pile below the tie member installation point, whereas the horizontal surface containing the installation point of the tie member at the anchorage is assumed as the ground surface.

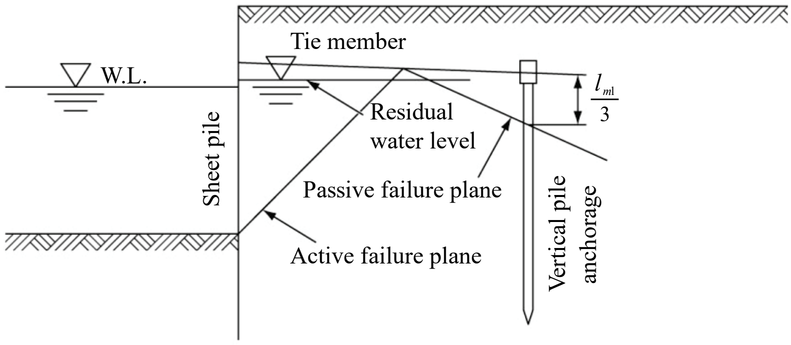


Fig. 2.3.4 Location of Vertical Pile Anchorage

④ The method for determining the location of vertical pile anchorage explained in ③ above is based on the model experiment result by Kubo et al.[33] However, the location obtained through the method is the calculated limit distance that enables the vertical pile anchorage to obtain the predetermined resistance, and the experiment was conducted under conditions that are different from actual ones. Therefore, it is preferable to determine the location of the vertical pile anchorage so that the passive failure plane of a pile drawn from the point $l_{m1}/3$ below the tie member installation point on the pile anchorage and the active failure plane of a sheet pile drawn from the sea bottom intersect with each other at the ground surface.

⑤ The location of a coupled-pile anchorage should be behind the active failure plane of the sheet pile wall drawn from the sea bottom when it is assumed that the tension of the tie member is resisted only by the axial bearing capacity of the piles (**Fig. 2.3.5**). When the tension of the tie member is resisted by both the axial and lateral bearing capacity in consideration of the bending resistance of the piles, it is necessary to locate the anchorage in accordance with the location of the vertical pile.

⑥ Refer to **Part II, Chapter 4, 2 Earth Pressure** for the angle between an active failure plane and a horizontal plane.

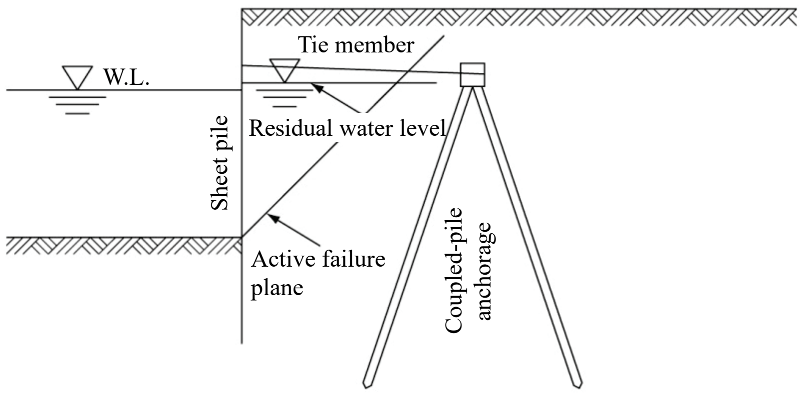


Fig. 2.3.5 Position of Coupled-Pile Anchorage

⑦ The location of a sheet pile anchorage may be determined in accordance with the location of a vertical pile when the sheet piles can be regarded as a long pile. When the sheet piles cannot be regarded as a long pile, the location of the anchorage may be determined by ignoring a part that is deeper than the level $l_{m1}/2$ below the tie member installation point at the sheet pile anchorage and by applying the location determination method for slab anchorage.

⑧ For the method to obtain the first zero point of the bending moment of the vertical pile anchorage and sheet pile anchorage and the method to determine whether a sheet pile anchorage can be considered a long pile, refer to the Port and Harbour Research Institute's method described in **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**.

⑨ For the ordinary sheet pile quaywalls with tie members that run horizontally, an angle of -15° may be used as the wall friction angle in the determination of the passive failure plane that is drawn from the vertical pile anchorage or sheet pile anchorage.

⑩ The location of the slab anchorage is preferably determined to ensure that the active failure plane starting from the intersection of the sea bottom and sheet pile wall and the passive failure plane of the slab anchorage drawn from the bottom of the anchorage do not intersect below the ground surface (**Fig. 2.3.6**).

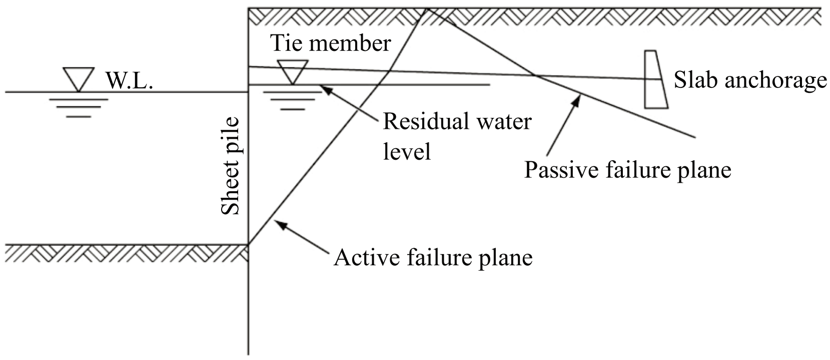


Fig. 2.3.6 Location of Slab Anchorages

2.3.4 Actions

(1) Types of Actions to Be Considered in Respective Design Situations

The stability verification of sheet pile quaywalls shall consider the following actions in respective design situations. However, the performance verification for an accidental situation can be omitted in the case where the sheet pile quaywalls to be designed are not high earthquake-resistance facilities.

① **Permanent state**

The dominant actions shall be the earth pressure acting on wall bodies and self-weight. Refer to **Part II, Chapter 4, 2 Earth Pressure** for the earth pressure.

② **Variable state**

The dominant actions shall be Level 1 earthquake ground motions and the traction force of ships. For the setting of Level 1 earthquake ground motions, refer to **Part II, Chapter 6, 1.2 Level 1 Earthquake Ground Motions Used for the Performance Verification of Facilities**. For the setting of traction force of ships, refer to **Part II, Chapter 8, 2.3 Actions due to the Oscillation of Ships** and **Part II, Chapter 8, 2.4 Actions due to Traction by Ships**.

③ **Accidental situation**

The dominant actions shall be Level 2 earthquake ground motions. For the setting of Level 2 earthquake ground motions, refer to **Part II, Chapter 6, 1.3 Level 2 Earthquake Ground Motions Used for the Performance Verification of Facilities**.

(2) Points of Caution When Setting Actions

① The active earth pressure is normally used as the earth pressure that acts on the sheet pile wall from the backside. For the front-side reaction that acts on the embedded part of the sheet pile, it is necessary to use an appropriate value such as passive earth pressure or a subgrade reaction that corresponds to modulus of subgrade reaction.

② When the free earth support method and the equivalent beam method described in this section are used in the performance verification for a sheet pile wall, the earth pressure and residual water pressure should be assumed to act as those shown in **Fig. 2.3.7**, and the pressure values can be calculated in accordance with **Part II, Chapter 4, 2 Earth Pressure** and **Part II, Chapter 4, 3.1 Residual Water Pressure**. The wall friction angle used for the calculation of the earth pressure acting on the sheet pile wall may usually be taken at 15° for the active earth pressure and -15° for the passive earth pressure when the ground is sandy soil layer.

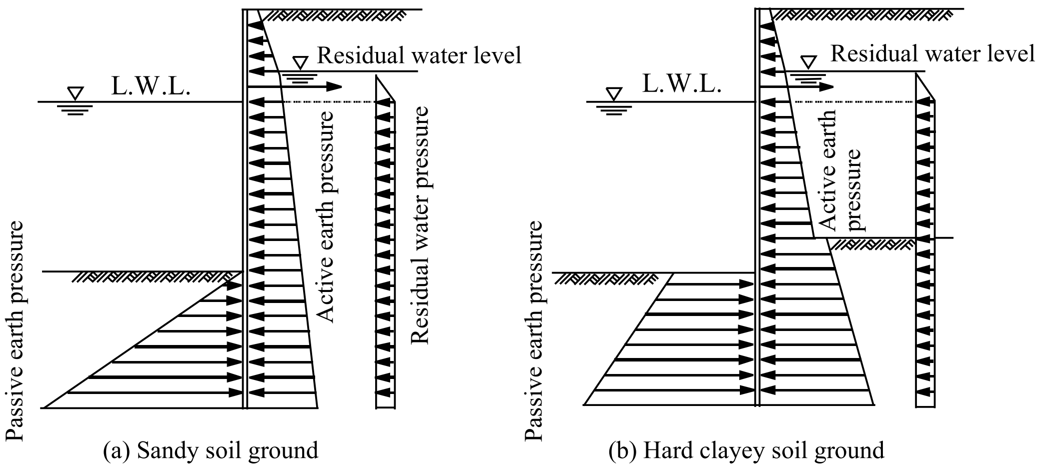


Fig. 2.3.7 Earth Pressure and Residual Water Pressure to Be Considered for the Performance Verification of Sheet Pile Walls

③ Considering that the earth pressure changes in response to the displacement of the sheet pile wall, the actual earth pressure that acts on the sheet pile wall varies depending on the following:

(i) The construction method, i.e., whether backfill is executed or the ground in front of the sheet piles is dredged to the required depth after the sheet piles have been driven in

(ii) The lateral displacement of the sheet pile at the tie member setting point

(iii) The length of the embedded part of the sheet pile

(iv) The relationship between the rigidity of the sheet pile and the characteristics of the sea-bottom ground. Therefore the earth pressure distribution is not necessary as shown in **Fig. 2.3.7**.[34), 35), 36), 37)]

④ When P.W. Rowe's method, i.e., the elastic beam analysis method, is used in a sheet pile stability calculation, it is assumed that the earth pressure and residual water pressure act as those shown in **Fig. 2.3.8**, and a reaction earth pressure that corresponds to the modulus of subgrade reaction and the earth pressure at rest act on the front surface of the sheet pile.

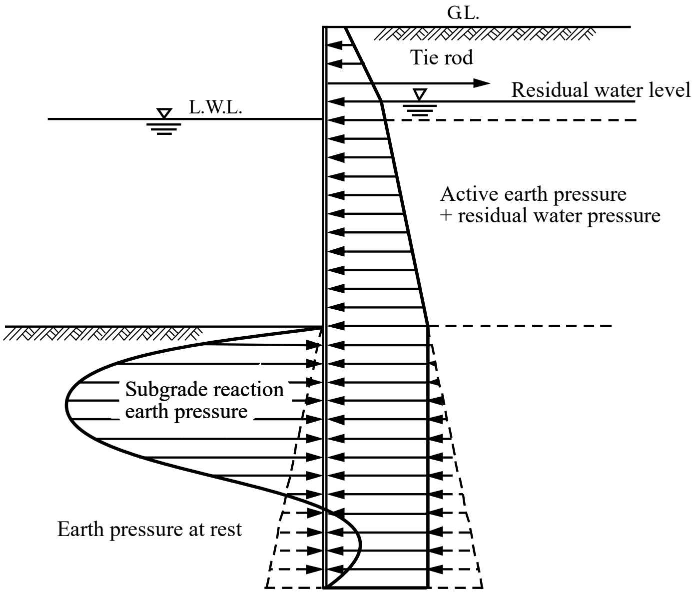


Fig. 2.3.8 Earth Pressure and Residual Water Pressure to Be Considered for the Performance Verification of Sheet Pile Walls Using P.W. Rowe's Method

⑤ When there is cargo handling equipment, such as cranes on the quaywall, it is necessary to take into consideration the earth pressure due to the self-weight and the live load of the equipment.

⑥ In the determination of the reaction force of earth pressure that acts on the front surface of the embedded part of the sheet pile, it is necessary to assume that the dredging of the sea bottom will be executed to a certain depth below the planned depth while considering the accuracy of the dredging work.

⑦ In the case of an earth retaining wall of an open-type wharf, the sea bottom in front of the sheet pile wall has a composite shape of horizontal and sloped surfaces. In such a case, the passive earth pressure may be calculated using Coulomb's method, in which the design passive earth pressure is calculated with several failure planes of different angles. The smallest value among them is adopted as the passive earth pressure.[38)] However, it is necessary to consider the empirical evidence by experiments that the behavior of the ground in front of the sheet pile wall can be well predicted under the assumption of the ground being an elastic body.

⑧ The residual water level to be used in the determination of the residual water pressure needs to be estimated appropriately in consideration of the structure of the sheet pile wall and the soil conditions. The residual water level varies depending on the characteristics of the subsoil and the conditions of sheet pile joints. However, in many cases, the elevation with the height equivalent to 2/3 of the tidal range above the mean monthly LWL is used for sheet pile walls. However, in the case of a steel sheet pile wall driven into cohesive soil ground, care should be exercised in the determination of the residual water level because it is sometimes nearly the same as the high water level. When sheet piles made of other materials are used, it is preferable to determine the residual water level on the basis of the result of investigations of similar structures.

⑨ The seismic coefficient is used in the earthquake-resistance performance verification of sheet pile quaywalls for variable state in respect of Level 1 earthquake ground motion.

(a) For the performance verification of seismic-resistant of sheet piles quaywalls for the variable state in respect of Level 1 earthquake ground motion, the performance verification by the direct evaluation of the amount of deformation by a detailed method such as nonlinear effective stress analysis can be performed. However, simplified methods such as the seismic coefficient method based on the static equation of equilibrium can also be used. In this case, it is necessary to use an appropriate seismic coefficient in accordance with the deformation amounts of facilities in the performance verification and to take into consideration the effects of the frequency characteristics and duration of the ground motions.

(b) The characteristic value of the seismic coefficient for verification used in the performance verification of sheet pile quaywalls with the installation depth of deeper than -7.5 m may be calculated from **Equation (2.3.2)** by using the corrected maximum acceleration α_c and the allowable amount of deformation of the top of the quaywall D_a[10]. For the corrected maximum acceleration, refer to **Reference (Part III), Chapter 1, 1 Detailed Items about Seismic Coefficient for Verification**.

$$k_{h_k} = 1.91\left(\frac{D_a}{D_r}\right)^{-0.69}\frac{\alpha_c}{g} + 0.03 \quad \text{(vertical pile anchorage type)} \qquad \textbf{(2.3.2 (a))}$$

$$k_{h_k} = 1.32\left(\frac{D_a}{D_r}\right)^{-0.74}\frac{\alpha_c}{g} + 0.05 \quad \text{(coupled-pile anchorage type)} \qquad \textbf{(2.3.2 (b))}$$

where

k_{h_k} : characteristic value of the seismic coefficient for verification;

α_c : corrected value of the maximum acceleration of the ground at the ground surface (cm/s^2);

g : gravitational acceleration (980 cm/s^2);

D_a : allowable amount of deformation at the top of the quaywall (15 cm);

D_r : standard deformation amount (10 cm).

(c) For the points of caution when using the seismic coefficient for verification, refer to **Part III, Chapter 5, 2.2.2 Actions, (2) Points of Caution When Setting Actions**.

(d) Setting of the allowable amount of deformation

1) It is necessary to appropriately set the allowable amount of deformation for a facility on the basis of the function required of the facility and the circumstances in which the facility is placed. The allowable value of the standard deformation amount of a sheet pile quaywall in Level 1 earthquake ground motion in **equation (2.3.2)** may be considered D_a = 15 cm.

2) It has been known that sheet pile walls, tie members, and vertical pile anchorage have enough cross-sectional capacity to allow them to undergo deformation of approximately 30 cm, which is the limit value from the viewpoint of serviceability. A D_a value of 15 cm does not cause the cross-sectional force to reach the yield point. However, it requires attention that the relative allowance of the cross-sectional force with respect to deformation becomes smaller as the wall heights decrease.

3) The sheet pile walls driven into very hard ground may have smaller deformation than those constructed in soft ground, but it is necessary to pay attention to the possibility that a small deformation may be the result of large cross-sectional force in members. Therefore, when setting the allowable deformation D_a at a standard value of 15 cm for sheet pile walls that have low wall heights and are driven into very hard ground, the additional deliberation of the earthquake resistance of sheet pile quaywalls shall be executed by 2D nonlinear earthquake response analysis or other appropriate methods.

(e) Considering that there may be a case that the above calculation method results in excessively large seismic coefficients for verification, any of the following measures can be taken when the calculation results in values larger than 0.25 provided that deformation is preferably confirmed directly by a dynamic analysis or other methods even when any of the following measures is used:

1) Setting of a cross section with a seismic coefficient for verification of 0.25 and evaluation of the cross section via a dynamic analysis that is capable of dealing with a dynamic mutual interaction between the ground and a structure

2) Deliberation of ground improvement

3) Adoption of another structural type

4) Deliberation of changing the allowable deformation D_a of a facility without excessively enlarging it while satisfying the requirement to prevent damage to a sheet pile quaywall due to Level 1 earthquake ground motions within a level enabling the function of the facility to be prevented from being impaired for its continuously use

⑩ The seismic coefficient for the verification of the coping of sheet pile under the accidental situation with respect to Level 2 earthquake ground motion may be conveniently calculated with **equation (2.3.2)** by using the acceleration time history of the ground surface at the free ground part. In this case, the allowable amount of deformation D_a may be considered 50 cm.

⑪ When using the method of ⑩ above, the seismic coefficient for the verification of copings shall be up to 0.25 and higher than that for Level 1 earthquake ground motions. However, when the seismic coefficient for the verification of Level 1 earthquake ground motion is higher than 0.25, the seismic coefficient for the verification of copings can also be higher than 0.25. Furthermore, the tensile strength force in tie members obtainable via dynamic analysis can be used when verifying the copings of an anchorage under an accidental situation with respect to Level 2 earthquake ground motions.

⑫ For the dynamic water pressure acting during an earthquake, refer to **Part II, Chapter 4, 3.2 Dynamic Water Pressure**.

⑬ There may be a case of sheet pile quaywalls with large copings that cause their actions on the sheet pile quaywalls due to earthquake ground motions to be too large to be omitted. In such a case, sheet pile quaywalls shall be verified with the possible action on sheet pile walls appropriately evaluated.

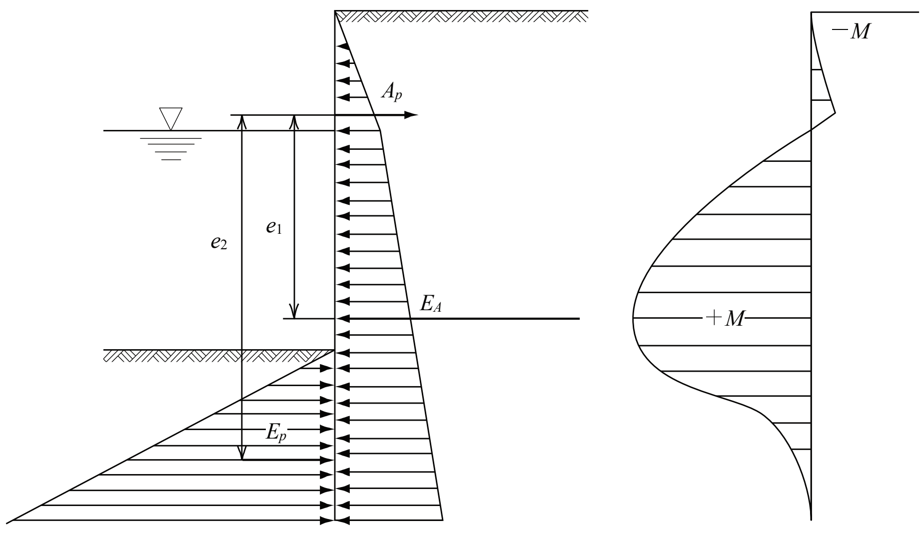
⑭ The fender reaction force is generally considered for the performance verification of the coping. The tractive force of a ship is not considered when the foundation for bollards needs to be constructed separately from the coping. However, when bollards need to be installed on the coping of the sheet pile wall, it is necessary to consider the tractive force of a ship in the performance verification of the coping, tie member, and waling.

2.3.5 Types of Performance Verification Methods for Sheet Pile Walls

(1) Free Earth Support Method[39)]

The free earth support method assumes that there is no negative bending moment in the embedded section of a sheet pile, i.e., a method for analyzing the structural stability assuming that no bending moment exists in the lower end of the embedment. In this method, the earth pressure and bending moment are generally assumed to act on a sheet pile as shown in **Fig. 2.3.9**.

The embedded length can be obtained by balancing the bending moment due to active earth pressure, passive earth pressure, and residual water pressure at the tie member installation position. The tensional force in a tie member can be obtained by subtracting the passive earth pressure from the sum of the active earth pressure and residual water pressure.



(a) Earth pressure + Residual water pressure (b) Flexural moment

$A_p = E_A - E_p \quad E_A e_1 = E_p e_2$

Fig. 2.3.9 Free Earth Support Method

(2) Equivalent Beam Method

The equivalent beam method calculates the maximum bending moment and reaction force at the tie member installation point of the sheet piles by assuming a simple beam supported at the tie member installation point and the sea bottom, with the earth pressure and residual water pressure acting as the load above the sea bottom as shown in **Fig. 2.3.10**.

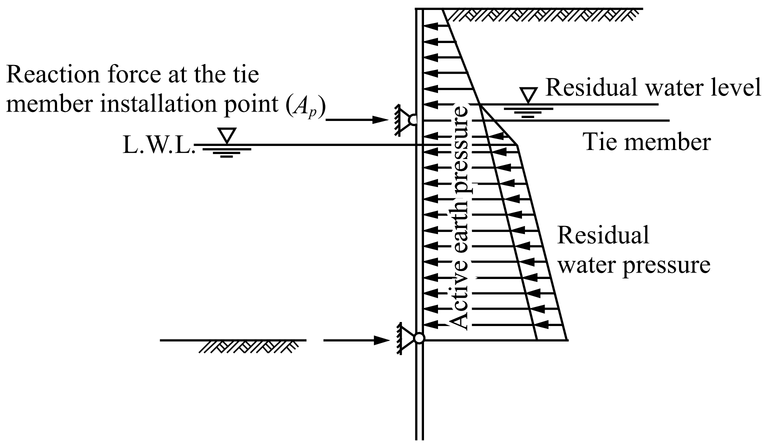
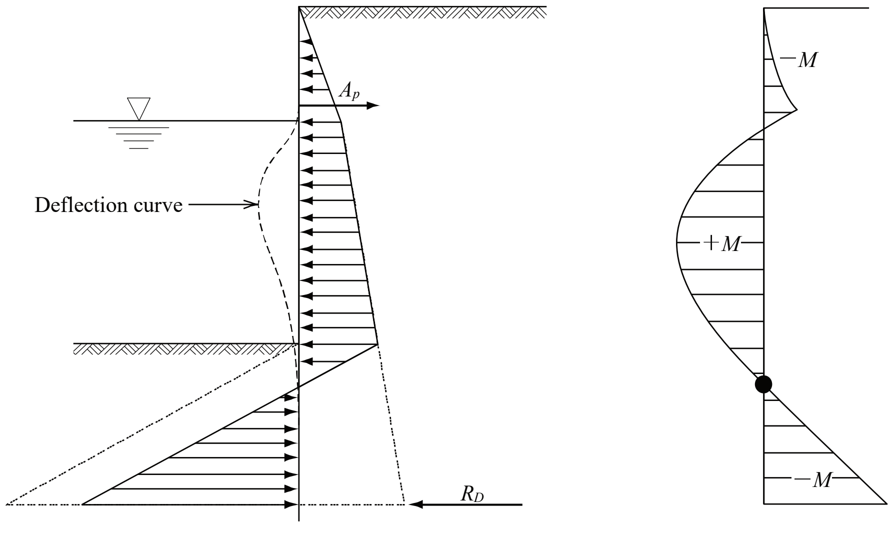


Fig. 2.3.10 Equivalent Beam Method for Obtaining Bending Moment

(3) Fixed Earth Support Methods

The fixed earth support methods analyze structural stability by assuming that a sheet pile is fixed at a certain depth of embedment in the ground. Therefore, the deflection curve of a sheet pile has a point of contrary flexure at a certain depth below a seafloor surface with negative bending moment acting on a section between the point of contrary flexure and the lower end of the sheet pile. Furthermore, these methods consider passive earth pressure in a negative direction at the lower end of the sheet pile, and the passive earth pressure is generally assumed as a concentrated load. In these methods, the earth pressure and bending moment acts on a sheet pile as shown in **Fig. 2.3.11 (b)**.

The deflection curve method is one of the typical fixed earth support methods. The deflection curve method obtains the force acting on a member by assuming an embedded length, drawing a deflection curve that approaches asymptotically to a vertical line at the lower end of the embedment, modifying the embedded length until the deflection at a tie member installation position becomes zero, and repeating the above process until the embedded length converges.



(a) Earth pressure + Deflection curve (b) Flexural moment

Fig. 2.3.11 Fixed Earth Support Method

(4) Elastic Beam Analysis Method for Sheet Piles

① The elastic beam analysis method applies the theory of a beam on elastic foundation to a sheet pile wall with an elastic modulus of subgrade reaction set for the ground where the sheet pile wall is embedded. The basic formula at the embedded section of a sheet pile is expressed as **equation** (**2.3.3**).

$$EI\left(\frac{d^4 y}{dx^4}\right) = p(x) = P_{A_0} - \left(\frac{l_h}{D}\right)xy \qquad \textbf{(2.3.3)}$$

where

E : Young's modulus of a sheet pile (MN/m^2);

I : geometrical moment of the inertia per unit width of the cross section of the sheet pile (m^4/m);

P_{A_0} : load intensity at the sea bottom generated by the active earth pressure and residual water pressure (MN/m^2);

l_h : modulus of the subgrade reaction of a sheet wall (MN/m^3);

D : embedded length of a sheet pile (m).

② Characteristic embedded lengths considering the effect of the cross-sectional stiffness of sheet piles

According to the elastic beam analysis method, the behavioral characteristics of sheet pile walls vary depending on their embedded lengths, i.e., the embedded lengths of sheet pile walls need to be longer than a certain length so that walls can be stabilized. The embedded length that places a sheet pile wall into a critically stabilized state is called a critical embedded length D_C.

When the extension of an embedded length is longer than the critical one, the bending moment on a sheet pile wall reaches peak maximum bending moment M_P in a free earth support state. The embedded length at this time is called a transitional embedded length D_P. The further extension of the embedded length causes the bending moment to reach convergent maximum bending moment M_F in a fixed earth support state. The minimum embedded length at this time is called a convergent embedded length D_F.

When calculating a critical embedded length under the conditions that all of the following partial factors in the performance verification equation with respect to embedded length in the free earth support method is set at 1.0 and that the passive earth pressure at an angle of wall friction δ is -15°, the calculation result is generally larger than a transitional embedded length D_P. This finding indicates that the sheet pile wall with the calculated embedded length has already come close to a fixed earth support state. Therefore, by considering that the free earth support method assumes the triangular distribution of reactive earth pressure similar to the case with passive earth pressure even though a sheet pile wall has already come close to the fixed earth support state and when the rigidity of a sheet pile is not considered in the calculation of embedded lengths, it can be said that the free earth support method cannot reflect actual phenomenon wherein the rigidity and the embedded length of a sheet pile affect the mechanical behavior of the embedded section and the distribution state of the reactive earth pressure of a sheet pile wall in the calculation of embedded lengths.

③ P.W. Rowe's method

Without following classical earth pressure theory, P.W. Rowe's method considers the passive earth pressure at the embedded section of a sheet pile as a subgrade reaction proportional to the lateral deflection and the depth from a seafloor surface and analyzes a sheet pile as a beam on elastic foundation[40]. P.W. Rowe's method requires complicated calculations, but it has been known that the calculation results of the method agree well with experimental results.

④ Correction of P.W. Rowe's method

(a) To simplify P.W. Rowe's method, Ishiguro[41] established the calculation charts of several coefficients necessary for a case of sheet piles embedded in sandy ground with unfixed upper ends (tie member installation position = hinge support) and unconfined lower ends. Furthermore, Takahashi and Kikuchi et al. [42), 43] used P.W. Rowe's method to analyze the behavior of a sheet pile in a fixed earth support state and established a method that enables several characteristic values to be calculated in proportion to the calculation results of the equivalent beam method by using indexes obtained by improving the flexibility numbers proposed by P.W. Rowe. The following is a method that is based on the modification of P.W. Rowe's method and can be used for solving the embedded section of a sheet pile as a beam on elastic foundation.

(b) Considering that there is no general solution to a differential equation of this form, a special technique is required to solve this equation. Broms and Rowe proposed a method for obtaining the coefficient of each term in a numerical analysis by assuming a power series as the solution. By using P.W.Rowe's method[40], Takahashi and Ishiguro[44] published details of a method that can derive a solution of the deflection curve equation of sheet pile wall and a computer-based numerical calculation method. Takahashi and Kikuchi amended this method to better reflect the behavioral characteristics of the actual sheet pile walls (see **Fig. 2.3.12**):

$$EI\left(\frac{d^4y}{dx_4}\right) = p(x) = p_{A_0} + K_{AD}\gamma x - K_0\gamma x - \left(\frac{l_h}{D_F\gamma_f}\right)xy \tag{2.3.4}$$

where

x : depth of the embedded section of a sheet pile below a ground level (m);

y : deformation of a sheet pile wall (m);

E : Young's modulus of sheet pile (MN/m^2),

I : geometrical moment of the inertia per unit width of the cross section of the sheet pile (m^4/m);

P_{A_0} : load intensity at the sea bottom generated by the active earth pressure and residual water pressure (MN/m^2);

K_{AD} : coefficient of active earth pressure in the embedded part of the sheet pile wall;

γ : unit weight of soil (MN/m^3);

K_0 : coefficient of earth pressure at rest;

D_F : convergent embedded length of sheet pile wall (m);

γ_f : ratio of the exerting depth of the primary positive reaction earth pressure acting on the front surface of the embedded part of the sheet pile to D_F.

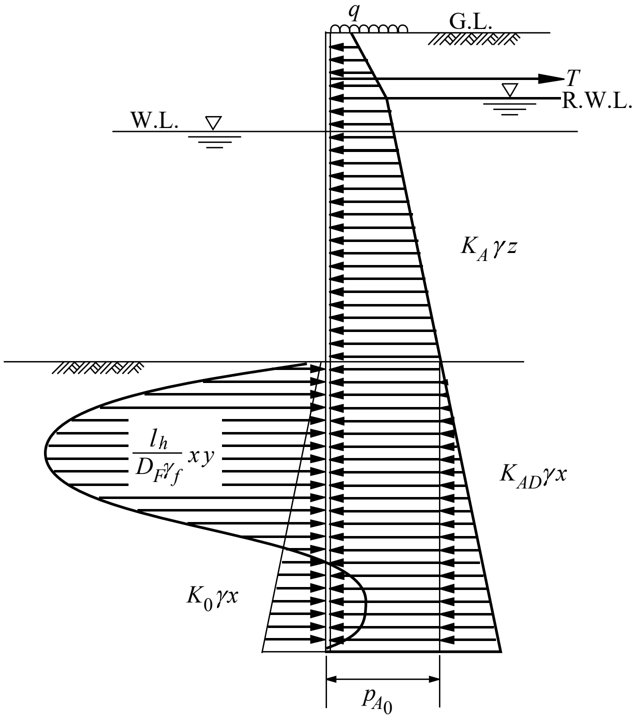


Fig. 2.3.12 Earth Pressure Distribution for the Analysis of Sheet Pile Wall

(c) Flexibility number of the sheet pile

As a measure to indicate the rigidity of a sheet pile wall as a structure, the following flexibility number proposed by Rowe is used in **equation (2.3.5)**:

$$\rho = H^4 / EI \tag{2.3.5}$$

where

ρ : flexibility number (m^3/MN);

H : total length of sheet pile (m);

E : Young's modulus of the sheet pile (MN/m^2);

I : geometrical moment of the inertia per unit width of the cross section of the sheet pile (m^4/m).

For H in **equation (2.3.5)**, P.W.Rowe uses the sum of the total height of the sheet pile wall from the sea bottom to the top of the sheet pile wall H and the embedded length D of a fixed earth support state ($H + D$) as the total length of the sheet pile.

Furthermore, Takahashi and Kikuchi et al. suggest a new index called the similarity number, which is derived using the flexibility number and ground characteristics. The height H_T from the sea bottom to the tie member installation point is used for the length H in this equation:

$$\omega = \rho l_h = (H_T{}^4 / EI) l_h \tag{2.3.6}$$

where

ω : similarity number;

ρ : flexibility number (m^3/MN);

l_h : modulus of subgrade reaction of the sheet pile wall (MN/m^3);

H_T : height from the tie installation point to the seabed surface (m);

E : Young's modulus of the sheet pile (MN/m^2);

I : geometrical moment of inertia per unit width of the cross section of the sheet pile (m^4/m).

By expressing the mechanical characteristics of a sheet pile wall with a similarity number ω, the effect of the rigidity of the sheet piles can be estimated quantitatively.

(d) Modulus of subgrade reaction of sheet piles

Little reference data provide the measured or suggested values of the modulus of the subgrade reaction of the sheet pile (l_h). Therefore, it is preferable to obtain these values by means of model tests and/or field measurements. The proposed values that have traditionally been used include the values proposed by Terzaghi and the ones proposed by Takahashi and Kikuchi et al., which have been obtained by modifying Terzaghi's values. The research conducted by Takahashi and Kikuchi et al. shows that the effect of errors in the modulus of subgrade reaction is not fatal for practical use. Therefore, the values proposed by Takahashi and Kikuchi et al. may normally be used as the coefficient of the subgrade reaction of the sheet pile wall.

1) Values proposed by Terzaghi[45)]

Table 2.3.1 shows the values proposed by Terzaghi.

Table 2.3.1 Modulus of Subgrade Reaction for Sheet Pile Wall in Sandy Ground (l_h) (MN/m^3)

Relative density of sand	Loose	Medium	Dense
Modulus of subgrade reaction (l_h)	24	38	58

2) Values proposed by Takahashi and Kikuchi et al.[43)]

Takahashi and Kikuchi et al.[43)] confirmed that the result of Tschebotarioff's model test of sheet pile wall[46)] does not contradict with the values proposed by Terzaghi. They related the modulus of subgrade reaction listed in **Table 2.3.1** with an *N* value by using the relationship between the modulus of subgrade reaction and the relative density proposed by Terzaghi, as well as the relationship between the *N* value and the relative density by Terzaghi.[47)] They then adopted the smaller value of the modulus of subgrade reaction to be on the safe side and connected the resultant values by using a smooth line (**Fig. 2.3.13**). They related the modulus of subgrade reaction with the angle of shearing resistance (**Fig. 2.3.14**) by using one equation from Dunham's equations for calculating the smaller angle of shearing resistance for a given *N* value:

$$\phi = \sqrt{12N} + 15 \qquad \textbf{(2.3.7)}$$

where

ϕ : angle of shearing resistance (°);

N : *N* value.

However, it should be noted that **Fig. 2.3.14** is an expedient graph to a certain degree because Dunham's equations include cases that provide the larger angle of shearing resistance depending on the grain size of sandy soil. **Figs. 2.3.13** and **2.3.14** also show the values proposed by Terzaghi in addition to the values proposed by Takahashi and Kikuchi, at al.

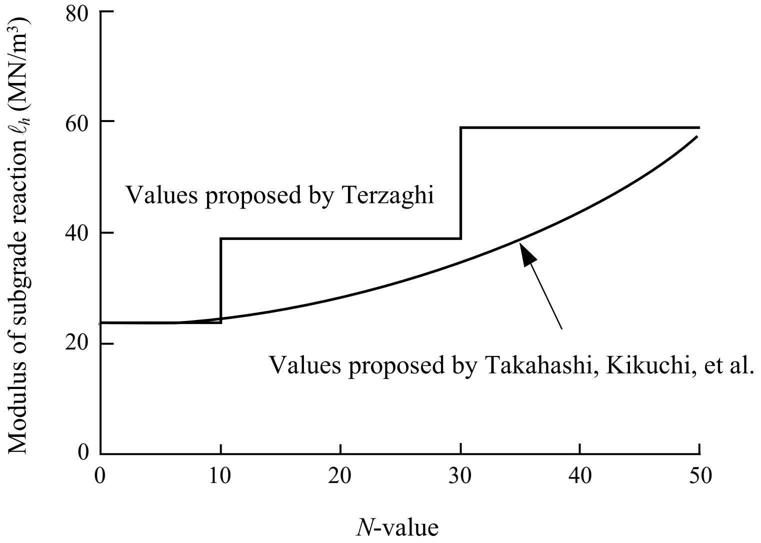


Fig. 2.3.13 Relationship between the Modulus of Subgrade Reaction (l_h) and the *N* value

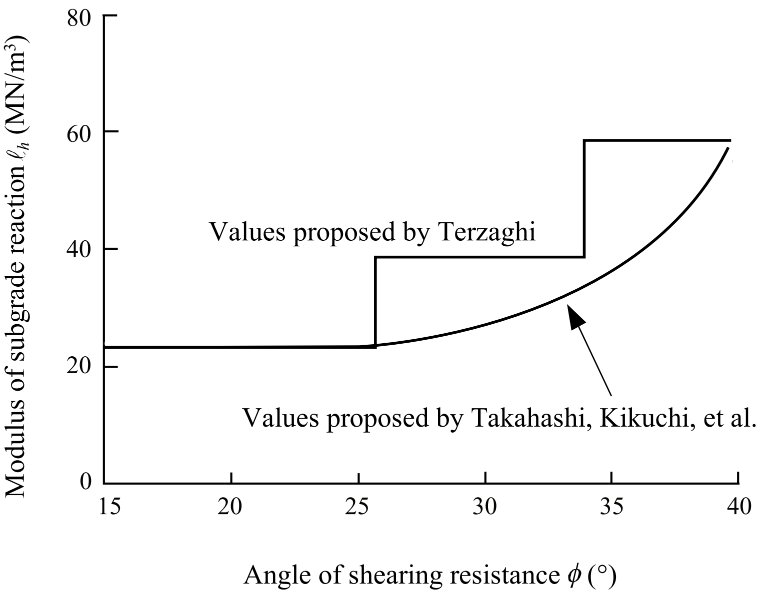


Fig. 2.3.14 Relationship between the Modulus of Subgrade Reaction (l_h) and the Angle of Shearing Resistance (ϕ)

⑤ Numerical calculation model by Morikawa et al.[48)]

By applying the spring model proposed by the Port and Harbour Research Institute to the subgrade reaction at the embedded section of a sheet pile, Morikawa et al.[48)] proposed a numerical calculation model in which the load intensity due to the active earth pressure on a rear face and earth pressure at rest on a front face that acts on the embedded section of a sheet pile wall as actions becomes p_{a0} (the load intensity on a seafloor surface due to active earth pressure and residual water pressure) on a seafloor surface and zero at the lower end of embedment. The bending moment in a sheet pile wall based on this model agrees well with experimental results.

2.3.6 Performance Verification that Takes into Consideration the Effects of the Cross-Sectional Rigidity of Sheet Pile Walls

(1) The cross section of a sheet pile shall be appropriately set by taking into consideration the cross-sectional rigidity of the sheet pile.

(2) The behavior of sheet pile walls with anchorages is strongly affected by the rigidity of sheet piles, ground characteristics, and embedded lengths. Particularly, the rigidity of sheet piles has a strong effect on the decision of embedded lengths. Therefore, it is necessary to study the effects of the cross-sectional rigidity of sheet piles when finally deciding the cross sections of sheet pile walls.

(3) Considering its simplicity and good past record, a method that combines the free earth support and equivalent beam methods has been frequently used. However, the method cannot be the method for the performance verification considering the effects of cross-sectional rigidity.

(4) In place of conventional design methods, the following are methods that take into consideration the effects of the cross-sectional rigidity of sheet pile walls: the combination or comparison of the free earth support and P.W. Rowe's methods when verifying embedded lengths; the combination or comparison of the equivalent beam and P.W. Rowe's methods when verifying the stress on sheet piles and tie members.

2.3.7 Performance Verification for the Overall Stability of Sheet Pile Walls

(1) Performance Verification Items for the Overall Stability of Sheet Pile Walls

When performing the performance verification for the overall stability of sheet pile walls under respective design situations, necessary items shall be appropriately set with reference to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls 【Interpretation】, Attached Tables 11-6 to 10** and **Part III, Chapter 5, 2.1 Common Items for Wharves【Interpretation】, and Attached Tables 11-1, 3 and 4**. In the case where the sheet pile quaywalls to be designed are not high-earthquake-resistance sheet pile quaywalls, performance verification with respect to accidental situation can be omitted.

(2) Performance Verification for the Slip Failure of the Ground under Permanent State

① The performance verification of sheet pile quaywalls with respect to the slip failure of the ground can be performed with reference to the performance verification for the slip failure of the ground in **Part III, Chapter 5, 2.2 Gravity-type Quaywalls**. In this performance verification, the subject to be generally studied is the circular slip failure of the slide planes passing below the lower end of a sheet pile wall. However, there are other cases wherein the circular slip failure of the slide planes passing through sheet pile walls is studied.[49)] The values listed in **Table 2.2.1** of **Part III, Chapter 5, 2.2 Gravity-type Quaywalls** can be used as standard partial factors for the performance verification.

② When a sheet pile quaywall is determined to be unstable against circular slip failure, it is necessary to implement ground improvement by using an appropriate measure or by selecting another structural type. It is not advisable to extend the embedded lengths of sheet piles as a countermeasure against circular slip failure.

(3) Performance verification for the embedded lengths of sheet pile walls under permanent state and variable state with respect to Level 1 earthquake ground motions

① The mechanical behavior of the sheet pile wall varies depending on the embedded length. For a short embedded length, the behavior characteristics are free earth support conditions. For a long embedded length, the behavior characteristics are fixed earth support conditions. To ensure the stability of the sheet pile wall under permanent situations and variable situations, it is preferable that the bottom of the sheet pile is fixed sufficiently in the ground, i.e., fixed earth support conditions are satisfied.

② Conventionally, the embedded length was obtained by the free earth support method on the basis of classical earth pressure theory. Takahashi and Kikuchi[43)] showed that the embedded length obtained with this method by considering appropriate partial factors is considered to be a fixed earth support condition. Furthermore, the equivalent beam method for obtaining the cross section of sheet piles assumes fixed earth support conditions.

③ The mechanical behavior of sheet pile walls with anchorages is largely affected by the rigidity of sheet piles, ground characteristics, and embedded lengths. Particularly, the mechanical behavior significantly differs according the embedded lengths. The performance verification methods under permanent state and variable state explained here are based on the prerequisite that the lower ends of the sheet pile walls are fixed.

④ The embedded lengths of sheet pile walls with their lower ends fixed vary depending on the rigidity of sheet piles and ground characteristics. Deciding the embedded lengths by using the free earth support method and earth pressure theory has a disadvantage in terms of determining the embedded lengths regardless of the stiffness of sheet piles and the discrepancy between the theory and actual behavior represented by the assumed distribution of passive earth pressure in disagreement with the triangle distribution of the Coulomb earth pressure. However, under certain conditions, even the embedded lengths decided via this method can achieve a fixed earth support state.

⑤ When obtaining the embedded length of sheet piles by using the free earth support method, the analysis of the embedded length of the sheet pile wall can be performed using **equation (2.3.8)** on the basis of the equilibrium of moments of the earth pressure and residual water pressure on the point of installation of the tie members (**Fig. 2.3.9**). In the following equation, subscripts k and d indicate the characteristic value and the design value, respectively. Furthermore, the partial factors in the equation can be selected from **Table 2.3.2**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$R_k = aP_{p_k} \tag{2.3.8}$$

$$S_k = bP_{a_k} + cP_{w_k} + dP_{dw_k}$$

where

P_p : resultant passive earth pressure acting on the sheet pile wall (kN/m);

P_a : resultant active earth pressure acting on the sheet pile wall (kN/m);

P_w : resultant residual water pressure acting on the wall structure (kN/m);

P_{dw} : resultant active water pressure acting on the wall body (kN/m) (only during earthquakes);

a to d : distance between the position of installation of the tie member and the point of action of the resultant force (m);

R : resistance term (kN·m/m);

S : load term (kN·m/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.2 Partial Factors Used for the Verification of the Embedded Lengths of Sheet Pile Walls

Verification object	Soil layer compositions	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Embedded length of sheet pile by the free earth support method (Permanent state)	Sandy ground	0.72	1.09	— (1.00)
	Soil layer with the inclusion of cohesive soil	0.77	1.11	
Embedded length of sheet pile by the free earth support method (Variable state with respect to Level 1 earthquake ground motions)	All soil layer	— (1.00)	— (1.00)	1.20

⑥ In **Table 2.3.2**, the soil layer compositions refer to the compositions of the soil layers from the ground surface to the lower end of embedment. If all soil layers are sandy ones, the partial factors for the sandy ground can be used. If cohesive soil is included even partially, the partial factors for the soil layer with the inclusion of cohesive soil can be used.

⑦ The partial factors for verifying the embedded lengths under the permanent state mentioned above are the coefficients calculated by the free earth support method on the basis of past design examples of sheet pile quaywalls with anchorages.[50)]

⑧ Embedded lengths by the P.W. Rowe's method

(a) The characteristic values of the embedded lengths of sheet pile walls using P.W. Rowe's method can be calculated to satisfy **equation (2.3.9)**. Considering that **equation (2.3.9)** considers the rigidity of sheet piles without earth pressure, attention is required to the possibility that earth pressure reduction effects do not necessarily contribute to the reduction in the embedded lengths of sheet piles when planning ground improvement methods for alleviating earth pressure on existing steel sheet pile quaywalls. Therefore, when expecting the earth pressure reduction effects, it is advisable to use the methods ① to ⑤ above in combination with P.W. Rowe's method.

$$\delta_s = \frac{D_F}{H_T} \geq 5.0916\omega^{-0.2} - 0.2591 \tag{2.3.9}$$

where

δ_s : ratio of the embedded length of a sheet pile wall to the height from a tie member installation position to a seafloor surface;

D_F : embedded length of a sheet pile wall (m);

H_T : height from a tie member installation position to a seafloor surface (m);

ω : similarity number (ρl_h);

ρ : flexibility number (H_T^4/EI) (m^3/MN);

E : Young's modulus of a sheet pile wall (MN/m^2);

I : geometrical moment of the inertia per unit width of the cross section of the sheet pile (m^4/m);

l_h : coefficient of subgrade reaction of a sheet pile wall (MN/m^3).

(b) The embedded length calculated with this equation is the convergent embedded length. According to the study conducted by Takahashi and Kikuchi et al., an increase of just 2% in the maximum bending moment occurs when an embedded length corresponding to 70% of the convergent embedded length is employed. Therefore, the use of the convergent embedded length as the design embedded length secures the safety, and there is no need to consider a margin against the safety.

(c) **Equation (2.3.9)** formulates the relationship between the ratio of the convergent embedded length D_F to the virtual wall height H_T, i.e., $\delta = (D_F/H_T)$, and the similarity number ω as shown in **Fig. 2.3.15**. This is based on the analysis performed by Takahashi and Kikuchi et al. by using a simulation model for 72 cases with a combination of conditions for the water depth of the quay (-4 to -14 m), soil conditions, seismic conditions (k_h = 0.20), and material conditions of steel sheet piles. In **Fig. 2.3.15**, δ for permanent situations and earthquake conditions are obtained as δ_N and δ_S, respectively; however, in **equation (2.3.9)**, δ_S is used for the action of earthquakes because it indicates large values.

(d) Furthermore, in the analysis by Takahashi and Kikuchi et al. the relationship between the similarity number ω, ratio μ (M_F/M_T), and ratio τ (T_F/T_T) were studied. The ratio μ is the ratio of the maximum bending moment M_F when there is convergent embedded length D_F in the bending curve analysis to the maximum bending moment M_T calculated by the equivalent beam method by assuming the tie installation point and the seabed surface as the support points. The ratio τ is the ratio of tie tension force T_F when there is convergent embedded length D_F in the bending curve analysis to the tie tension force T_T calculated from the equivalent beam method. These relationships are shown in **Figs. 2.3.16** to **2.3.17**.

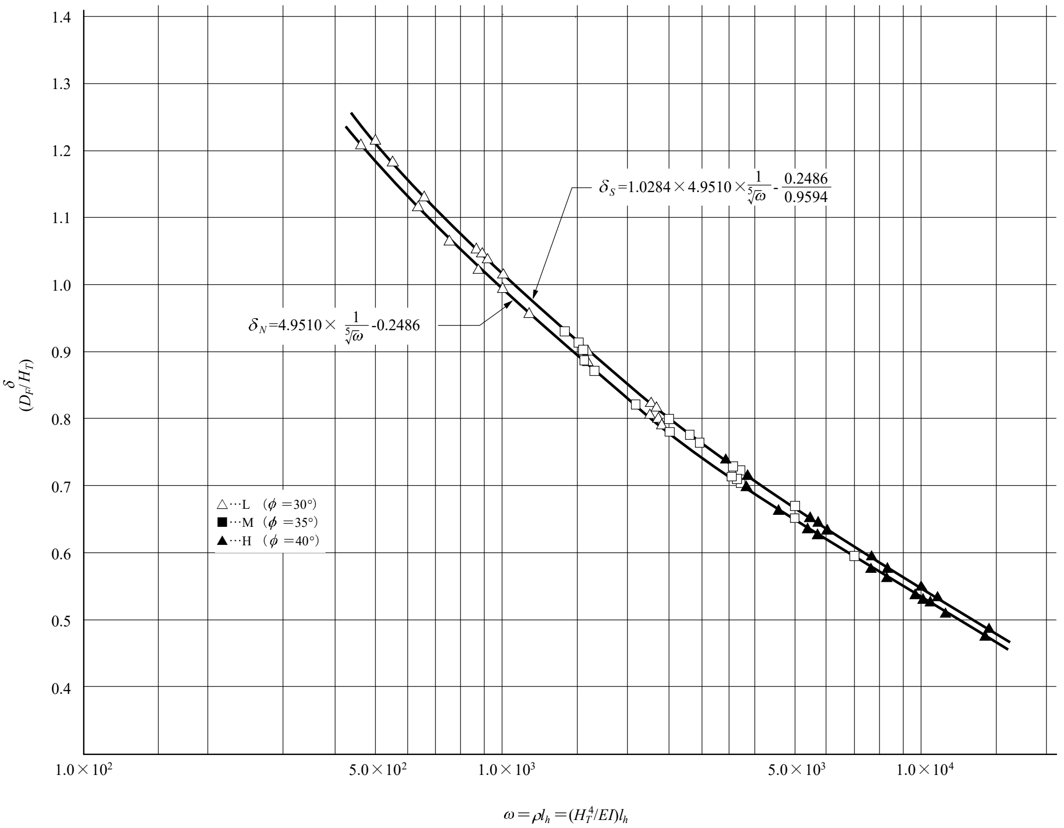


Fig. 2.3.15 Relationship between ω and δ

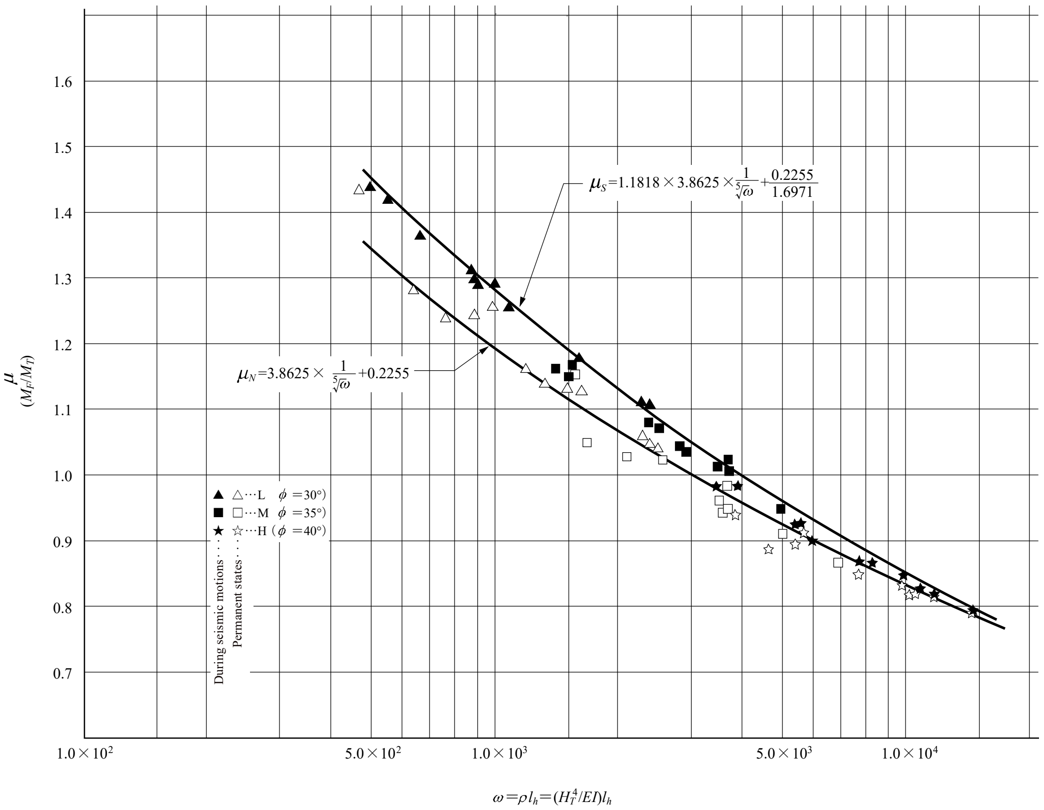


Fig. 2.3.16 Relationship between μ and ω

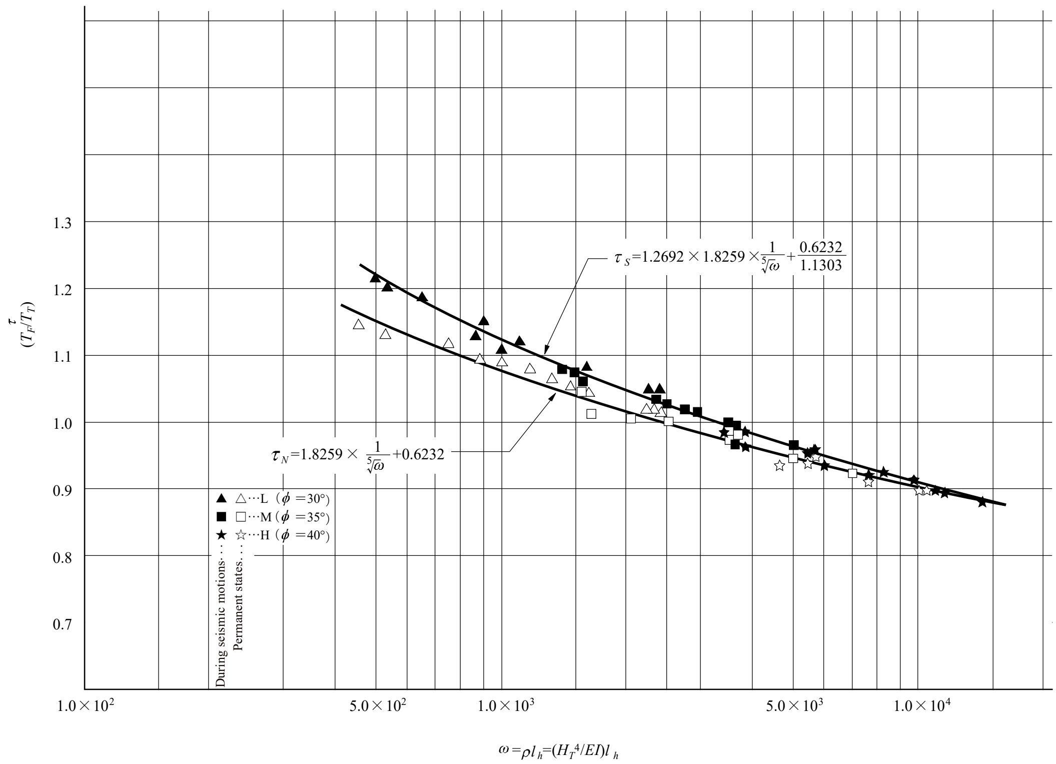


Fig. 2.3.17 Relationship between τ and ω

(e) Comparison of the embedded length by the free earth support and P.W. Rowe's methods

1) Once the fixed earth support state is achieved, the structure of a sheet pile wall is stabilized, and the further extension of an embedded length does not cause any more changes in the maximum bending moment in a sheet pile wall and an action distance D_R (refer to **Fig. 2.3.18**) of the first reaction earth pressure on the front face of the sheet pile wall. Therefore, the required embedded length for a sheet pile quaywall with an anchorage shall be the embedded length that achieves fixed earth support state. In other words, it is rational to set a minimum embedded length that achieves the fixed earth support state as the required embedded length (D_D).

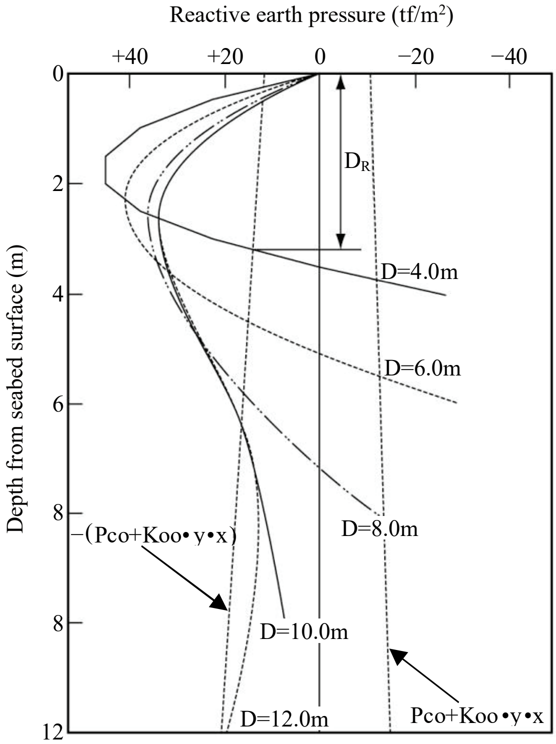


Fig. 2.3.18 Action distance D_R[43)] of the First Reaction Earth Pressure

2) The relationship between the ratio v (D_D/D_t) of the required embedded length (D_D) to the embedded length (D_t) obtained by the free earth support method and the flexibility number ρ is shown in **Fig. 2.3.19**. This figure means that the embedded length obtained by the free earth support method is not sufficient enough to achieve the fixed earth support state in the region with v = 1 or larger.

3) Under any design situation, v is likely to be increased with a decrease in ρ. Therefore, it is necessary to be fully aware of the possibility that the embedded length obtained through the free earth support method may not achieve a complete fixed earth support state.

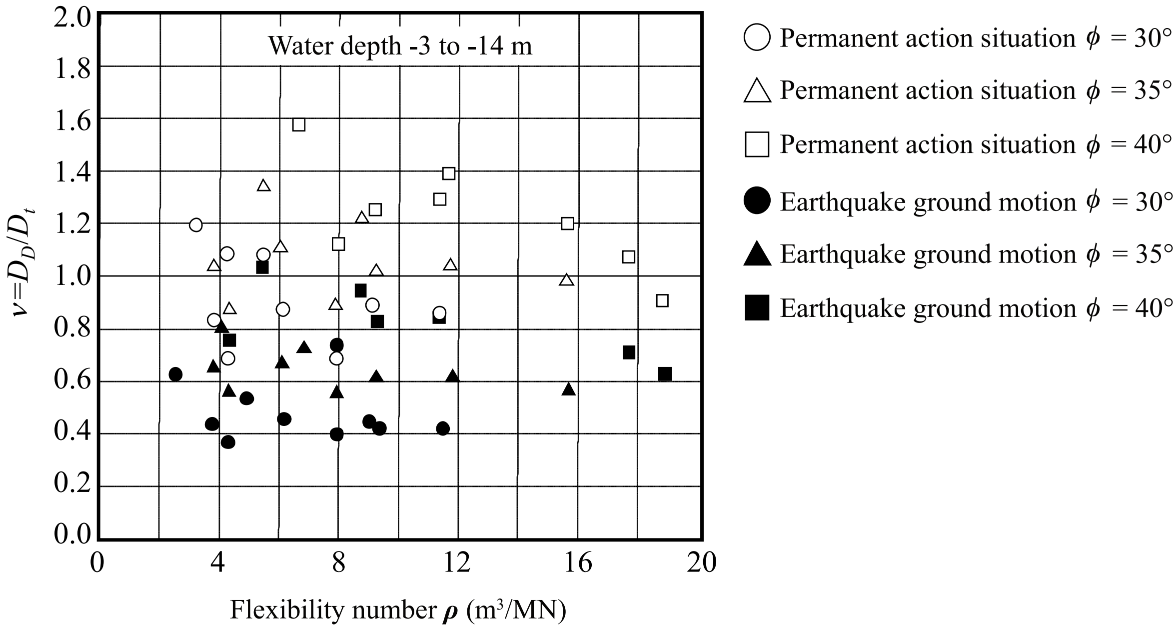


Fig. 2.3.19 Relationship between ρ and v

(4) Performance Verification of Stress on Sheet Pile Walls under Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion

① The maximum bending moment of sheet piles and reaction at the tie member installation point which are necessary for the verification of the stress on sheet pile walls and tie members shall be calculated with an appropriate method that takes into consideration the rigidity and embedded length of the sheet piles and the characteristics of the ground.

② The characteristic values of maximum bending moment in the sheet pile wall and the reaction force at the tie member installation point can normally be calculated using the **equations (2.3.10)** and **(2.3.11)**. In the following equation, subscript k indicates the characteristic value.

(a) The reaction force at the tie member installation point

$$A_{p_k} = P_{a_k} + P_{w_k} + P_{dw_k} - \frac{(aP_{a_k} + bP_{w_k} + cP_{dw_k})}{L} \tag{2.3.10}$$

where

A_p : reaction force at the tie member installation point (kN/m);

P_a : resultant active earth pressure from the top of the sheet piling to the seabed surface (kN/m);

P_w : resultant residual water pressure from the top of the sheet piling to the seabed surface (kN/m);

P_{dw} : resultant dynamic water pressure acting on the sheet pile wall (kN/m) (only during earthquakes);

a to c : distance from the installation position of the tie member to the point of action of the resultant force (m);

L : distance from the installation position of the tie member to the seabed surface (m).

(b) Maximum bending moment

$$M_{\max_k} = aA_{p_k} - bP'_{a_k} - cP'_{w_k} - dP'_{dw_k} \tag{2.3.11}$$

where

A_p : reaction at the tie installation point (kN/m);

P'_a : resultant active earth pressure from the top of the sheet pile to the position where the shear force S becomes 0 (kN/m);

P'_w : resultant residual water pressure from the top of the sheet pile to the position where the shear force S becomes 0 (kN/m);

P'_{dw} : resultant dynamic water pressure from the top of the sheet pile to the position where the shear force S becomes 0 (kN/m) (during an earthquake only);

a : distance from the position where the shear force S becomes 0 to the tie member installation position (m);

b to d : distance from the position where the shear force S becomes 0 to the point of action of the resultant force (m).

③ The maximum bending moment and reaction force at the tie member installation points on sheet piles may be determined using the equivalent beam method or P.W. Rowe's method. However, care should be exercised when using the equivalent beam method for sheet piles with high rigidity because the method causes the point of contraflexure of the bending moment to be deeper than a seafloor surface and may underestimate the sectional force in the sheet piles.

④ When the maximum bending moment of sheet piles is to be determined by taking the effects of the modulus of subgrade reaction and the rigidity of the sheet piles into consideration, the maximum bending moment can be obtained by getting a correction factor (**Figs. 2.3.16** and **2.3.17**) and multiplying the value of the maximum bending moment preliminarily obtained by the equivalent beam method by the correction factor.

Although the characteristic value of the seismic coefficient for performance verification purposes shown in **Figs. 2.3.16** and **2.3.17** has been set at 0.20, the values obtained from these figures may be used for the performance verification under variable state with respect to Level 1 earthquake ground motion.

⑤ The seabed surface used in calculating the bending moment should take the margin of the depth into consideration.

⑥ When the seabed surface in front of a sheet pile is not flat, it is necessary to pay attention to the possible underestimation of a bending moment calculated with the seabed surface as a point of support.

⑦ The analysis of stresses in the sheet pile wall may be performed using **equation (2.3.12)**. In this equation, subscripts k and d indicate the characteristic value and the design value, respectively. The partial factor in the equation can be selected from the values listed in **Table 2.3.3** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = \sigma_{yk} \tag{2.3.12}$$

$$S_k = \frac{M_{\max k}}{Z}$$

where

σ_y : bending yield stress of the steel material (N/mm²);

M_{max} : maximum bending moment in the sheet pile wall (N·mm/m);

Z : section modulus of the steel material (mm³/m);

R : resistance term (N/mm²);

S : load term (N/mm²);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.3 Partial Factors for the Stress Verification of Sheet Pile Walls

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in the sheet pile wall (Permanent state)	0.84	1.18	– (1.00)
Stress in the sheet pile wall (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.12

⑧ The above partial factors for the stress verification of sheet pile walls under a permanent state are the values calculated on the basis of the past design examples of sheet pile quaywalls with anchorages.[50)]

⑨ The adjustment factors for the stress verification of sheet pile walls under a variable state with respect to Level 1 earthquake ground motion are the values set with reference to practical safety factors for the yield stress of steel materials in the previous design methods.

⑩ When the reaction force at the tie member installation point of sheet piles need to be determined by taking the effects of the modulus of subgrade reaction and the rigidity of the sheet piles into consideration, the reaction force at the tie member installation point can be obtained by getting a correction factor from **Figs. 2.3.16** and

2.3.17 and by multiplying the value of the reaction force at the tie member installation point that is preliminarily obtained by the equivalent beam method by the correction factor.

⑪ Refer to **Part II, Chapter 11, 2 Steel Materials** for the yield stress of steel sheet piles.

(5) Performance Verification of Stress on Tie Members under a Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion and the Tractive Forces Of Ships

① The analysis of stresses in tie members may be performed using **equation (2.3.13)**. In this equation, subscripts k and d indicate the characteristic value and the design value, respectively. The partial factor in the equation can be selected from the values listed in **Table 2.3.4** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = \sigma_{yk} \qquad \textbf{(2.3.13)}$$

$$S_k = \frac{T_k}{A}$$

where

σ_y : tensile yield stress of a tie member (N/mm^2);

T : tension force on a tie member (N);

A : cross-section area of a tie member (mm^2);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

For the design value of the tension force in the tie members, refer to ④ **Tension force acting on tie members**.

② The partial factors in **Table 2.3.4** for the stress verification of tie members under a permanent are the values calculated on the basis of past design examples of sheet pile quaywalls with anchorages.[50)]

③ The adjustment factors for the stress verification of tie members under a variable state with respect to Level 1 earthquake ground motion are the values set with reference to practical safety factors for the yield stress of steel materials in the previous design methods.

Table 2.3.4 Partial Factors for the Stress Verification of Tie Members

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in the tie member (Permanent state)	0.64	1.29	– (1.00)
Stress in the tie member (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.67

④ **Tension force acting on tie members**

(a) The tension acting on a tie member can be calculated on the basis of the reaction at the tie installation point calculated in accordance with **(4) Performance Verification of Stress on Sheet Pile Walls under Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion** above.

In this case, the reaction at the tie member installation point should be calculated by taking the rigidity of the sheet pile wall cross section into consideration. Note that the reaction at the tie member installation point that is calculated in accordance with **(4) Performance Verification of Stress on sheet Pile Walls under Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion** above is the reaction per meter of quaywall length. Tie members are usually installed at fixed intervals, and tie members may be attached in some cases at a certain angle with the line perpendicular to the sheet pile wall to avoid the existing structure located behind the wall. Therefore, it is necessary to calculate the tie member tension force by considering these site conditions.

(b) The tension force that acts on a tie member is generally calculated by **equation (2.3.14)**. In the equation below, subscript k stands for the characteristic value.

$$T_k = A_{p_k} l \sec\theta \qquad \textbf{(2.3.14)}$$

where

T : tension force of the tie member (kN);

A_p : reaction at the tie member installation point (kN/m);

l : tie member installation interval (m);

θ : inclination angle of the tie member to the line perpendicular to the sheet pile wall (°).

(c) In some cases, bollards are installed on the coping of a sheet pile wall, and the tractive forces of ships acting on the bollards are transmitted to the tie members. Usually, the coping is assumed to be a beam with the tie members as elastic supports, and the tie member tension force may be calculated using **Equation (2.3.15)** by assuming that the tractive force is evenly shared by four tie members near a bollard. In the equation below, subscript k stands for the characteristic value.

$$T_k = \left(A_{p_k} l + \frac{P_k}{4} \right) \sec\theta \qquad \textbf{(2.3.15)}$$

where

T : tension force acting in the tie member (kN);

A_p : reaction force at the installation point of the tie member (kN/m);

l : spacing of installation of tie members (m);

θ : inclination angle of tie member in perpendicular to the sheet pile wall and the tie member (°);

P : horizontal component of the tractive force of a ship acting on a bollard (kN).

Refer to **Part II, Chapter 8, 2.4 Actions due to Traction by Ships** for details on the tractive forces of ships.

(d) In the case of soft ground with a risk of settlement, a certain degree of safety margin shall be considered when verifying the tension force.

⑤ Tie rods

(a) For the yield stress of tie rods, refer to **Table 2.3.5**.

Table 2.3.5 Characteristics of Tie Rod Materials

Type	Rupture strength (N/mm^2)	Yield stress (N/mm^2)	Elongation (%)	Yield stress/rupture strength
SS400	≥402	(dia. 40 mm or less) ≥ 235	≥24	0.58
		(dia. > 40 mm) ≥ 215	≥24	0.53
SS490	≥490	(dia. 40 mm or less) ≥ 275	≥21	0.56
		(dia. > 40 mm) ≥255	≥21	0.52
High tensile strength steel 490	≥490	≥325	≥24	0.66
High tensile strength steel 590	≥590	≥390	≥22	0.66
High tensile strength steel 690	≥690	≥440	≥20	0.64
High tensile strength steel 740	≥740	≥540	≥18	0.73

(b) Tie rods are subjected to bending moment at their installation positions on a sheet pile when backfill soil at the back of the sheet pile settles. When tie rods made of brittle materials with small elongation are subjected to tension force with bending moment acting on them, the tie rods undergo the reduction in rupture strength. Therefore, it is advisable that tie rods are made of steel materials that ensure characteristics corresponding to the elongation of 18% or more when tested using type 3 test pieces as stipulated in the **Test Pieces for the Tensile Test of Metallic Materials (JIS Z 2201)**.

(c) The tensile stress in the tie rod is calculated using the cross section from which the amount of corrosion has been deducted. For the amount of corrosion, refer to **Part II, Chapter 11, 2.3.3 Corrosion Rates of Steel**.

⑥ Tie wires

(a) The tie rods can be replaced by tie wires made by intertwining hardened steel wires with characteristics that are equivalent to **hardened steel wire rods (JIS G 3506)** or PC steel wires with characteristics equivalent to **piano wire rods (JIS G 3502)**. Considering that tie wires do not have clear yield points, the stress causing 0.2% permanent distortion can be considered the yield point for tie wires, and it is necessary to confirm that the ratio of the yield point to rupture strength does not become lower than 2/3.

(b) When using polyethylene materials as the corrosion protection coating for tie wires, it is necessary to pay attention to ensuring their durability and carefully implementing backfilling and other work so that damage can be prevented to the coating for tie wires.

⑦ In the case where a loose sandy surface layer behind a sheet pile wall is saturated with water, there is a risk of liquefaction on the occurrence of earthquakes and a significant increase in tension force on tie members with earth pressure increased owing to a liquefied sandy layer. In such a case, it is advisable to take necessary measures such as ground improvement to prevent liquefaction.

⑧ Although it is normal practice to select the most economic type of tie rod via a comparative study, tie rods with high yield stress in tension are advantageous when subjected to a large tension force. However, when adopting high-strength steel, it is necessary to pay attention to the fact that the ratio of yield stress to rupture strength of high-strength steel is smaller than that of ordinary steel; when subjected to bent anchorage at 15°, the rupture strength of even the high-strength steel with elongation of approximately 20% may be reduced to approximately 85% of normal anchorage depending on the manufacturing processes.[51)]

2.3.8 Performance Verification of Stresses in Waling

(1) For waling, it is necessary to perform the analysis of stresses in waling under the permanent state and the variable state with respect to Level 1 earthquake ground motion.

(2) The analysis of stresses in waling may be performed using **equation (2.3.16)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected

from the values in **Table 2.3.6** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$R_k = \sigma_{y_k} \qquad \textbf{(2.3.16)}$$

$$S_k = \frac{M_{\max k}}{Z}$$

where

σ_y : bending yield stress in the waling (N/mm^2);

M_{max} : maximum bending moment in the waling (N·mm/m);

Z : section modulus of the waling (mm^3/m);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

For the calculation of the maximum bending moment in the waling, refer to (3) below.

Table 2.3.6 Partial Factors for the Stress Verification of Waling

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in waling (Permanent state)	– (1.00)	– (1.00)	1.67
Stress in waling (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.12

(3) In general, the maximum bending moment of waling may be calculated using **equation (2.3.17)**. In the equation below, subscript k stands for the characteristic value.

$$M_{\max_k} = \frac{T_k l}{10} \qquad \textbf{(2.3.17)}$$

where

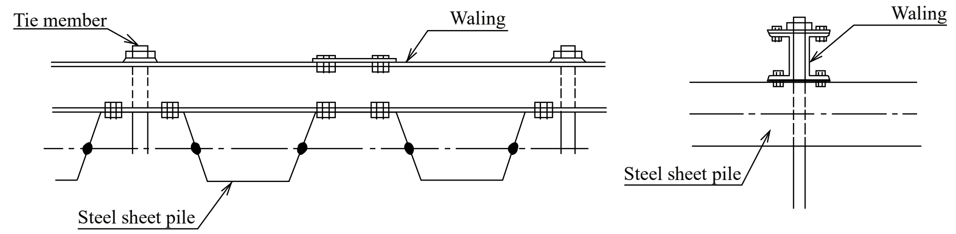
M_{max} : maximum bending moment of waling (kN·m);

T : tension force of a tie member calculated in accordance to **Part III, Chapter 5, 2.3.7 (5) ④ Tension force acting on tie members** (kN);

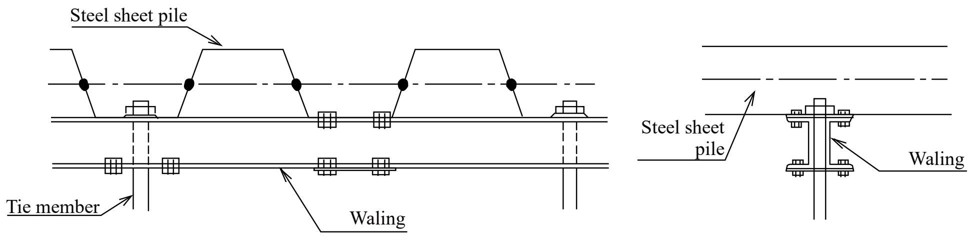
l : tie member installation interval (m).

This equation is obtained by analyzing a three-span continuous beam supported at the tie member installation points and subjected to the reaction at the tie installation point (A_p) as a uniformly distributed load.

(4) Sheet piles and tie members need to be connected via waling materials horizontally installed on the upper portions of the sheet piles. The waling materials are generally fabricated by assembling channel steel (**Fig. 2.3.20**) in general, but angle steel or H-section steel can also be used in place of channel steel.



(a) Case of the installation at the seaside of a sheet pile



(b) Case of the installation at the landside of a sheet pile

Fig. 2.3.20 Examples of Waling Installation

(5) Refer to **Part II, Chapter 11, 2 Steel Materials** for the yield stress of waling.

(6) It is advisable that waling be subjected to less stress and be embedded in copings from the viewpoint of corrosion protection. In the case of waling not embedded in copings, the analysis of the stress applied to the waling shall be based on cross sections without corrosion allowance. For the amount of corrosion, refer to **Part II, Chapter 11, 2.3.3 Corrosion Rates of Steel**.

(7) When bollards are installed on copings, it is necessary to verify the performance of waling near one of the bollards by using a tie member tension force that takes into consideration the tractive force of the ship in accordance with **Part III, 2.3.7 (5) ④ Tension force acting on tie members** above. However, when the wale is embedded in the copings, the effect of the tractive force of the ship may be ignored.

(8) Waling can be installed at either the seaside or landside of sheet piles (refer to **Fig. 2.3.20**). In the case of wales installed at the seaside of sheet piles, the number of locations to bolt the sheet piles to the waling can be reduced without forcibly bolting each sheet pile to the waling because the sheet piles naturally lean on the waling. However, it is necessary to increase the thickness of copings to embed waling fully in concrete because the corrosion on or damage to waling causes the fatal failure of sheet pile walls. In the case of waling installed at the landside of sheet piles, a larger number of bolts that have sufficient strength to fix sheet piles to waling are required. Although construction becomes complex, the thickness of copings for the waling installed at the landside of sheet piles can be reduced compared with the case of waling installed at the seaside.

2.3.9 Performance Verification of Anchorages

(1) The stability of anchorages shall be verified for permanent state and Level 1 earthquake ground motions. Ensuring the stability of anchorages is necessary by using appropriate methods on the basis of the structural characteristics of sheet pile quaywalls and anchorages.

(2) Examination of the Stability of Vertical Pile Anchorages

① The vertical pile anchorages can be verified as vertical piles to which the tension force of tie members is horizontally applied.

② For the performance verification of the vertical pile anchorages, refer to **Part III, Chapter 2, 3.4.6 Deflection of Piles Receiving Force Perpendicular to Axes**.

③ The analysis of stresses in vertical pile anchorages may be performed using **equation (2.3.18)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.3.7** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = \sigma_{y_k} \tag{2.3.18}$$

$$S_k = \frac{M_{\max k}}{Z}$$

where

σ_y : bending yield stress of a pile anchorage (N/mm²);

$M_{\max}$: maximum bending moment in a pile anchorage (N·mm/m);

Z : section modulus of a pile anchorage (mm³/m);

R : resistance term (N/mm²);

S : load term (N/mm²);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.7 Partial Factors for the Stress Verification of Vertical Pile Anchorages

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in vertical pile anchorage (Permanent state)	– (1.00)	– (1.00)	1.67
Stress in vertical pile anchorage (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.12

④ When the active failure plane of a sheet pile and the passive failure plane of a pile anchorage drawn in accordance with **Part III, 2.3.3 Setting of Cross-Sectional Dimensions, (3) Installation Locations of Anchorages** intersect each other at a position lower than a tie member installation point on a pile, the analysis of stresses in vertical pile anchorage can be generally performed by assuming a horizontal plane including the intersection point as a virtual ground surface with no soil above it.[33)]

(3) Examination of the Stability of Coupled-pile Anchorages

① The coupled-pile anchorages can be verified as coupled piles to which the tension force of tie members is horizontally applied.

② The performance verification of the coupled-pile anchorages can refer to **Part III, Chapter 2, 3.4.6 Deflection of Piles Receiving Force Perpendicular to Axes**.

③ The analysis of stresses in coupled-pile anchorages may be performed using **equation (2.3.19)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.3.8** in which the symbol "—" in a column means that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \le 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = \sigma_{y_k} \qquad \textbf{(2.3.19)}$$

$$S_k = \left(\frac{N}{A} + \frac{M_{\max}}{Z} \right)$$

where

σ_y : bending yield stress of a coupled-pile anchorage (N/mm^2);

N : axial force acting on coupled piles (N);

A : cross-section area of coupled piles;

$M_{\max}$: maximum bending moment in a pile anchorage (N·mm/m);

Z : section modulus of a pile anchorage (mm^3);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.8 Partial Factors for the Stress Verification of Coupled-Pile Anchorages

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in coupled-pile anchorage (Permanent state)	– (1.00)	– (1.00)	1.67
Stress in coupled-pile anchorage (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.12

④ The analysis of axial force in coupled-pile anchorages may be performed using the **equation (2.3.20)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.3.9** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \le 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = R_{u_k} \qquad \textbf{(2.3.20)}$$

$$S_k = N_k$$

where

N : axial force of a coupled-pile anchorage (N);

R_u : maximum static axial resistance force of a coupled-pile anchorage (N);

R : resistance term (N);

S : load term (N);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.9 Partial Factors for the Axial Force Verification of Coupled-Pile Anchorages

<table>
<tr><th>Verification object</th><th colspan="2">Type of pile</th><th>Partial factor multiplied by resistance term γ_R</th><th>Partial factor multiplied by load term γ_S</th><th>Adjustment factor m</th></tr>
<tr><td rowspan="2">Axial force acting on coupled-pile anchorage (Permanent state)</td><td colspan="2">Tension pile</td><td>1.00</td><td>1.00</td><td>3.00</td></tr>
<tr><td colspan="2">Compression pile</td><td>1.00</td><td>1.00</td><td>2.50</td></tr>
<tr><td rowspan="3">Axial force acting on coupled-pile anchorage (Variable state with respect to Level 1 earthquake ground motions)</td><td colspan="2">Tension pile</td><td>1.00</td><td>1.00</td><td>2.50</td></tr>
<tr><td rowspan="2">Compression pile</td><td>Bearing pile</td><td>1.00</td><td>1.00</td><td>1.50</td></tr>
<tr><td>Friction pile</td><td>1.00</td><td>1.00</td><td>2.00</td></tr>
</table>

⑤ The surface friction force on the portion of the coupled-pile anchorage above the active failure plane is not generally considered in the analysis of the bearing force of the coupled-pile anchorage when a portion of a coupled-pile anchorage is positioned above an active failure plane of a sheet pile. Furthermore, when determining the position of an anchorage in the verification of a coupled-pile anchorage by taking into consideration of pile bearing force perpendicular to pile axes, it is advisable to position the anchorage with a sufficiently safe distance behind a sheet pile in a manner that assumes the coupled-pile anchorage as a vertical pile anchorage.

(4) Examination of the Stability of Sheet Pile Anchorages

① When the sheet pile anchorage below the tie member installation point is long enough to be regarded as a long pile, the cross section of the sheet pile anchorage may be determined in accordance with **(2) Examination of the Stability of Vertical Pile Anchorages**.

② On the assumption that the earth pressure acts on a range down to $l_{m1}/2$ point below the tie member installation point (**Fig. 2.3.21**), sheet pile anchorage that cannot be regarded as a long pile may be verified in accordance with **(5) Examination of the Stability of Slab Anchorage** below. The length l_{m1} is the vertical distance from the tie member installation point to the first zero point of the bending moment of a sheet pile assuming that the sheet pile anchorage is a long pile.

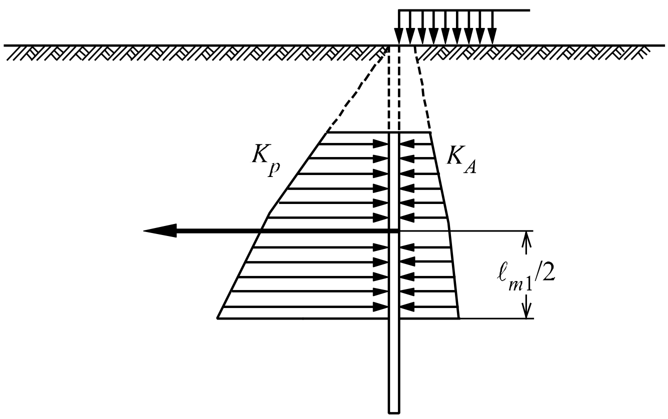


Fig. 2.3.21 Virtual Earth Pressure for Short Sheet Pile Anchorage

③ To determine if a sheet pile anchorage can be regarded as a long pile and to calculate the first zero point of the bending moment of the sheet pile anchorage, refer to the Port and Harbour Research Institute's method shown in **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**.

④ Sheet pile anchorage shall be provided with waling at the tie member installation points in a manner that equally transmits the tension force of tie members to sheet piles. For the performance verification of the waling and waling installation methods, refer to **Part III, 2.3.8 Performance Verification of Stresses in Waling**.

⑤ Short sheet pile anchorages are subjected to large bending moment at the tie member installation points; therefore, it is desirable that the stresses applied to sheet piles are calculated for cross sections without holes for installing tie members and waling fastening bolts.

⑥ When a sheet pile anchorage cannot be installed with an enough distance from the sheet pile body, a double sheet pile structure may need to be examined as an alternative to the sheet pile anchorage. In such a case, refer to **Part III, Chapter 5, 2.7 Double Sheet Pile Quaywalls**.

⑦ There have been reports indicating that the coefficients of lateral subgrade reaction of sheet piles (2D k value) are smaller than those of piles. Therefore, care should be exercised when examining the stability of sheet pile anchorage. For example, in the case of cohesive ground with c_u of 9.8 N/cm^2, the 2D k value is 14.7 N/cm^3 compared with the k value of 19.6 N/cm^3 for piles (by Yokoyama). In the case of sandy ground with an N value of 10, the 2D k value is 14.7 N/cm^3 compared with the k value of 19.6 N/cm^3 for piles.[52)]

(5) Examination of the Stability of Slab Anchorage

① The height and placing depth of slab anchorage may be determined to satisfy **equation (2.3.21)** on the assumption that the tie member tension force and the active earth pressure behind the slab anchorage are resisted by the passive earth pressure in front of the slab anchorage as shown in **Fig. 2.3.22**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.3.10** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = Ep_k \qquad \textbf{(2.3.21)}$$

$$S_k = (A_{P_k} + E_{A_k})$$

where

E_P : passive earth pressure acting on slab anchorage (kN/m);

A_P : reaction at the tie member installation point calculated according to **Part III, 2.3.7 (5) Performance Verification of Stress on Tie Members under a Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion and the Tractive Forces Of Ships** (kN/m);

E_A : active earth pressure acting on slab anchorage (kN/m).

However, to calculate the earth pressure acting on a slab anchorage, it is normally assumed that the surcharge act shown in **Fig. 2.3.22** considers active earth pressure but not passive earth pressure.

Table 2.3.10 Partial Factors for the Stability Verification of Slab Anchorages

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stability of slab anchorage (Permanent state)	– (1.00)	– (1.00)	2.50
Stability of slab anchorage (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	2.00

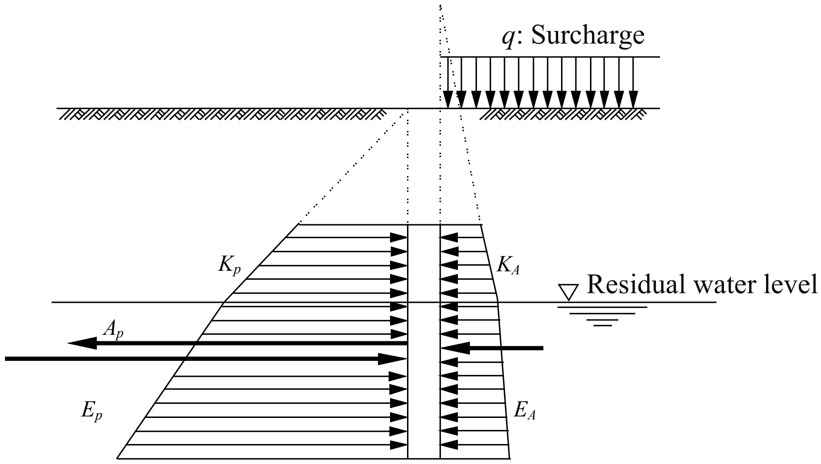


Fig. 2.3.22 Force Acting on Slab Anchorage

② For the calculation of the earth pressure acting on a slab anchorage, refer to **Part II, Chapter 4, 2 Earth Pressure**.

③ The wall surface friction angle used in calculating the earth pressure is normally assumed to be 15° in the case of active earth pressure and 0° in the case of passive earth pressure. However, in the case of a dead man anchor, an upward acting tension force acts on the slab anchorage; therefore, the wall surface friction force acts upwards, which is the opposite of the normal case of passive earth pressure. Furthermore, the passive earth pressure will be reduced. In this case the wall surface friction angle is normally assumed to be 15°.

④ There is no clear definition of a dead man anchor in terms of the inclination angle of a tie member. It has been reported that in the calculation of the sheet pile walls subjected to deformation due to the 1968 Tokachi-oki Earthquake, the calculation results by assuming the slab anchorages with tie members inclined 10° or more with respect to a horizontal plane as dead man anchors agreed well with the actual damage to the sheet pile walls.[53)]

⑤ The causes of the damage to the sheet pile walls in past earthquakes, such as the Niigata Earthquake, are mostly the insufficient resistance of slab anchorages[54)] attributable to the increase in the tension force in tie members as a result of the increase in the earth pressure around the upper part of sheet pile walls due to oscillation and the reduction in the passive resistance of slab anchorages due to the liquefaction of surface layers.

⑥ When the active failure plane of the sheet pile and the passive failure plane of the slab anchorage drawn in accordance with **Part III, Chapter 5, 2.3.3 (3) Installation Locations of Anchorages** intersect below the ground surface level, it is preferable to consider the fact that the passive earth pressure acting on the vertical surface above the intersection point does not function as a resistance force (**Fig. 2.3.23**); it should be subtracted from the design value of E_P of **equation (2.3.21)**. When the intersection point is located above the residual water level, the passive earth pressure to be subtracted may be calculated using **equation (2.3.22)**. In the following equation, subscript k indicates the characteristic value.

$$\Delta E_{P_k} = \frac{K_{P_k} w_k h_f}{2} \quad \textbf{(2.3.22)}$$

where

w : weight of soil (kN/m^2);

h_f : depth from the ground surface to the intersection of the failure planes (m);

K_P : coefficient of passive earth pressure.

The characteristic value w_k for the weight of soil is expressed as the product of the characteristic value for the unit weight of the soil layer under review and the depth h_f from the ground surface to the intersection of the failure planes.

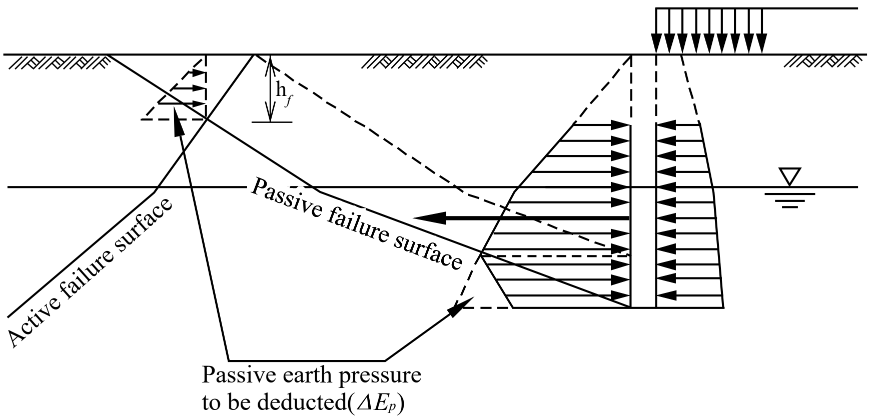


Fig. 2.3.23 Earth Pressure to Be Subtracted from the Passive Earth Pressure that Acts on Anchorage Wall when the Active Failure Plane of the Sheet Pile Wall and the Passive Failure Plane of the Slab Anchorage Intersect

⑦ When a soft cohesive soil layer exists below the area around the bottom of a slab anchorage, there is a risk that the slab anchorage does not have sufficient resistance owing to the generation of a slip surface below the lower edge of the slab anchorage. Therefore, in such a case, it is advisable to examine the stability of a slab anchorage by assuming circular or linear slip surfaces in general.

When examining circular slips, it is generally considered that a slab anchorage is unstable if the slab anchorage is positioned within a slip circle drawn on the basis of an action–resistance ratio of 1.0 or more without considering a sheet pile wall.

When examining linear slips, a slab anchorage can be determined to be stable if an action–resistance ratio with respect to the tension force of a tie member is 1.5 or lower by taking into consideration the resistance to slip of a soil mass obtainable by balancing the force acting on the soil mass defined by a slip plane passing through the lower edge of the slab anchorage, a vertical plane passing through the slab anchorage, and an active failure plane of a sheet pile wall.

⑧ Cross sections of slab anchorages

A slab anchorage should have stability against the bending moment caused by the earth pressure and the tension force of tie members. In general, the maximum bending moment may be calculated by **Equation (2.3.23)** on the assumption that the earth pressure is approximated to an equally distributed load, and the slab anchorage is a continuous slab in the horizontal direction and a cantilever slab fixed at the tie member installation point in the vertical direction. In the following equation, the subscript k indicates the characteristic value.

$$M_{H_k} = \frac{T_k l}{12}$$

$$M_{V_k} = \frac{T_k h}{8l} \qquad \textbf{(2.3.23)}$$

where

M_H : horizontal maximum bending moment (kN·m);

M_V : vertical maximum bending moment per meter in length (kN·m /m);

T : tie member tension according to **Part III, 2.3.7 (5) Performance Verification of Stress on Tie Members under a Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion and the Tractive Forces Of Ships** (kN);

l : tie member interval (m);

h : height of slab anchorage (m).

The layout of the reinforcing bars for M_H may be determined on the assumption that the effective width of the slab anchorage is $2b$ with the tie member installation point as the center, where b is the thickness of the slab anchorage at the tie member installation point.

⑨ Slab anchorages are constructed from reinforced concrete or prestressed concrete. For the performance verification of reinforced concrete and prestressed concrete slab anchorages, refer to **Part III, Chapter 2, 2 Structural Members**.

⑩ In many cases, the installation position of a tie member on a slab anchorage is the point of resultant earth pressure or the center of the heights of slab anchorages.

2.3.10 Performance Verification of Accidental Situation with Respect to Level 2 Earthquake Ground Motion

(1) When performing the performance verification of sheet pile quaywalls that are high earthquake-resistance facilities under accidental situation with respect to Level 2 earthquake ground motions, it is necessary to examine deformation amounts via nonlinear earthquake response analysis or other methods that take into consideration the dynamic interactions of ground and structures.

(2) In sheet pile quaywalls, stress distribution in soil varies depending on construction processes, and the responses of sheet piles and anchorages during earthquakes are largely affected by the initial stress states of the ground. Therefore, it is necessary to select analysis methods that can reproduce the stress distribution in soil before being hit by earthquakes on the basis of construction processes, including the reclamation and excavation of land and the installation of sheet piles and anchorages.

(3) For the performance verification of sheet pile quaywalls against earthquake ground motions, refer to **Reference (Part III), Chapter 1, 2 Basic Items for Earthquake Response Analyses**.

(4) For the standard threshold levels used for the performance verification of deformation amounts under the accidental situation with respect to Level 2 earthquake ground motions, refer to **Part III, Chapter 5, 1.5 Points of Cautions for High Earthquake-resistance Facilities**.

(5) Calculation Method of Threshold Levels for Steel Pipe Members

The limit curvatures, which are the standard threshold levels when performing the performance verification of the damage to sheet piles and anchorages using steel pipe members under the accidental situation with respect to Level 2 earthquake ground motions, can be appropriately set by referring to **Reference (Part III), Chapter 1, 2.5.2 Pile Modeling Methods**.

It is also necessary to consider the effect of diameter–thickness ratios on the load-bearing characteristics of steel pipe members. When assuming that a beam element on the basis of the infinitesimal deformation theory has a bilinear relation between the bending moment and curvatures, the limit curvatures ϕ_u can be calculated with **equation (2.3.24)** in consideration of the diameter–thickness ratio.[56)]

Case of compressive axial force $(N \geq 0)$ $\qquad \phi_u = \mu \phi_y'$

Case of tensile axial force $(N < 0)$ $\qquad \phi_u = \mu \phi_y$ $\qquad$ **(2.3.24)**

where

ϕ_u : limit curvature (1/mm);

ϕ_y : curvature corresponding to yield moment (1/mm);

ϕ'_y : curvature corresponding to yield moment taking into consideration the reduction in yield stress in the axial compression direction (1/mm);

μ : plasticity rate.

The limit curvatures can be calculated by multiplying the curvatures corresponding to the yield moment by the plasticity rate. However, the yield moment is subjected to the reduction in the yield stress in the axial compression direction in accordance with the diameter–thickness ratios. The plasticity rates can be calculated with **Equation (2.3.25)** for each structural type.[56)]

Case of steel pipe sheet pile and vertical pile anchorage $\qquad \mu = \gamma\left(280\,t/D - 1.2\right)$

Case of coupled-pile anchorage $\qquad \mu = \gamma\left[\left(-5.78\,l/r + 440\right)t/D + 0.0506\,l/r - 2.55\right]$ $\qquad$ **(2.3.25)**

where

t : wall thickness (mm);

D : diameter (mm);

l : effective member length (mm);

r : radius of gyration of area (mm);

γ : correction coefficient for yield stress.

The applicability of the correction coefficients has been confirmed for yield stress up to 450 N/mm^2.[57] Furthermore, the effective member lengths that need to be set in accordance with the moment distribution of respective structural types can be appropriately set by referring to **Reference (Part III), Chapter 1, 2.5.2 Pile Modeling Methods**.

2.3.11 Performance Verification of Copings

(1) A coping may be verified as a cantilever beam that is fixed at the top of the sheet pile and subjected to the earth pressure as an action. However, it is necessary to consider the tractive forces of ships and the active earth pressure behind the wall for the parts on which bollards are installed and the fender reaction force and the passive earth pressure behind the wall for the parts on which fenders are installed. The only factor that should be considered with regard to conditions during an earthquake is the active earth pressure.

(2) The tractive forces of ships and fender reactions may be applied (**Figs. 2.3.24 (a)** and **Fig. 2.3.25 (a)**) by assuming that they are acting over the width b of the coping as shown in the figures. In this case, when considering the tractive forces, a surcharge shall be considered in the active earth pressure calculation. When applying the fender reactions, a surcharge shall not be considered in the passive earth pressure calculation. The wall surface friction angle may be taken to be 15° for active earth pressure and 0° for passive earth pressure. For the tractive forces of ships and fender reactions, refer to **Part II, Chapter 8, 2 Actions Caused by Ships**.

(3) The performance verification of copings shall be generally performed by assuming that they are constructed of reinforced concrete.

(4) Copings can be considered as beams on elastic foundations when determining the reinforcement arrangement in a horizontal direction.

(5) It is necessary to ensure the reliable transmission of bending moment acting on copings to sheet piles by sufficiently embedding the upper sections of sheet piles in the copings and by connecting the reinforcement to the sheet piles by welding.

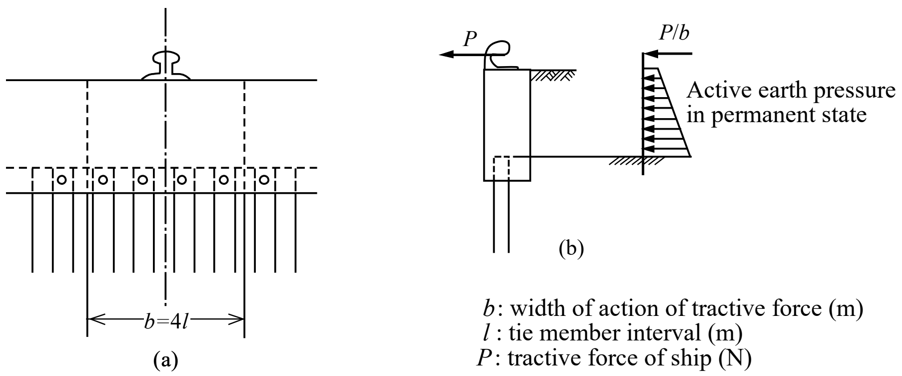


Fig. 2.3.24 Tractive Forces of Ships Acting on Coping

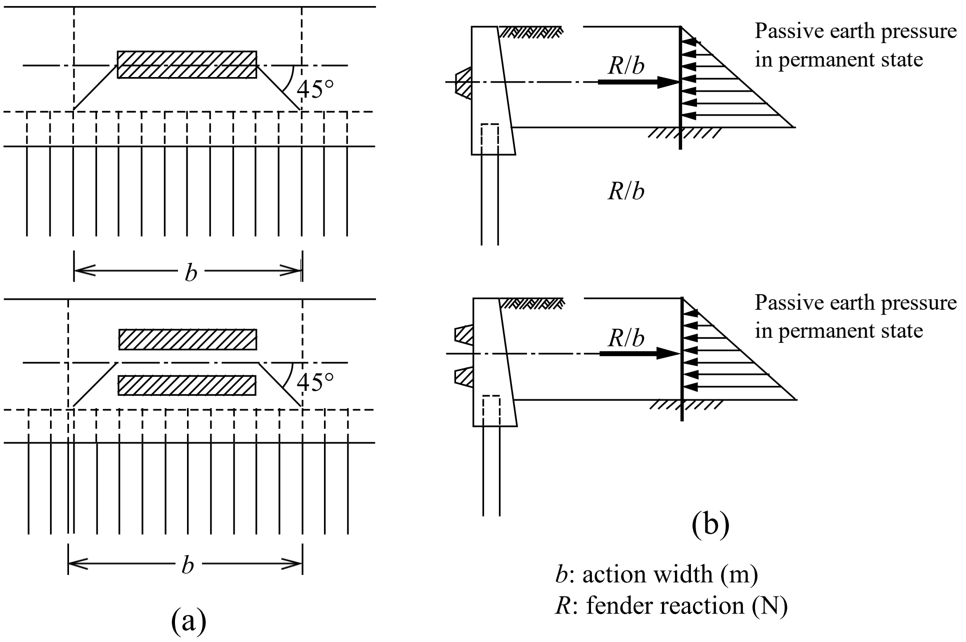


Fig. 2.3.25 Fender Reactions Acting on Coping

2.3.12 Structural Details

(1) Installation of Tie Members, and Waling on Sheet Piles

① Tie members and waling shall be installed on sheet piles so that the horizontal force acting on sheet pile walls is safely and equally transmitted to each tie member via the waling.

② The structural analyses of sheet piles, tie members, and waling are generally performed by assuming that they work integrally as a sheet pile structure. Therefore, the horizontal force acting on a sheet pile wall needs to be equally transmitted to respective tie members.

③ Tie members are generally fixed to sheet piles in a manner that allows the tie members to penetrate the sheet piles via the holes provided on the sheet piles and to be fixed to the sheet piles with washers suitable for installation angles and nuts as shown in **Part III, Chapter 5, 2.3.8 Performance Verification of Stresses in Waling** (**Fig. 2.3.20**).

④ Waling is normally installed by sandwiching tie members and is fixed to the sheet pile with bolts or similar. If waling is installed to the rear of the sheet pile, the cross section of the fastening bolts can be determined from **equation (2.3.26)**. However, if not embedded in the coping, it is necessary to consider a corrosion allowance. In the following equation, subscript k indicates the characteristic value.

$$A = m\frac{A_{p_k} l_w}{n\sigma_{y_k}} \tag{2.3.26}$$

where,

A : bolt cross-sectional area (cm^2);

A_p : reaction at tie member installation point obtained from **Part III, Chapter 5, 2.3.7 Performance Verification for the Overall Stability of Sheet Pile Walls** (N/m);

l_w : spacing of sheet pile fastened to the waling (m), when installed at one position intermediate between tie members, equivalent to a half of the tie member spacing;

n : number of bolts at one location (No.);

σ_y : tensile yield stress of bolt (N/cm^2);

m : adjustment factor.

If bolts are used, the adjustment factor may be taken to be 2.5 for permanent situations and 1.67 for variable situations with respect to the Level 1 earthquake ground motion.

(2) Tie Members

① Tie members shall safely transmit the tension force obtained in **Part III, Chapter 5, 2.3.7 (5) ④ Tension force acting on tie members**, to the anchorages. When bending stress due to the settlement of backfill soil is anticipated, tie members shall be able to cope with such bending stress.

② The performance verification of tie members shall be performed by taking into consideration the fact that tie members fulfill an important role of connecting sheet piles and anchorages and are subjected to uncertain actions.

③ Tie members are generally fixed to sheet piles in a manner that allows the tie members to penetrate the sheet piles via the holes provided on the sheet piles and to be fixed to the sheet piles with nuts installed at the tips of the tie members. The work to drill holes on sheet piles is executed after sheet piles are driven to align the holes, but it is necessary to pay attention to the difficulty in executing the hole drilling work underwater.

④ Tie rods

(a) Tie rods shall be provided with turnbuckles at their joints to make the lengths of the tie rods adjustable (**Fig. 2.3.26**).

(b) Given that tie rods have a risk of being subjected to bending stress owing to the settlement of backfill soil, they shall be provided with ring joints. Tie rods are also subjected to large bending stress at the positions where they are fixed to sheet piles and anchorages; therefore, it is advisable to install ring joints as close to the sheet piles and the anchorages as possible with half the ring joints normally embedded in coping concrete. In some cases with the risk of settlement, tie rods are supported by piles at their centers or protected by casing pipes.

(c) Given that the cross sections of tie rods are reduced when threaded, the thread sections need to have larger diameters than the other section so that the core diameters do not fall below the diameters of the other section of the tie rods.

(d) Turnbuckles, ring joints, and nuts shall not be damaged before tie rods are ruptured.

(e) The safety of turnbuckles shall be verified at the portions having the least cross-section areas.

(f) The items to be verified with respect to the interfaces between ring joints and tie rods shall ensure the safety of portions having the least cross-section areas against tension force, the upper and lower faces of pin holes against shear force applied through pins, and the pins against double shear.

(g) For the threaded portions of nuts and turnbuckles, the safety of the roots of threads against shear force shall be verified with cross-sectional force acting on them multiplied by the safety factor of 1.1 to 1.2 by taking into consideration the possible stress concentration.

(h) For the tensile and shear yield stress of steel materials, refer to **Part II, Chapter 11, 2 Steel Materials**.

⑤ Tie wires

(a) With the compression sections on both ends directly threaded, a tie wire is configured to function as a turnbuckle. Therefore, tie wires need to be verified by taking into consideration the fixation lengths. Furthermore, similar to the case with turnbuckles used for tie rods, performance verification shall be made with respect to the portions with the least cross-section areas.

(b) When constructing coping, the end sections of tie wires shall be embedded in the coping concrete with attention not to cut sleeves.

(c) When tie wires need to be crossed at corner sections, tie wires should be prevented from coming into contact with each other by calculating the sags of tie wires.

⑥ For the materials and accessories of tie wires, refer to the Guideline for **Construction of Steel Sheet Piles**[55] (The Ports and Harbours Association of Japan).

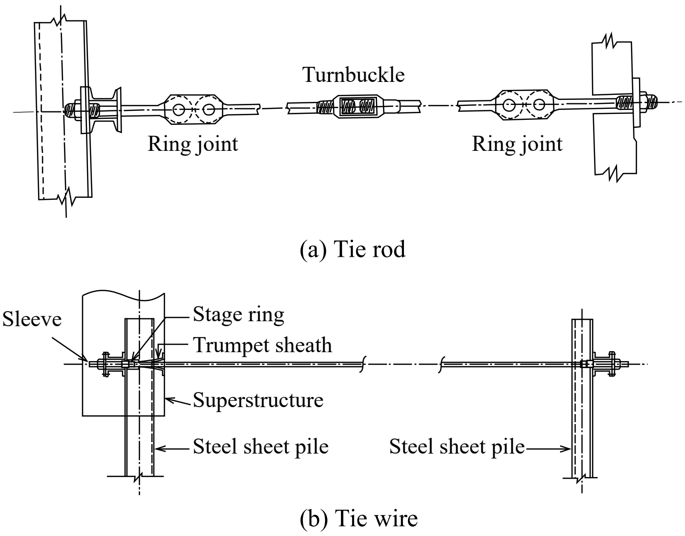


Fig. 2.3.26 Installation of Tie Members

(3) Installation of Anchorages and Tie Members

① The interfaces between anchorages and tie members are structurally important. Therefore, it is necessary to ensure that the tension force in tie members calculated in accordance with **Part III, Chapter 5, 2.3.7 (5) ④ Tension force acting on tie members** can be safely and equally transmitted to anchorages.

② A continuous beam along the face line of a quaywall is usually constructed on top of a pile anchorage, and the tie members are attached to the beam. This beam may be verified for performance as a continuous beam subjected to the tie member tension force and the reaction force of the piles.

③ Tie members are generally fixed to a slab anchorage or the beam on top of pile anchorages in a manner that allows the tie members to penetrate the slab or beam via the holes provided on them and to be fixed to them with washers suitable for installation angles and nuts. The slab thicknesses and the sizes of washers need to be verified on the basis of the fact that the positions on the slabs or beams where the tie members are installed are subjected to bearing and punching stress. Furthermore, it is advisable that the slab has distribution reinforcement in the area around the tie member installation positions to enable the tension force in tie members to be distributed to the slab.

④ Refer to **Part III, Chapter 5, 2.3.12 (1) Installation of Sheet Piles, Tie Members, and Waling** for the installation of tie members to sheet pile anchorages.

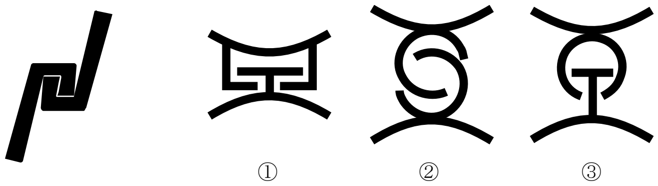
(4) Items Related to the Joints of Steel Sheet Piles

① The types of joints of steel sheet piles are as follows (refer to **Fig. 2.3.27**). For the performance verification of the joint sections, it can be determined that the joint sections satisfy predetermined requirements as long as proven sheet piles walls are used. However, appropriate performance verification is required when adopting new types of joints.

(a) Hook-type joint

(b) Male–female type joint

Hook-type joints are used for hat-shaped and U-shaped sheet piles, and the male–female type joints (①, ②, and ③ in the figure) are used for steel pipe sheet piles.



(a) Hook type (b) Male-female type

Fig. 2.3.27 Shapes of Steel Sheet Pile Joints

② The joint length of steel sheet piles should be as long as possible from the point of view of maintaining the integrity of the sheet piles. However, by taking into consideration the damage to joints during construction, the joints do not normally extend to the bottoms of the sheet piles. Normally, the bottom end of the joint is at the depth where the active earth pressure strength and the passive earth pressure strength are equal or is continuous to the virtual fixity point ($1/\beta$; refer to the virtual fixing point shown in **Part III, Chapter 5, 5.2.2 Setting of Basic Cross-section, (8) Virtual Fixed Point**) and is frequently located 2 to 3 m below the seabed surface. If the residual water level difference is large, the joint length of steel sheet piles should be determined by taking the piping phenomenon into account. The top end of the joint is often extended up to 30 to 40 cm above the bottom surface of the coping.

③ When a U-shaped steel sheet pile is subjected to bending, there is a possibility that vertical slip will occur at joints located at the center of the wall. In this case, the U-shaped steel sheet piles will not act integrally with the adjacent sheet piles. In this situation, the section modulus and geometrical moment of inertia of the cross section calculated by assuming that steel sheet piles act integrally in the wall may not be obtained. The methods for evaluating the effect of this slip in the joints include the method of reducing cross-section performance by multiplying by a joint efficiency coefficient.

(5) Corner Section

① Corner sections are subjected to complex action conditions and require full attention when verifying their performance. Furthermore, corner sections require attention in that they are particularly vulnerable to damage during the action of earthquake ground motions, i.e., they are structural weak points.

② The corner sections of sheet pile quaywalls are particularly vulnerable to damage during the action of earthquake ground motions and require sufficient reinforcement measures. For the performance verification of corner sections, refer to **Part III, Chapter 5, 9.20 Installation Sections**.

③ Special attention is required when determining the structures and installation positions of anchorages because of the possibility that the passive earth pressure regions of anchorages interfere with each other or overlap with the active earth pressure regions of sheet pile walls. Construction should also be performed in a manner that ensures the sufficient compaction of the reclaimed areas around the corner sections.

2.4 Cantilevered Sheet Pile Quaywalls

[Public Notice] (Performance Criteria of Sheet Pile Quaywalls)

Article 50

2 In addition to the provisions in the preceding paragraph, the performance criteria for cantilevered sheet piles shall indicate that the risk in which the amount of deformation of the top of the pile may exceed the allowable limit of deformation is equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating actions are Level 1 earthquake ground motion, ship berthing, and traction by ships.

[Interpretation]

11. Mooring Facilities

(4) Performance Criteria of Sheet Pile Quaywalls

② **Cantilevered sheet pile quaywalls** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance on Criteria and the interpretation related to Article 50, Paragraph 2 of the Public Notice)

(a) The performance criteria and commentary for cantilevered sheet pile quaywalls shall conform to those for sheet pile quaywalls, except the items related to tie member and anchorages, and the following provisions:

(b) The required performance of cantilevered sheet pile quaywalls under the permanent action situation in which the dominant action is earth pressure and the variable action situation in which the dominant actions are Level 1 earthquake ground motions and traction by ships shall focus on serviceability. **Attached Table 11-11** shows the performance verification items and standard indexes used for determining the limit values with respect to the actions.

Attached Table 11-11 Performance Verification Items and Standard Indexes Used for Determining the Limit Values under the Respective Design Situations of Cantilevered Sheet Pile Quaywall

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	50	2	–	Serviceability	Permanent	Earth pressure	Water pressure, surcharge	Deformation of the normal line	Residual deformation amount of the crown of the quaywall
							Variable	Level 1 earthquake ground motions [Traction of ships]	Earth pressure, water pressure, surcharge		

* [ ] indicates the alternative dominant action to be studied as design situations.

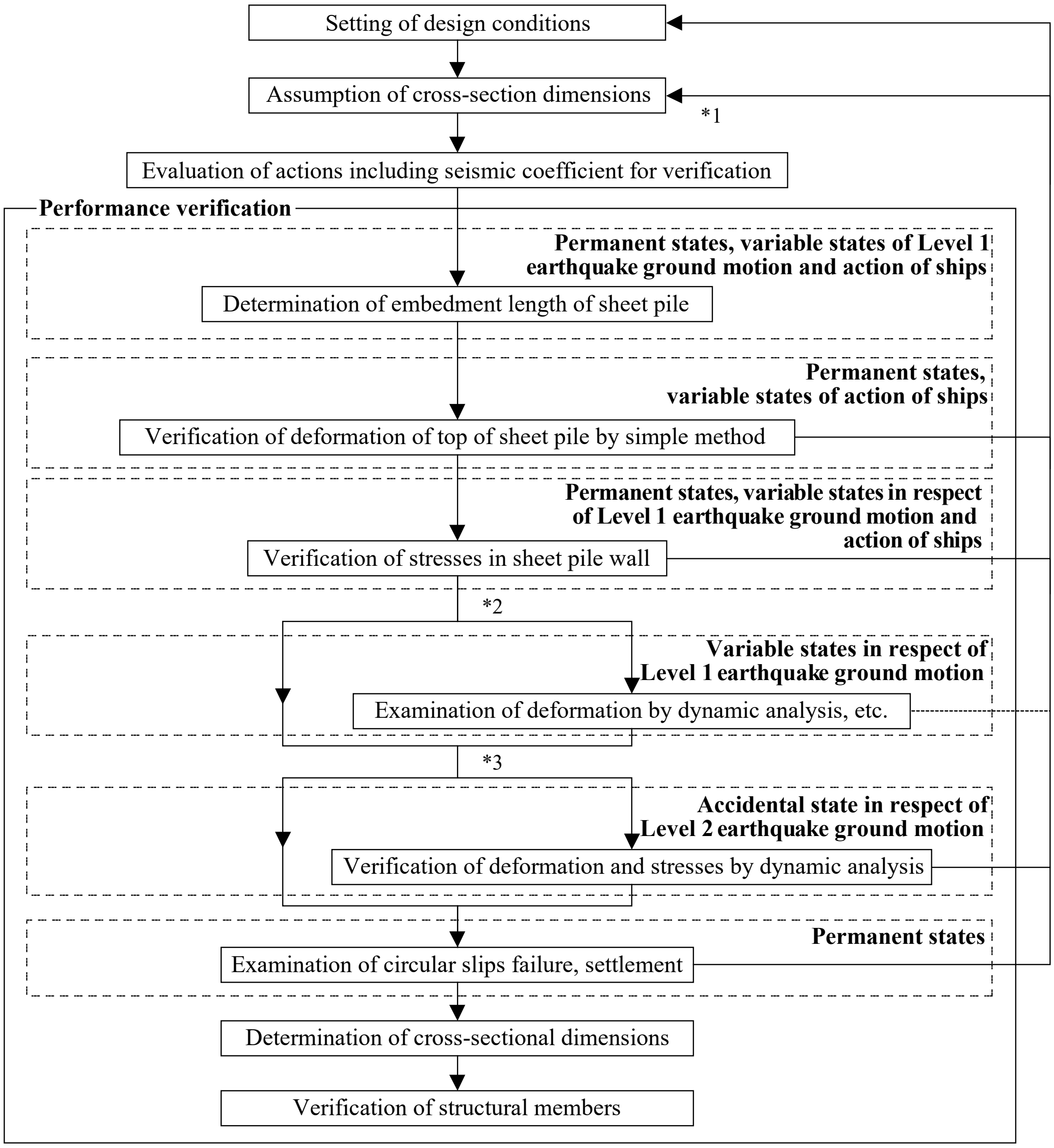
(c) In addition to the above, the requirements and the commentaries in Paragraph 3 (Scouring and Wash Out), Article 22 and Article 28 (Performance Criteria of Armor Stones and Blocks) shall be applied as needed.

2.4.1 General

(1) The descriptions in this section can be applied to the performance verification of mooring facilities that use cantilevered sheet pile walls to support the soil behind them.

(2) The performance verification methods described here are applicable to sheet pile walls driven into sandy soil ground but not to those driven into cohesive soil ground. There have been many unknown factors in the performance verification methods for cantilevered sheet pile walls driven into clayed soil ground. Furthermore, from an engineering viewpoint, because the structure of cantilevered sheet pile walls is susceptible to creep, the application of cantilevered sheet pile walls to clayed soil ground is preferably avoided.

(3) **Fig. 2.4.1** shows an example of the sequence of the performance verification of cantilevered sheet pile quaywall. However, it should be noted that **Fig. 2.4.1** does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions. Thus, regarding liquefaction, the possibility of and countermeasures against liquefaction shall be appropriately deliberated with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the performance of cantilever sheet pile walls under a variable action situation with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method on the basis of the static equation of equilibrium. However, for high earthquake-resistance facilities, it is preferable that the deformation is deliberated by a nonlinear seismic response analysis or other methods that take into consideration the dynamic interaction between the ground and structures. For cantilevered sheet pile quaywall other than high earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.



*1: Evaluation of the effect of liquefaction is not shown, so it is necessary to consider these separately.
*2: When necessary, an examination of the amount of deformation by dynamic analysis can be carried out for Level 1 earthquake ground motion.
For high earthquake-resistance facilities, it is preferable that the amount of deformation be examined by dynamic analysis.
*3: Verification in respect of Level 2 earthquake ground motion is carried out for high earthquake-resistance facilities.

Fig. 2.4.1 Example of the Sequence of the Performance Verification of Cantilevered Sheet Pile Quaywall

(4) Cantilever sheet pile walls have a structure that resists the earth and water pressure acting on the rear faces of sheet piles with the horizontal subgrade reaction at the embedded sections of the sheet piles. The flexural moment on cantilevered sheet pile walls can be calculated with reference to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(5) Sheet piles of cantilevered sheet pile quaywall are subjected to a severely corrosive environment. So the sheet piles shall be designed with appropriate corrosion protection measures on the basis of **Part III, Chapter 2, 1.3.4 Examination of Performance Deterioration with Age**.

(6) **Fig. 2.4.2** shows an example of a cross section of a cantilevered sheet pile quaywall.

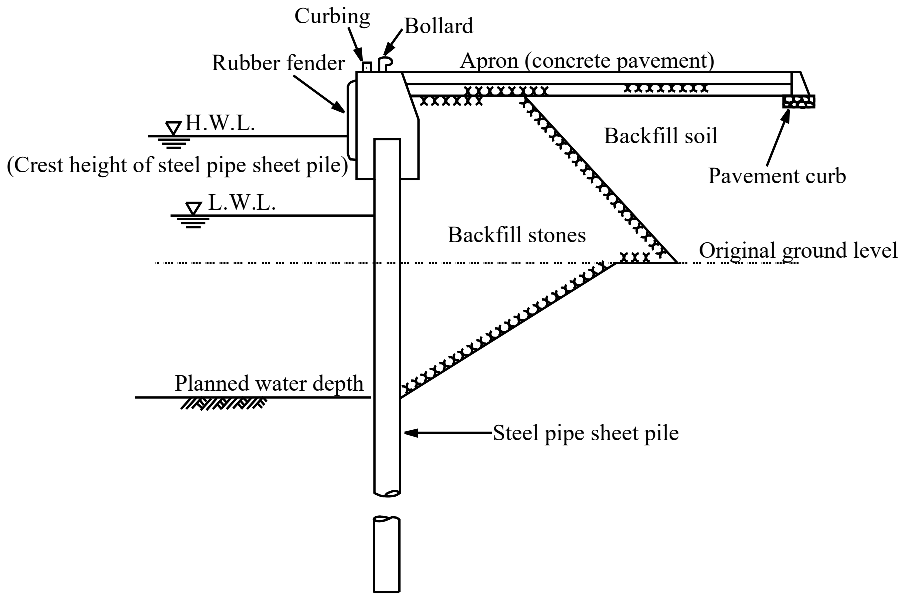


Fig. 2.4.2 Example of the Cross Section of Cantilevered Sheet Pile Quaywall

2.4.2 Actions

(1) Types of Actions to Be Considered in Respective Design Situations

For the types of actions to be considered in the stability verification of cantilevered sheet pile quaywall, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(2) Points of Caution When Setting Actions

① In the case of sandy seabed ground, the earth pressure and residual water pressure is generally considered to act on a cantilever sheet pile at a point above a virtual sea bottom (**Fig. 2.4.3**) where the sum of the active earth pressure and the residual water pressure is equal to the passive earth pressure.

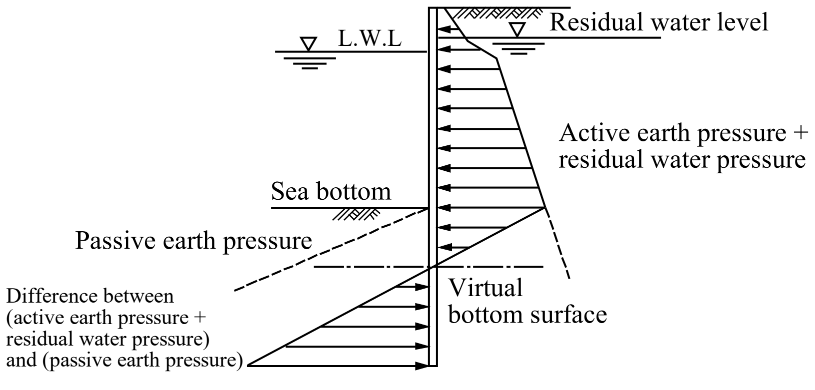


Fig. 2.4.3 Virtual Sea Bottom

② In the seabed ground just below a seafloor surface, the sum of the active earth pressure and the residual water pressure at the rear face of a sheet pile wall is larger than the passive earth pressure on the front face of the sheet pile wall. Therefore, the soil near the seafloor surface in front of the sheet pile wall has a risk of undergoing plastic deformation and will not produce elastic earth pressure, which acts as spring reaction force of the ground. Thus, with a plane called a virtual sea bottom, wherein the sum of the active earth pressure and the residual water pressure is equal to the passive earth pressure, as a boundary, the differential pressure obtained by subtracting the passive earth pressure from the sum of the active earth pressure and the residual water pressure can be considered to act on the portion of the sheet pile wall above the virtual sea bottom, and only the spring reaction force of the ground can be considered to act on the portion of the sheet pile wall below the virtual sea bottom without considering the active earth pressure acting on the rear face of the sheet pile. For performance verification during the action of earthquake ground motions, the dynamic water pressure acting on the portion of the sheet pile wall above the sea bottom should be considered.

③ **Equation (2.4.1)** can be used to calculate the characteristic value of the seismic coefficient to be used in the performance verification of cantilevered sheet pile quaywall under the variable action situation with respect to Level 1 earthquake ground motions.[58)] In this equation, the subscript k represents the characteristic value. For the filter and reduction coefficient required when reflecting the frequency characteristics into the calculation of the maximum acceleration α_c in the equation, refer to **Reference (Part III), Chapter 1, 1 Detailed Items about Seismic Coefficient for Verification**.

$$k_{h_k} = 1.40\left(\frac{D_a}{D_r}\right)^{-0.86} \cdot \frac{\alpha_c}{g} + 0.06 \qquad \textbf{(2.4.1)}$$

where

k_h : seismic coefficient for verification;

D_a : allowable deformation amount (20 cm);

D_r : reference deformation amount (10 cm);

g : gravitational acceleration (980 cm/s^2);

α_c : corrected maximum acceleration on a ground surface (cm/s^2).

④ The maximum flexural moment in a sheet pile wall shall be calculated appropriately by using an analysis method that corresponds to the mechanical behavior characteristics of the sheet pile wall. The maximum flexural moment in a sheet pile wall is normally calculated in accordance with **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**.

⑤ For the calculation of the lateral resistance of a pile, refer to **Part III, Chapter 2, 3.4.6 Deflection of Piles Receiving Force Perpendicular to Axes**.

⑥ Unlike the actions on a pile, those on a cantilevered sheet pile wall are distributed loads; therefore, the maximum flexural moment cannot be expressed by a simple equation. In the calculation of the maximum flexural moment, the distributed loads acting on a sheet pile wall can be replaced with concentrated loads acting on the center of gravity of the distributed loads.

2.4.3 Performance Verification

(1) Performance Verification of the Stress in Sheet Pile Walls

① When steel pipes are used as sheet piles, secondary stress often develops in the steel pipes of a sheet pile wall owing to the deformation of the steel pipe cross section (i.e., a circular cross section is deformed into an elliptic one) due to the earth and residual water pressures. Cantilevered sheet pile walls are structures that tend to experience large displacement, and there is a risk that a relatively high secondary stress may develop in the areas around the point where the flexural moment becomes maximum. A larger steel pipe diameter leads to a higher level of secondary stress. Therefore, in such a case, it is preferable to perform an examination of strength against the secondary stress. The secondary stress of a steel pipe is calculated using **equation (2.4.2)**:

$$\sigma_t = \alpha p \left(\frac{D}{t}\right)^2 \times 10^{-3} \qquad \textbf{(2.4.2)}$$

where

σ_t : secondary stress (N/mm^2);

p : earth pressure and residual water pressure acting on the sheet pile wall (kN/m^2);

D : diameter of a pile (mm);

t : plate thickness of a pile (mm);

α : coefficient.

The coefficient α in the equation may be defined with reference to **Fig. 2.4.4** by taking into consideration the width of action, foundation conditions, and constraint conditions.[59)] In this figure, "Sliding" and "Fixed" indicate the displacement conditions of the joints of steel pipe sheet piles. "Fixed" assumes that the joints of steel pipe sheet piles are subjected to a filling treatment with concrete or other materials.

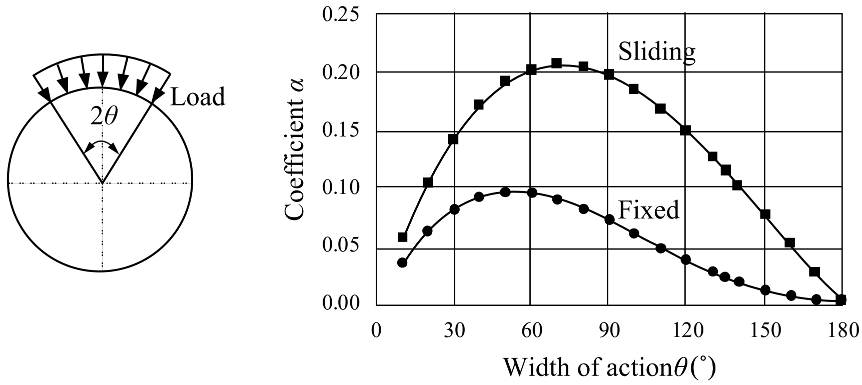


Fig. 2.4.4 Coefficient α for Secondary Stress

② The verification of stress may be performed using **equation (2.4.3)** on the basis of the axial stress σ_l in the pile obtained in accordance with **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles** and the secondary stress σ_t obtained from **equation (2.4.2)**. In the following equation, subscripts k and d indicate the characteristic value and the design value, respectively. Furthermore, the partial factors in the equation can be selected from **Table 2.4.1**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience. Here, it is necessary to use positive secondary stress when the flexural yield stress is negative and negative secondary stress when the flexural yield stress is positive.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_s S_k$$

$$R_k = f_{yk} / \gamma_m \tag{2.4.3}$$

$$S_k = \gamma_b \sqrt{\sigma_{lk}^{\ 2} + \sigma_{tk}^{\ 2} - \sigma_{lk}\sigma_{tk}}$$

where

σ_l : axial stress of a pile (N/mm^2);

σ_t : secondary stress of a pile (N/mm^2);

f_{yk} : yield stress of a pile (N/mm^2);

γ_m : material coefficient (1.05);

γ_b : member coefficient (1.1);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.4.1 Partial Factors Used for the Verification of Stress in Sheet Pile Walls

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in sheet pile wall (Permanent state)	– (1.00)	– (1.00)	1.20
Stress in sheet pile wall (Variable state of Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.00

(2) Performance Verification of the Embedded Lengths of Sheet Piles

The embedded lengths of sheet piles shall be equal to or longer than the effective length of piles calculated in accordance with **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**. Considering that cantilevered sheet piles retain earth behind them on the basis of the same mechanism as piles do, the embedded lengths of sheet piles can be calculated in the same way as that for piles. When using the PHRI method on the basis of the lateral resistance of piles in the calculation of embedded lengths, the required embedded length is expressed as 1.5 l_{m1}, where l_{m1} is the depth of the first point of zero flexural moment on a free head pile. In such a case, it should be noted that the embedded length needs to be measured not from a seafloor surface but from the virtual bottom surface.

(3) Performance Verification of the Displacement Amounts of the Tops of Sheet Pile Walls

① There are many unknown factors on the deformation of cantilevered sheet pile walls during the action of earthquake ground motions. Considering that there may be cases wherein the method described in this paragraph produces different results from those of accurate dynamic analyses, it is preferable that dynamic analyses are used for the performance verification of the displacement amounts of the tops of sheet pile walls during earthquakes.

② The displacement amount δ of the tops of a sheet pile wall can be expressed by the sum of the following three values (refer to **Fig. 2.4.5**):

(a) The deflection amount δ_1 of a sheet pile wall at a virtual bottom surface

(b) The deflection amount δ_2 of the portion of the sheet pile wall above the virtual bottom surface

(c) The deflection amount δ_3 of the top of the sheet pile wall generated as a result of the rotation of the portion of the sheet pile wall above the virtual bottom surface corresponding to the deflection angle of the sheet pile wall at the virtual bottom surface

The deflection amounts δ_1 and δ_3 above can be generally calculated by the PHRI method described in **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**. Furthermore, the deflection amount δ_2 is generally calculated as the deflection of a cantilever beam subjected to earth pressure behind the beam.

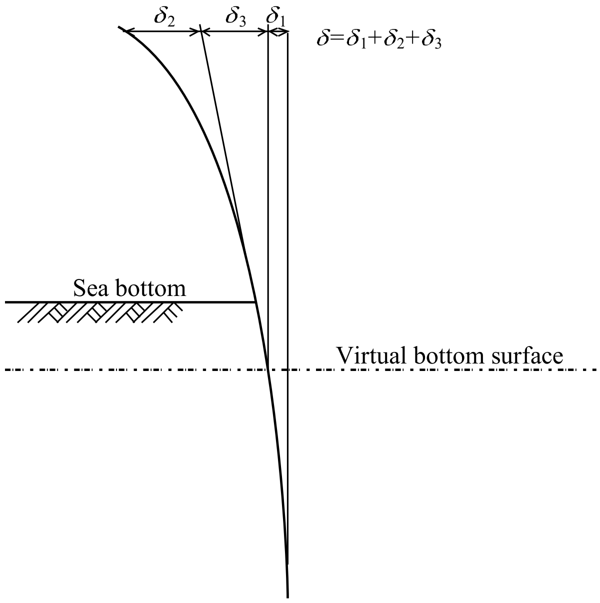


Fig. 2.4.5 Displacement Amount of the Tops of a Sheet Pile Wall

③ The deflection amount of the top of a sheet pile wall is the displacement from a state with no load applied to the sheet pile wall. Therefore, the displacement amount of the top of a sheet pile due to the surcharge after quaywall completion or the earth pressure during the action of earthquake ground motions is preferably calculated in a manner that applies such a surcharge or earth pressure to a sheet pile wall subjected to residual deflection.

④ In the calculation of the deflection amount δ_2 with a sheet pile wall assumed as a cantilever beam, the earth pressure acting on the beam can be simplified as a triangular distribution load, with the resultant earth pressure equivalent to that of the earth pressure illustrated in **Fig. 2.4.6** for convenience.

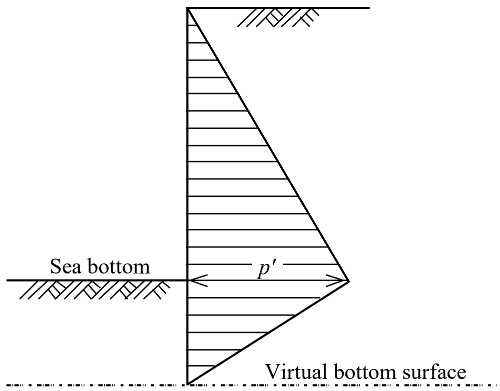


Fig. 2.4.6 Assumption of Earth Pressure

⑤ The deformation amounts of the tops of cantilevered sheet pile quaywalls shall be kept to a level that does not interfere with the use of the quaywalls. According to the past design examples, the deformation amounts of the tops of cantilevered sheet pile quaywalls have often been set at approximately 5 cm under the permanent action situation.

(4) Performance Verification of Copings

The performance verification of the superstructures of cantilevered sheet pile quaywalls can refer to **Part III, Chapter 5, 2.3.11 Performance Verification of Copings**.

(5) Examination of the Actions during Construction

The performance verification of cantilevered sheet pile quaywalls shall take into consideration the stability of the quaywalls against the actions during construction. During construction, cantilevered sheet piles are vulnerable to landward actions because there is no backfill to resist such actions. Therefore, when planning to construct cantilevered sheet pile quaywall by driving sheet pile walls into seabed ground, it is necessary to ensure that they have structures that are capable of sufficiently resisting actions, such as high waves, which are expected to occur during construction periods and may place the sheet pile walls into unstable states.

2.4.4 Performance Verification of Structural Members

For the performance verification of the structural members of cantilevered sheet pile quaywalls, refer to **Part III, Chapter 5, 2.3.12 Structural Details**.

2.5 Sheet Pile Quaywalls with Raking Pile Anchorages

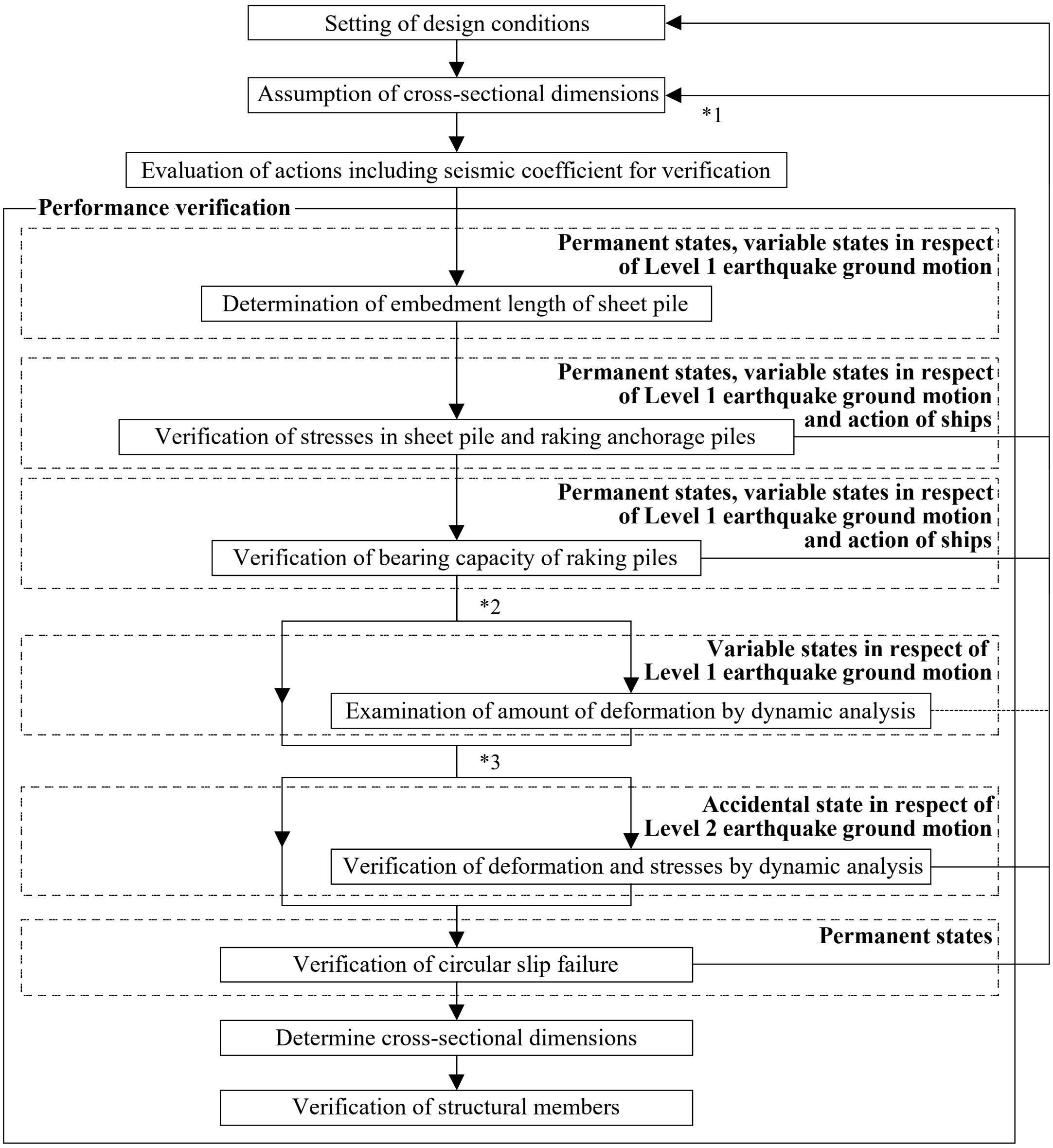
2.5.1 General

(1) The following descriptions are applicable to the performance verification of mooring facilities in which raking piles are driven behind the sheet pile walls and in which the tops of the sheet pile walls and the raking piles are connected to support the soil behind the sheet pile walls.

(2) **Fig. 2.5.1** shows an example of the sequence of the performance verification of sheet pile quaywall with raking pile anchorages. However, **Fig. 2.5.1** does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions. Thus, regarding liquefaction, the possibility of and countermeasures against liquefaction shall be appropriately deliberated with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the performance of sheet pile walls with raking pile anchorages under a variable action situation with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method based on the static equation of equilibrium. However, for high-earthquake-resistance facilities, the deformation should be deliberated by nonlinear seismic response analysis or other methods that take into consideration the dynamic interaction between the ground and structures. For sheet pile quaywall with raking pile anchorages other than high-earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.

(3) Considering that the sheet piles of sheet pile quaywall with raking pile anchorages are subjected to severely corrosive environments, the sheet piles shall be designed with appropriate corrosion protection measures on the basis of **Part III, Chapter 2, 1.3.4 Examination of Performance Deterioration with Age**.

(4) **Fig. 2.5.2** shows an example of a cross section of a sheet pile quaywall with a raking pile anchorage.



*1: As the effects of liquefaction are not shown, it is necessary to consider these separately.
*2: When necessary, an examination of the amount of deformation by dynamic analysis can be carried out for the Level 1 earthquake ground motion.
For high earthquake-resistance facilities, it is preferable that the examination of the amount of deformation be carried out by dynamic analysis.
*3: Verification in respect of Level 2 earthquake ground motion is carried out for high earthquake-presistance facilities.

Fig. 2.5.1 Example of the Sequence of the Performance Verification of Sheet Pile Quaywall with Raking Pile Anchorages

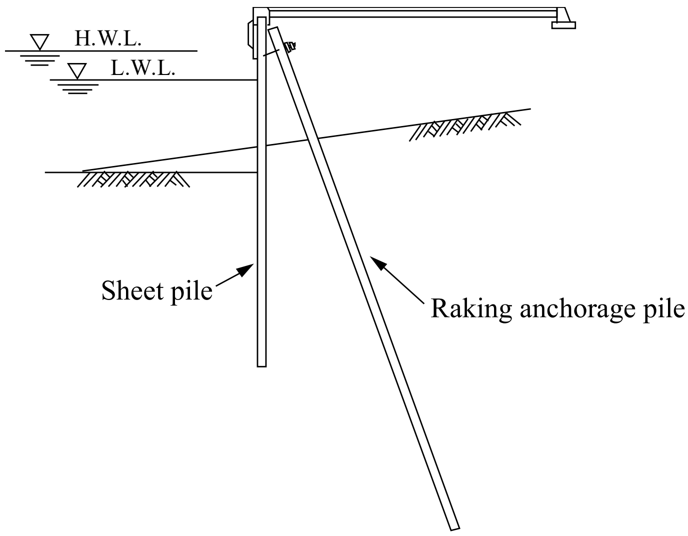


Fig. 2.5.2 Example of the Cross Section of a Sheet Pile Quaywall with Raking Pile Anchorage2.5.2 Actions

2.5.2 Actions

(1) Types of Actions to Be Considered in Respective Design Situations

For the types of actions to be considered in the stability verification of a sheet pile quaywall with raking pile anchorages, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(2) Points of Caution When Setting Actions

① The characteristic values of the seismic coefficients for verification to be used in the performance verification of sheet pile quaywalls with raking pile anchorages under the variable action situation with respect to Level 1 earthquake ground motions shall be appropriately calculated by taking into consideration the structural characteristics of the quaywalls. Here, the characteristic values of the seismic coefficient can be calculated for convenience in accordance with the equation for calculating the seismic coefficient for verification with respect to sheet pile quaywalls with vertical pile anchorages (i.e., **equation 2.3.2 (b)**) in **Part III, Chapter 5, 2.3.4 Actions**.

2.5.3 Performance Verification

(1) It is preferable that the performance verification of sheet pile quaywalls with raking pile anchorages is performed by accurate methods (such as model experiments or reliable numerical analysis methods); however, the performance verification method described in this section can be used as a simplified performance verification method.

(2) The performance verification methods proposed for sheet pile walls with raking pile anchorages include Ishiguro's formula[60] and Oshima's formula,[61] which are classified as simplified performance verification methods. The performance verification of quaywalls during the actions of earthquake ground motions is preferably performed by detailed methods, such as dynamic analyses.

(3) In Ishiguro's formula, the flexural moment and axial force are theoretically calculated on the assumptions that the distance between a sheet pile wall and a raking anchor pile does not change and that the embedded sections of the sheet pile wall and the raking anchor pile are beams placed on elastic supports. In Oshima's formula, the maximum flexural moment and axial force are calculated on the assumptions that earth pressure is equally distributed between a sheet pile wall and a raking anchor pile and that the sheet pile and the ranking anchor pile are beams with respective first fixed points defined as anchorage points. A published report compared the calculation results by using two formulas with the actual stress and earth pressure acting on existing sheet pile walls and raking anchor piles measured on their completion.[62] The report indicates that the calculated stress in sheet pile walls and raking anchor piles correspond relatively well to the measured stress. However, the distribution ratio of earth pressure still needs to be reconsidered because the measured earth pressure acting on sheet piles is remarkably lower than that acting on the raking anchor piles.

(4) Verification of the Stress in Sheet Piles and Raking Anchor Piles

① For sheet pile quaywall with raking pile anchorages, verification may be performed for the resistance of the sheet pile and the piles against the actions in the horizontal and vertical directions at the connection point, earth pressure, and residual water pressure.

② The horizontal and vertical forces acting on the connection point between a sheet pile and a raking pile can be calculated by assuming that the connection is a pin structure.

(5) Determination of the Embedded Lengths of Sheet Piles and Raking Anchor Piles

The embedded length of the sheet pile or raking anchorage pile that is required to resist the forces acting in the axial direction and the direction perpendicular to the axis can be calculated in accordance with **Part III, Chapter 2, 3.4 Pile Foundations**. However, the bearing capacity in the axial direction of the sheet pile and that of the raking anchorage pile is preferably examined via loading and pulling tests.

2.5.4 Performance Verification of Structural Members

The performance verification of sheet pile quaywalls with raking pile anchorages can be performed in accordance with the performance verification of sheet pile quaywalls and open-type wharves on vertical piles with reference to **Part III, Chapter 5, 2.3.12 Structural Details** and **Part III, Chapter 5, 5.2.6 Performance Verification of Structural Members**.

2.6 Open-type Quaywall with Sheet Pile Wall Anchored by Forward Batter Piles

2.6.1 General

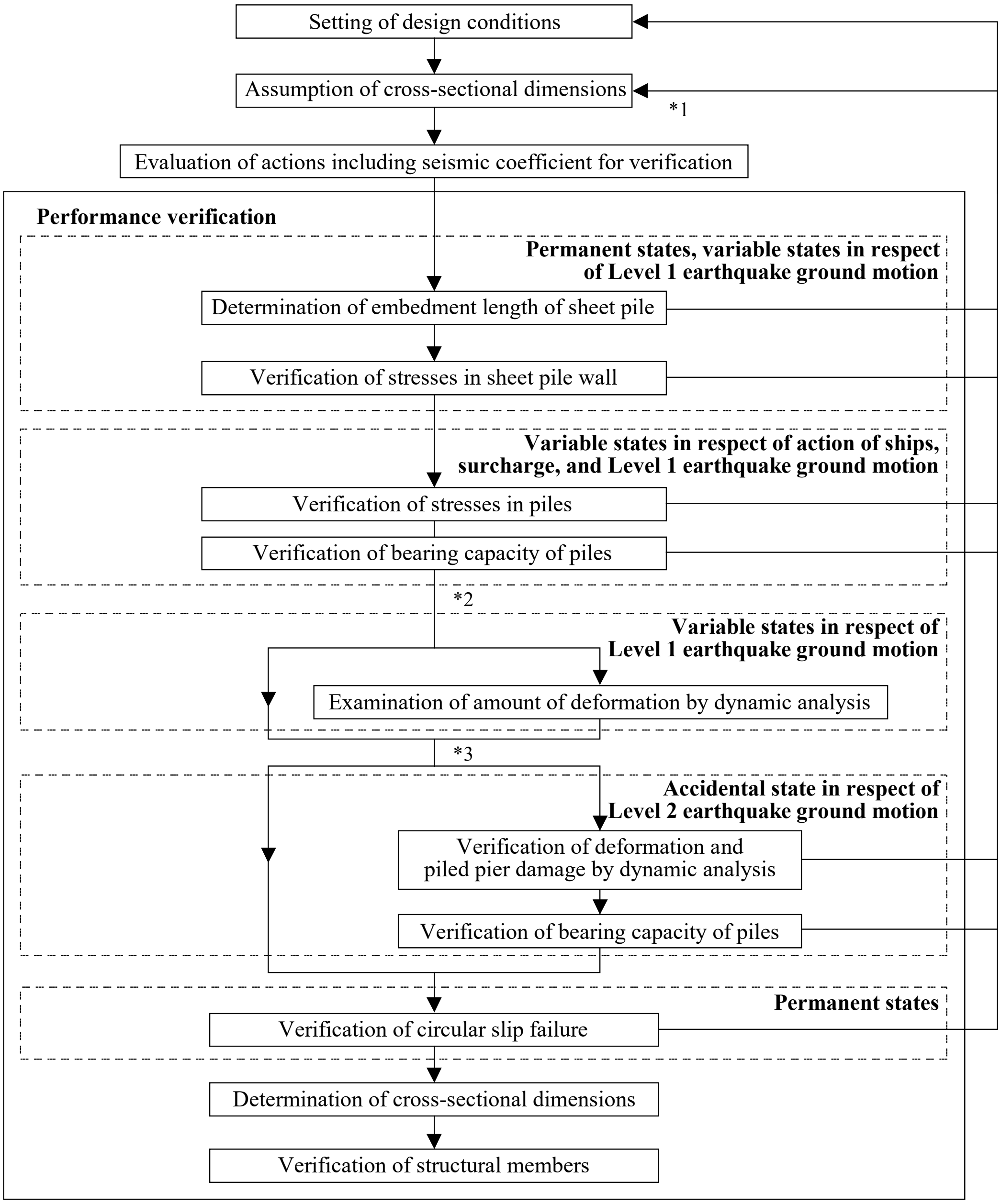
(1) The provisions in this section shall be applied to the performance verification of mooring facilities that retain the earth pressure behind them by sheet pile walls and raking anchor piles driven in front of the sheet pile walls with respective top sections coupled together.

(2) An open-type quaywall with sheet pile wall anchored by forward batter piles is normally constructed with open-type wharves in front of the sheet pile walls. The open-type wharf may be integrated into or separated from the sheet pile walls. This section provides guidelines for cases in which open-type wharves and sheet pile walls are integrated. For cases in which open-type wharves are separated from sheet pile walls, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**; **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**; and **Part III, Chapter 5, 5.3 Open-type Wharves on Coupled Raking Piles**. The performance verification method described in this section is based on the equivalent beam method applied to the performance verification of sheet piles. Therefore, the structural types covered by this section are steel sheet pile walls driven into sandy soil ground or hard clayed soil ground.

(3) **Fig. 2.6.1** shows an example of the sequence of the performance verification of an open-type quaywall with sheet pile wall anchored by forward batter piles. However, **Fig. 2.6.1** does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions. Thus, regarding liquefaction, the possibility of and countermeasures against liquefaction shall be appropriately deliberated with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the performance of open-type quaywall with sheet pile wall anchored by forward batter piles under a variable action situation with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method based on the static equation of equilibrium. However, for high earthquake-resistance facilities, the performance verification should be performed by nonlinear seismic response analysis or other methods that take into consideration the 3D dynamic interaction between piles and the ground. For open-type quaywall with sheet pile wall anchored by forward batter piles other than high earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.

(4) An open-type quaywall with sheet pile wall anchored by forward batter piles is subjected to severe environments and have structures that are susceptible to the performance reductions of members due to material degradation such as chloride-induced corrosion to concrete members and corrosions of steel pipe piles. Therefore, open-type quaywalls with sheet pile wall anchored by forward batter piles can be designed in accordance with open-type wharves on vertical piles with reference to **Part III, Chapter 5, 5.2.1 General**.

(5) **Fig. 2.6.2** shows an example of the cross section of an open-type quaywall with a sheet pile wall anchored by forward batter piles.



*1: The evaluation of the effect of liquefaction is not shown because it is necessary to consider these separately.

*2: When necessary, an examination of the amount of deformation by dynamic analysis can be performed for the Level 1 earthquake ground motion.
For high earthquake-resistance facilities, the examination of the amount of deformation should be performed by dynamic analysis.

*3: Verification with respect to Level 2 earthquake ground motion is performed for high earthquake-resistance facilities.

Fig. 2.6.1 Example of the Sequence of the Performance Verification of Open-type Quaywall with Sheet Pile Wall Anchored by Forward Batter Piles

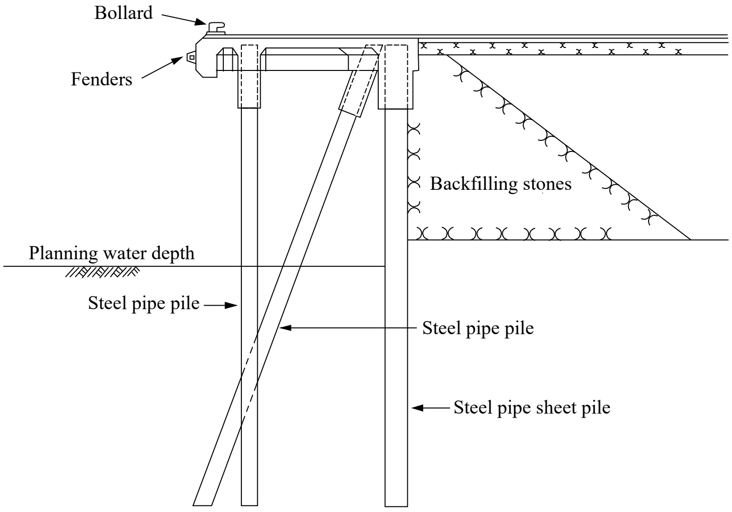


Fig. 2.6.2 Example of Cross-section of Open-type Quaywall with Sheet Pile Wall Anchored by Forward Batter Piles

2.6.2 Layouts and Dimensions

(1) For the size of one block of a superstructure and the pile layout of an open-type wharf, refer to the size of open-type wharf block and pile layouts in **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

(2) It is preferable that the layouts and inclinations of batter piles are determined with due consideration given to their positional relationship with other piles and construction work-related constraints such as those concerning the capacity of pile driving equipment. A pile inclination of approximately 20° is normally used for batter piles.

(3) For the dimensions of the superstructures, refer to the dimensions of superstructures in **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

2.6.3 Actions

(1) For the actions on open-type wharves, refer to **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

(2) For the actions of the sheet piles, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(3) The self-weight of the reinforced concrete of the superstructures of open-type wharves can be calculated with a unit weight of 21 kN/m^2 in the performance verification of piles and sheet piles in accordance with **Part III, Chapter 5, 5.3 Open-type Wharves on Coupled Raking Piles**.

(4) The fender reaction force can be calculated using the calculation methods described in **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

(5) The characteristic values of the seismic coefficients for verification used in the performance verification of an open-type quaywall with a sheet pile wall anchored by forward batter piles for the variable action situation with respect to Level 1 earthquake ground motions shall be appropriately calculated by taking the structural characteristics into consideration. For convenience, the characteristic values of the seismic coefficients for verification used in the performance verification of open-type quaywalls with sheet pile wall anchored by forward batter piles may be calculated using **equation (2.3.2 (a))** in **Part III, Chapter 5, 2.3.4 (2) Points of Caution When Setting Actions** for sheet pile walls and in accordance with **Part III, Chapter 5, 5.2 Open-type Wharf on Vertical Piles** for open-types wharves.[63)]

2.6.4 Performance Verification

(1) The performance verification of open-type quaywalls with sheet pile wall anchored by forward batter piles should be performed by using accurate methods (such as model experiments or reliable numerical analysis methods).[63), 64)] Given that an open-type quaywall with sheet pile wall anchored by forward batter piles has a structure that allows sheet pile walls to retain earth pressure behind them, the sheet pile walls are subjected to deformation during the action of earthquake ground motions. Therefore, the performance verification methods should be able to

appropriately evaluate the effects of the deformation. The following methods are simplified ones that are applicable to be the performance verification of an open-type quaywall with sheet pile wall anchored by forward batter piles.

(2) Performance Verification of Sheet Piles and Other Types of Piles

① The performance verification of sheet pile walls may be performed by assuming the connection points between batter piles and sheet piles as fulcrums in accordance with the performance verification in **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

② On the basis of the reaction force at the connection points between batter piles and sheet piles as the horizontal force acting on the superstructures of open-type wharves, the axial force in sheet piles and piles can be calculated in accordance with the performance verification of open-type wharves on coupled raking piles.

(3) Performance Verification of Open-type Wharves

① For the performance verification of open-type wharves, refer to **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-type Wharves on Coupled Raking Piles**.

② For the assumptions regarding the seabed, refer to **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**. The estimation of the behavior of piles, including the lateral resistance capacity of piles, may be performed using Chang's method.

③ Generally, the vertical loads distributed to respective pile heads can be calculated as the fulcrum reaction force under the assumption that the superstructures of open-type wharves are simple beams supported at the positions of pile heads. The axial force on raking piles and sheet piles may be calculated using the horizontal force on open-type wharves and the vertical loads distributed to pile heads according to **equation (3.4.52)** in **Part III, Chapter 2, 3.4.9 Bearing Capacity of Coupled Piles**. As a compression force acting on vertical, the vertical loads distributed to pile heads can be used.

④ Generally, the flexural moment at the connection points between batter piles and sheet piles may be calculated as the flexural moment acting on a rigid frame fixed at the virtual fixed points of the batter and sheet piles subjected to the earth pressure, residual water pressure, dynamic water pressure during the actions of earthquake ground motions, and other horizontal forces.

⑤ The performance verification of open-type wharves shall be performed with due consideration to the rotation of open-type wharf blocks as needed.

(4) The examinations of the embedded lengths of piles with respect to axial force and lateral resistance can be made in accordance with **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

2.6.5 Performance Verification of Structural Members

(1) The performance verification of structural members on sheet pile walls anchored by foreword batter piles can be made by referring to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls** and **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

(2) The connecting points between sheet pile walls and batter piles shall have structures that are capable of fulfilling their full function to transmit loads.

(3) The superstructures of open-type wharves shall have structures that are capable of fully withstanding the flexural moment transmitted from the sheet pile walls.

(4) Considering that damage to the connecting points could lead to the collapse of the entire quaywalls, the connecting points between sheet pile walls and batter piles must have sufficient reinforcement. The flexural moment generated in the heads of the sheet piles is transmitted to the superstructures of open-type wharves. Therefore, the flexural moment needs to be taken into consideration in the performance verification of the superstructures of open-type wharves.

2.7 Double Sheet Pile Quaywalls

[Public Notice] (Performance Criteria for Double Sheet Pile Quaywalls)

Article 50

3 In addition to the provisions in the paragraph (1), the performance criteria for double sheet pile structures shall be as prescribed respectively in the subsequent items:

(1) The risk of occurrence of sliding of the structural body shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(2) The risk that the deformation of the top of the front or rear sheet piles may exceed the allowable limit of deformation shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(3) The risk of losing the stability due to the shear deformation of the structural body shall be equal to or less than the threshold level under the permanent situation in which the dominating action is earth pressure.

[Interpretation]

11. Mooring Facilities

(4) Performance Criteria of Sheet Pile Quaywall

③ **Double sheet pile quaywall** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance on Criteria and the interpretation related to Article 50, Paragraph 3 of the Public Notice)

(a) The performance criteria and commentaries for a double sheet pile quaywall shall conform to those for sheet pile quaywall and the following provisions:

(b) The required performance of a double sheet pile quaywall under the permanent action situation in which the dominant actions are self-weight and earth pressure and the variable action situation in which the dominant action is Level 1 earthquake ground motions shall focus on serviceability. **Attached Table 11-12** shows the performance verification items and standard indexes used for determining the limit values with respect to the actions.

Attached Table 11-12 Performance Verification Items and Standard Indexes Used for Determining the Limit Values under the Respective Design Situations of a Double Sheet Pile Quaywall

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non dominating action		
26	1	2	50	3	1	Serviceability	Permanent	Earth pressure	Self-weight, water pressure, surcharge	Sliding of wall body	Action–resistance ratio with respect to the sliding of the wall body
							Variable	L1 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge		
			50	1	4		Permanent	Self-weight	Water pressure, surcharge	Circular slip failure of ground	Action–resistance ratio with respect to circular slip failure

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non dominating action		
			50	3	2		Permanent	Earth pressure	Self-weight, water pressure, surcharge	Deformation of the crown of the front and back sheet piles	Residual deformation amount of the crown of the quaywall
							Variable	L1 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge		
					3		Permanent	Earth pressure	Water pressure, surcharge	Shear deformation of wall body	Action–resistance ratio with respect to shear deformation of wall body

(c) In addition to the above information, the requirements and the commentaries in Paragraph 3 (Scouring and Wash Out), Article 22 and Article 28 (Performance Criteria of Armor Stones and Blocks) of the Public Notice shall be applied as needed.

2.7.1 General

(1) The following is applicable to the performance verification of mooring facilities that use double sheet pile structures.

(2) A double sheet pile quaywall is a mooring facility that has an earth-retaining wall structure constructed in a manner that drives two rows of sheet pile walls, connects the two walls through tie members or similar materials, and fills the space between the two walls with soil.

(3) **Fig. 2.7.1** shows an example of the cross section of a double sheet pile quaywall.

(4) **Fig. 2.7.2** shows an example of the sequence of the performance verification of a double sheet pile quaywall. There may be another case of the performance verification of the shear deformation of double sheet pile walls based on virtual cross sections that assume sheet pile wall structures have sheet pile anchorages with the embedded lengths of sheet piles and the distance between two sheet piles equivalent to those of double sheet pile walls. However, **Fig. 2.7.2** does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions. Therefore, regarding liquefaction, the possibility of and countermeasures against liquefaction shall be appropriately deliberated with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the performance of double sheet pile walls under a variable action situation with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method based on the static equation of equilibrium. However, for high earthquake-resistance facilities, the deformation is preferably deliberated by nonlinear seismic response analysis or other methods by taking into consideration the dynamic interaction between the ground and structures. For double sheet pile quaywalls other than high earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.

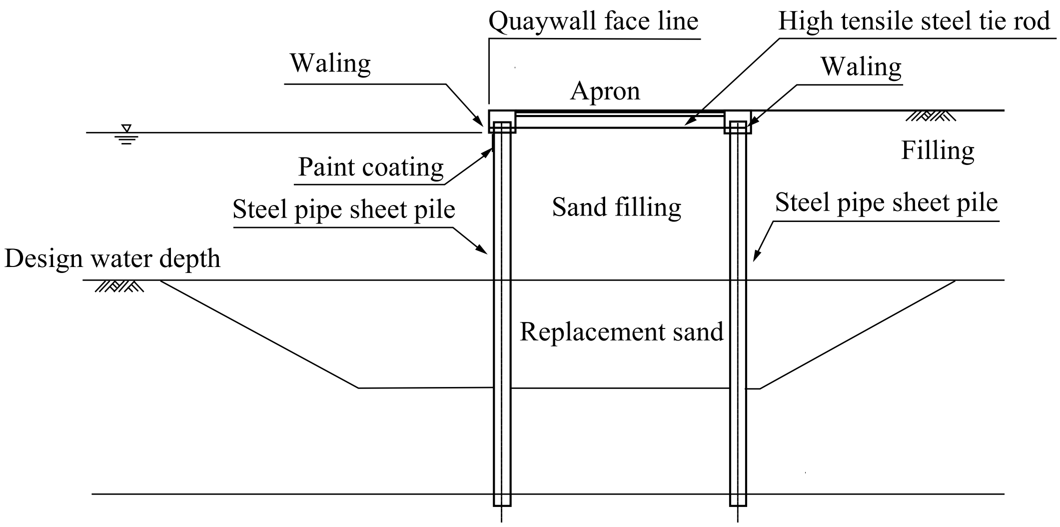
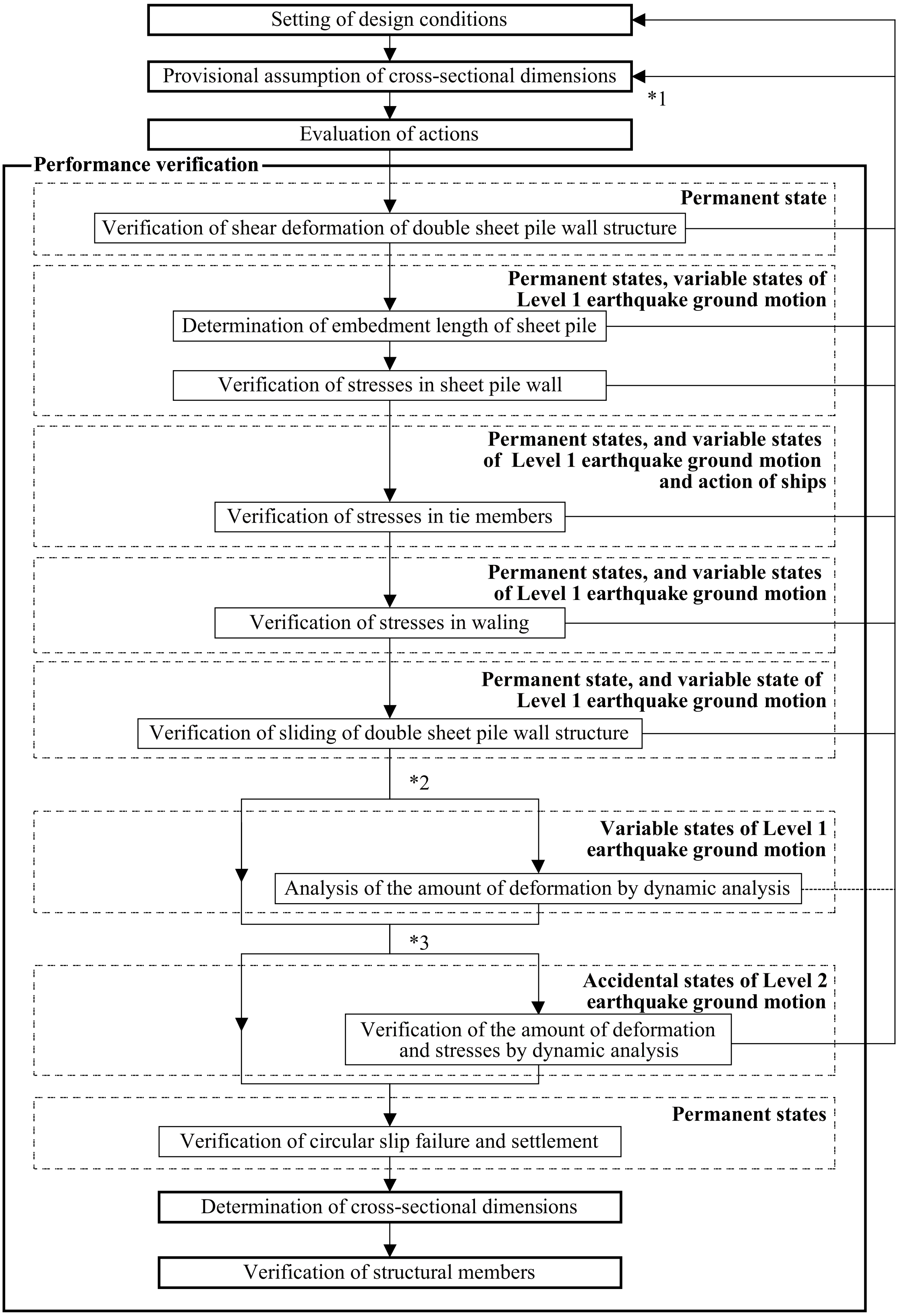


Fig. 2.7.1 Example of the Cross Section of a Double Sheet Pile Quaywall



*1: The evaluation of the effect of liquefaction is not shown because this must be separately considered.
*2: Analysis of the amount of deformation due to Level 1 earthquake ground motion may be carried out by dynamic analysis when necessary.
For high earthquake-resistance facilities, analysis of the amount of deformation by dynamic analysis is desirable.
*3: For high earthquake-resistance facilities, verification is carried out for Level 2 earthquake ground motion.

Fig. 2.7.2 Example of the Sequence of the Performance Verification of Double Sheet Pile Quaywalls

(5) The performance verification of double sheet pile quaywalls has conventionally been performed in accordance with the performance verification methods for steel pile cellular-bulkhead quaywalls or sheet pile quaywalls with sheet pile anchorages. Therefore, the performance verification methods described in this section can be applied to double sheet pile quaywall under conditions that are similar to those frequently applied to existing quaywalls.

(6) The performance verification of deformation is important when applying double sheet pile structures to large-scale permanent structures. The methods for evaluating the deformation of double sheet pile structures include Sawaguchi's method[65] and Ohori's method,[66] which was established by modifying Sawaguchi's method so that the deformation of double sheet pile walls can be comprehensively evaluated. Considering that these methods are simplified ones, it is preferable that detailed methods, such as dynamic analyses, are used for the performance verification of the deformation during the actions of earthquake ground motions.

(7) Similar to the case with a cellular-bulkhead structure, a double sheet pile structure can ensure structural stability after the completion of filling work but has a risk of collapsing when hit by small waves during construction with no materials filled in between two sheet piles. Therefore, filling work is preferably implemented as soon as sheet piles are driven. To facilitate the early implementation of filling work, it is common practice to install supplemental sheet piles that function as bulkheads between the rows of sheet piles at the intervals depending on the wave height, the types of infill materials to be used, and the construction site conditions. It is also common practice to use tie members in combination with rigid beams to brace sheet piles during construction.

(8) In constructing a double sheet pile quaywall, the installation of a double sheet pile wall (two rows of sheet piles with filling sand placed in between) is generally implemented before backfilling work. Therefore, sheet piles with identical shapes and dimensions are normally used for both rows.

(9) When used for purposes other than mooring facilities such as enclosing bunds, breakwaters, or revetments, the performance of double sheet pile structures shall be verified by appropriate methods. For example, the performance verification of those used as temporary enclosure bunds or earth-retaining walls shall be performed for the embedded lengths followed by seepage control effects (seepage path lengths) and the heaving and piping prevention. Furthermore, the performance verification of double sheet pile walls can be performed with reference to **references 67)** and **68)**.

(10) Considering that the sheet piles of double sheet pile quaywall are subjected to severely corrosive environments, the sheet piles shall be designed with appropriate corrosion protection measures on the basis of **Part III, Chapter 2, 1.3.4 Examination of Performance Deterioration with Age**.

2.7.2 Actions

(1) For the actions on double sheet pile quaywalls, refer to **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections**.

(2) **Equation (2.7.1)** can be used for calculating the characteristic value of the seismic coefficient to be used in the performance verification of double sheet pile quaywalls under the variable action situation with respect to Level 1 earthquake ground motions.[58] In this equation, the subscript k indicates the characteristic value. For the filter and reduction coefficient required when reflecting the frequency characteristics into the calculation of the maximum acceleration α_c in the equation, refer to **Reference (Part III), Chapter 1, 1 Detail Items about Seismic Coefficient for Verification**.

$$k_{h_k} = 1.91\left(\frac{D_a}{D_r}\right)^{-0.69} \cdot \frac{\alpha_c}{g} + 0.03 \qquad \textbf{(2.7.1)}$$

where

k_h : seismic coefficient for verification;

D_a : allowable deformation amount (15 cm);

D_r : reference deformation amount (10 cm);

g : gravitational acceleration (980 cm/s^2).

2.7.3 Performance Verification

(1) The examination to determine the distance between two sheet pile walls to achieve the required strength against shear deformation can be made in accordance with **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections**.

(2) The calculation of the deformation moment can be made in accordance with **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections**.

(3) The calculation of the resistance moment can be made in accordance with **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections** provided that the moment due to the tension force between the joints on bulkhead sheet piles is not generally considered to contribute to the resistance moment.

(4) The embedded length of sheet piles is generally the following value, whichever is larger: the value calculated by the method for sheet piles with an anchorage (the examination of the embedded lengths of sheet piles in **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**) or the value satisfying the allowable horizontal displacement of the top of cellular bulkhead (the examination of the stability of a wall body as a whole and the examination of displacement of the top of cellular bulkhead in **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections**). However, for important facilities, the performance verification is preferably performed by accurate methods (such as model experiments or numerical analyses capable of reproducing mechanisms). When using numerical analyses, refer to **Reference (Part III), Chapter 1, 2 Basic Items for Earthquake Response Analyses**.

(5) The flexural stress in sheet piles of double sheet pile quaywalls is considered to be caused by the actions of waves on the series of sheet piles during construction without any support, the earth pressure after the completion of filling work, the earth pressure of the soil reclaimed behind double sheet pile walls, and seaward water pressure as a result of the lowering of water levels in front of double sheet pile walls. The cross sections of sheet piles shall be determined by using the most severe flexural stress after examining all the cases described above.

(6) For the calculation of the tension force in tie members, refer to the tension force in tie members in **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(7) For the performance verification of waling, refer to the verification of waling in **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(8) A double sheet pile quaywall can be considered a kind of gravity-type quaywall. Therefore, it is necessary to verify the stability of quaywalls against the sliding and the circular slip failure including a wall body as is the case with a cellular-bulkhead type quaywall. The performance verification of double sheet pile quaywalls can be performed with reference to the performance verification described in **Part III, Chapter 5, 2.2 Gravity-type Quaywalls**. The examination of sliding failure shall generally be performed on both failure surfaces: one is a seafloor surface that is virtually set as the bottom face of a sheet pile wall, and the other is a surface that passes through the toe of the sheet pile wall. In the former case, the resistance of the sheet pile wall below the virtual seafloor surface should be ignored. In the examination of the overall slope stability, including the double sheet pile wall with the embedded length of the double sheet pile wall equal to or longer than the required embedded length calculated for a sheet pile quaywall with an anchorage, the portion of a sheet piles below the required embedded length shall not be considered to contribute to resisting circular slip failures that have failure surfaces passing below the required embedded length.

(9) For the performance verification of the slabs and upright sections of a superstructure, refer to the performance verification of the relieving platforms in **Part III, Chapter 5, 2.8 Quaywalls with Relieving Platforms**. Foundation piles are driven into the filling material to support the superstructure according to the circumstances. These piles should have sufficient safety against the horizontal and vertical force transmitted from the superstructure. Here, it is assumed that the vertical force transmitted from the superstructure is entirely borne by the piles, and the vertical bearing capacity of the piles is calculated by ignoring the skin friction between the piles and filling material. The horizontal force that acts on the superstructure is transmitted to a double sheet pile wall partly through the piles and partly through the sheet piles. Therefore, the appropriate shares of the horizontal force between the piles and sheet piles should be determined.

2.8 Quaywalls with Relieving Platforms

[Public Notice] (Performance Criteria of Quaywalls with Relieving Platforms)

Article 51

The performance criteria of quaywalls with relieving platforms shall be as prescribed respectively in the following items:

(1) Sheet piles shall have the embedment length as necessary for the structural stability and shall contain the degree of risk that the stresses in the sheet piles may exceed the yield stress at a level equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(2) The risk of occurrence of sliding or overturning of the wall body shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(3) The following criteria shall be satisfied under the permanent situation in which the dominating action is self-weight:

(a) The risk that the axial forces acting in the relieving platform piles may exceed the resistance force based on the failure of the ground shall be equal to or less than the threshold level.

(b) The risk of impairing the integrity of the members of the relieving platform shall be equal to or less than the threshold level.

(4) The following criteria shall be satisfied under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating actions are Level 1 earthquake ground motions, ship berthing, and traction by ships:

(a) The risk that the axial forces acting on the relieving platform piles may exceed the resistance force based on the failure of the ground shall be equal to or less than the threshold level.

(b) The risk that the stress acting on the relieving platform piles may exceed the yield stress shall be equal to or less than the threshold level.

(c) The risk of impairing the integrity of the members of the relieving platform shall be equal to or less than the threshold level.

(5) The risk of occurrence of a slip failure in the ground that passes below the bottom end of the sheet piling shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is self-weight.

[Interpretation]

11. Mooring Facilities

(5) Performance Criteria of Quaywalls with Relieving Platforms (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance and the Interpretation related to Article 51of the Public Notice,)

① Serviceability shall be the required performance for quaywalls with relieving platforms under the permanent situation in which the dominating action is self-weight or earth pressure and under the variable situation in which the dominating actions are Level 1 earthquake ground motions, ship berthing, and traction by ships. Performance verification items for those actions and standard indexes for setting limit values shall be in accordance with **Attached Tables 11-13** and **11-14**. In **Attached Table 11-13**, the wall body is equivalent to the wall body in the case of a gravity-type quaywall in the verification of the stability of the structure of quaywalls with relieving platforms.

Attached Table 11-13 Performance Verification Items and Standard Indexes for Setting Limit Values for Sheet Piles and Structural Stability of Quaywalls with Relieving Platforms under Different Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	51	–	1	Serviceability	Permanent	Earth pressure	Water pressure, surcharges	Necessary embedment length	Embedment length necessary for structural stability
										Yielding of sheet pile	Design yield stress of sheet pile
							Variable	Level 1earthquake ground motion	Earth pressure, water pressure, surcharges	Necessary embedment length	Embedment length necessary for structural stability
										Yielding of sheet pile	Design yield stress of sheet pile
					2		Permanent	Earth pressure	Self-weight, water pressure, surcharge	Sliding/overturning of wall body	Action-to-resistance ratio for sliding and overturning of wall body
							Variable	Level 1earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Sliding/overturning of wall body	Action-to-resistance ratio for sliding and overturning of wall body
					5		Permanent	Self-weight	Water pressure, surcharge	Circular slip failure of ground	Action-to-resistance ratio for circular slip failure

Attached Table 11-14 Performance Verification Items and Standard Indexes for Setting Limit Values for Relieving Platforms and Relieving Platform Piles of Quaywalls with Relieving Platforms under Different Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	51	–	3 (a)	Serviceability	Permanent	Self-weight	Surcharging, water pressure	Axial forces on relieving platform piles	Action-to-resistance ratio for bearing capacity of anchorage work (pushing, pulling)
					3 (b)				Earth pressure, water pressure, surcharge	Sectional stress of cross-section of relieving platform	Bending compressive stress
					4 (a)		Permanent	Earth pressure	Self-weight, water pressure, surcharge	Axial forces on relieving platform piles	Action-to-resistance ratio for bearing capacity of piles (pushing, pulling)
							Variable	Level 1earthquake ground motion Traction of ships	Self-weight, earth pressure, water pressure, surcharge		

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
					4 (b)		Permanent	Earth pressure	Water pressure, surcharge	Yielding of relieving platform piles	Design yield stress
							Variable	Level 1earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge		
								Traction of ships			
					4 (c)		Permanent	Earth pressure	Water pressure, surcharge	Sectional stress of cross-section of relieving platform	Bending compressive stress
							Variable	Level 1earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Failure of cross-section of relieving platform	Design cross-sectional resistance force
								Ship berthing, traction of ships			

② In addition to these provisions, provisions regarding the **Public Notice**, **Article 22**, **Paragraph 3** (Scouring and Sand Washing Out), and **Article 28** (Performance Criteria for Armor Stones and Blocks) and their interpretations shall apply, as needed.

2.8.1 General

(1) The provisions in this section may be applied to the performance verification of a quaywall with a relieving platform comprising a relieving platform, a sheet pile wall in front of the relieving platform, and relieving platform piles.

(2) A quaywall with a relieving platform normally comprise a relieving platform, a sheet pile wall in front of the relieving platform, and relieving platform piles. The relieving platform is in many cases constructed as an L-shaped structure of cast-in-place reinforced concrete and the upper part of the platform is usually buried under landfill material, but sometimes a box-shaped platform is used to reduce the weight of the platform and the earthquake forces that act on it (see **Figs. 2.8.1** and **2.8.2**.).

(3) The performance verification of a quaywall with a relieving platform can be conducted separately for the sheet piles, relieving platform, and relieving platform piles. For the sheet piles, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**. For the relieving platform and the relieving platform piles, refer to **Part III,** , **Chapter 2**, **3.4 Pile Foundations**. For sliding and overturning of a quaywall with a relieving platform as a whole, refer to **Part III, Chapter 5**, **2.2 Gravity-type Quaywalls**. For circular slip failure, refer to **Par III, Chapter 2**, **4.2.1 Stability Analysis by Circular Slip Failure Plane**. Analysis of circular slip failure is often omitted for ground of relatively good quality, such as sandy ground.

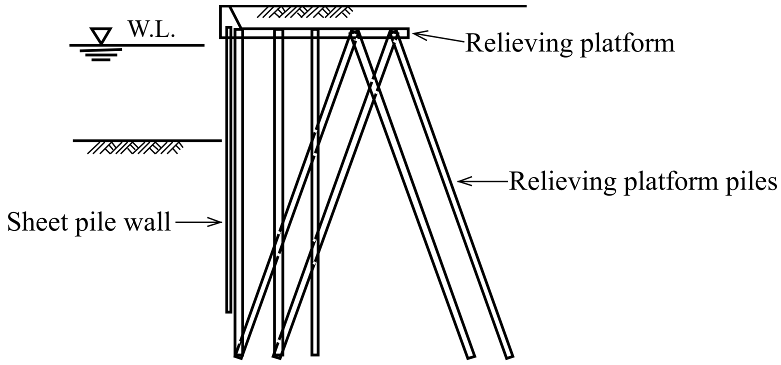


Fig. 2.8.1 Structure of Quaywall with Relieving Platform (L-Shaped Platform)

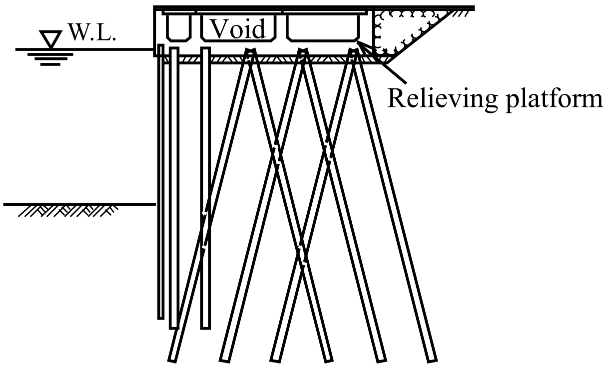


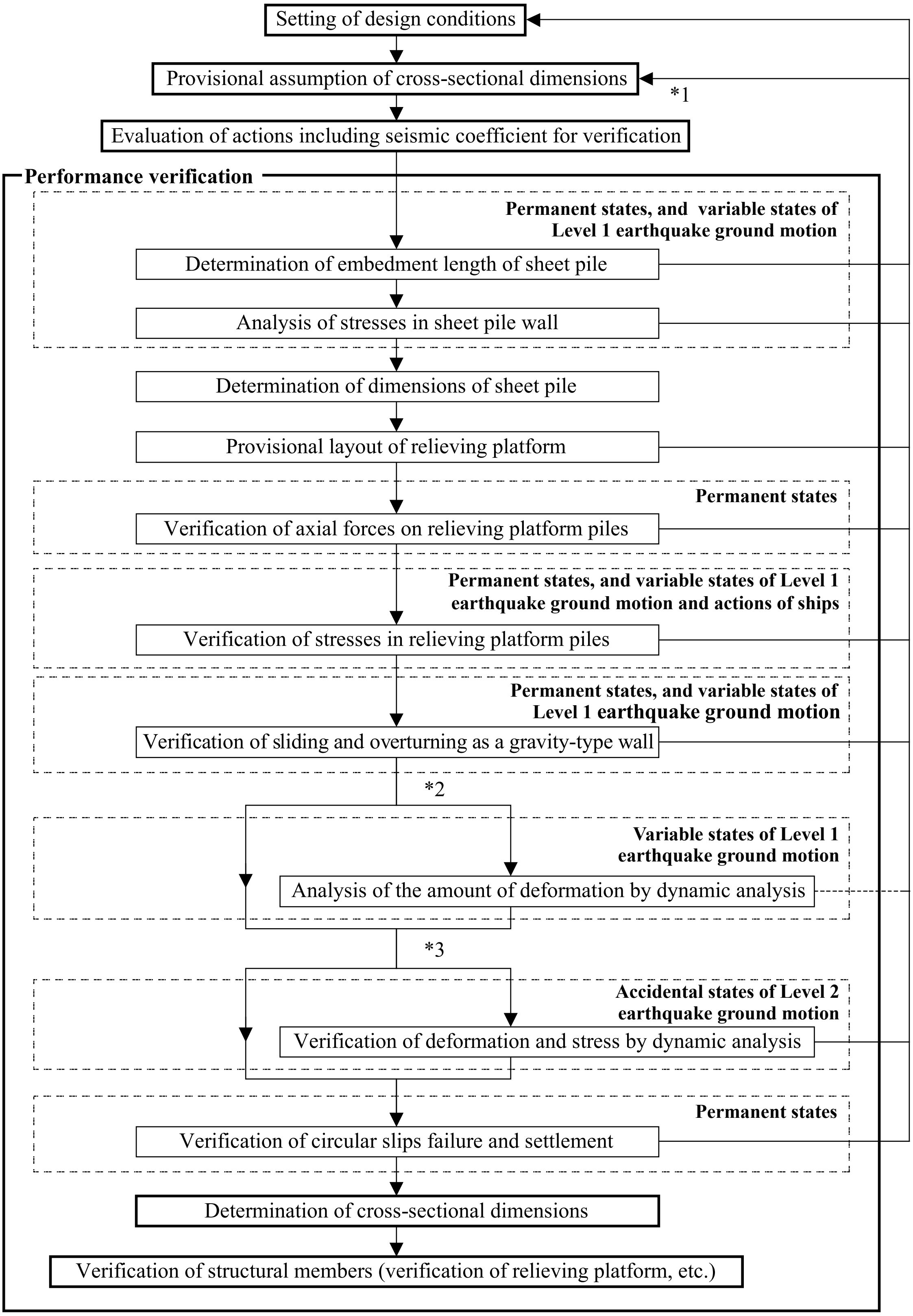
Fig. 2.8.2 Structure of Quaywall with Relieving Platform (Box-Shaped Platform)

(4) The approaches given in **Part III, Chapter 5**, **2.8.3 Actions** and **Part III, Chapter 5**, **2.8.4 Performance Verification** use simplified techniques and thus caution is advised when adopting these approaches. Since quaywalls with relieving platforms have complex structures, highly precise methods (model tests and numerical analysis techniques that can simulate mechanisms) should be used for verification.

(5) An example of the sequence of performance verification of quaywalls with relieving platforms is shown in **Fig. 2.8.3**. Note that the evaluation of the effects of liquefaction, settlement, and other phenomena caused by earthquake ground motions is not shown in **Fig. 2.8.3**. Therefore, for liquefaction, as an example, it is necessary to determine whether there is liquefaction and to consider measures against it appropriately by reference to **Part II, , Chapter 7 Ground Liquefaction**. It is possible to verify the variable situation associated with Level 1 earthquake ground motions using the seismic coefficient method. For high earthquake-resistance facilities, however, it is desirable to analyze the amount of deformation, for example, using the nonlinear seismic response analysis in consideration of dynamic interaction between the ground and structure. For quaywalls with relieving platforms that are not categorized as high earthquake-resistance facilities, verification of the accidental situation associated with Level 2 earthquake ground motions can be omitted.

(6) For a quaywall that has a retaining sheet pile wall on the back of the relieving platform, performance verification of the sheet pile wall and relieving platform can be conducted with reference to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls** and **Part III, Chapter 5**, **5.2 Open-Type Wharves on Vertical Piles**, respectively. In the performance verification of such a quaywall, the earth pressure on the back of the relieving platform and the reaction in the upper part of sheet piling shall be taken into consideration as forces acting on the relieving platform.

(7) Sheet piles of quaywalls with relieving platforms are placed in severely corrosive environments. Therefore, the sheet piles shall be designed with appropriate corrosion protection measures in accordance with **Part III, Chapter 2**, **1.3.4 Examination of Change in Performance Over Time**.



*1: The evaluation of the effect of liquefaction is not shown; therefore, it must be considered separately.
*2: Analysis of the amount of deformation due to Level 1 earthquake ground motion may be carried out by dynamic analysis when necessary.
For high earthquake-resistance facilities, analysis of the amount of deformation due to Level 1 earthquake ground motion should also be carried out by dynamic analysis.
*3: For high earthquake-resistance facilities, verification is conducted for Level 2 earthquake ground motion.

Fig. 2.8.3 Example of the Sequence of Performance Verification of a Quaywall with a Relieving Platform

2.8.2 Setting of Cross-sectional Dimensions

(1) Determining the height and width of a relieving platform

Appropriate installation height and shape of a relieving platform shall be determined by considering conditions of actions, economic efficiency and constructability, and paying particular attention to the following points:

① The earth pressure acting on a sheet pile wall can be reduced by increasing the height of the relieving platform and placing its bottom at a lower level, which makes it possible to design the sheet pile wall with a smaller sectional area and shorter embedment length. At the same time, however, this makes it necessary to increase the quantity and length of reliving platform piles to support the increased weight of the relieving platform and resist a greater force of Level 1 earthquake ground motion acting on it.

② There is a possibility that the ground sinks at the bottom of the relieving platform and causes a gap below it. It is desirable to placing the bottom of the relieving platform at the level equal to or lower than the residual water level because piles may corrode.

③ To reduce the earth pressure acting on the sheet pile wall, it is common to determine the width of the relieving platform so that it intersects the sheet pile active failure plane extending from the seabed. When determining the width of the relieving platform, it is necessary to ensure that the required number of relieving platform piles can be arranged appropriately.

2.8.3 Actions

(1) The earth pressure and residual water pressure acting on the sheet pile wall vary according to the structural characteristics. Therefore, they shall be calculated appropriately in consideration of the height and width of the relieving platform as well as support conditions.

(2) When the relieving platform intersects the sheet pile active failure plane extending from the seabed, the active earth pressure acting on the sheet pile wall can be calculated assuming that the bottom of the relieving platform is the virtual ground plane and no surcharge is on it as shown in Fig. 2.8.4.

(3) Normally, the residual water pressure acting on the sheet pile wall may be considered as the residual water pressure acting on the bottom of the relieving platform and below on the assumption that the water pressure distribution is the same as that of a quaywall with no relieving platform (see **Fig. 2.8.4**.).

(4) As for passive earth pressure in front of the embedded section of a sheet pile wall, refer to **Part III, Chapter 5, 2.3 Sheet Pile quaywalls**.

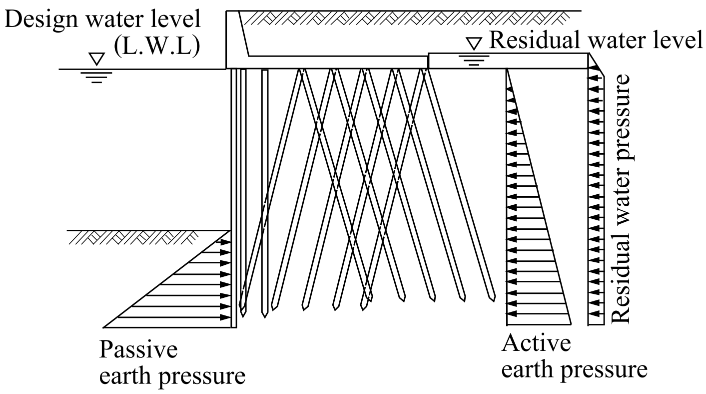


Fig. 2.8.4 Earth Pressure and Residual Water Pressure Acting on a Sheet Pile Wall

(5) The characteristic value of seismic coefficient for verification used in the performance verification of quaywalls with relieving platforms for the variable situations associated with Level 1 earthquake ground motion shall be appropriately calculated taking the structural characteristics into consideration. For convenience, the characteristic value of seismic coefficient for verification of quaywalls with relieving platforms may be calculated in accordance with **Part III, Chapter 5**, **2.2 Gravity-type Quaywalls**.

(6) For calculations of the earth pressure and residual water pressure acting on a sheet pile wall, refer to **Part II, Chapter 4**, **2 Earth Pressure,** and **Part II, Chapter 4**, **3.1 Residual Water Pressure**. In this case, the angle of wall friction of the sheet pile wall may be taken to be 15° for active earth pressure, and −15° for passive earth pressure. For the residual water level, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**.

(7) It is desirable that the width of the relieving platform be extended to the range where it intersects with the active failure plane extending from the seabed. However, if the use of a narrow relieving platform is unavoidable, the following method can be used as the method of calculating the active earth pressure acting on the sheet pile wall.

As shown in **Fig. 2.8.5**, the earth pressure acting on the sheet pile wall is calculated as the earth pressure acting in the case that there is no relieving platform below the intersection point of the active failure plane extending from the rear end of the relieving platform and the sheet pile, and as the earth pressure acting in (2) above, above the point of intersection of the natural failure plane during Level 1 earthquake ground motion extending from the rear end of the relieving platform and the sheet pile. Between these two, it may be assumed that the earth pressure varies linearly.

The angle (α) formed between the natural failure plane and the horizontal during an earthquake can generally be obtained from **equation (2.8.1)**.

$$\alpha = \phi - \tan^{-1} k_h' \qquad \textbf{(2.8.1)}$$

where

ϕ : angle of shearing resistance of the soil (°)

k_h' : apparent seismic coefficient.

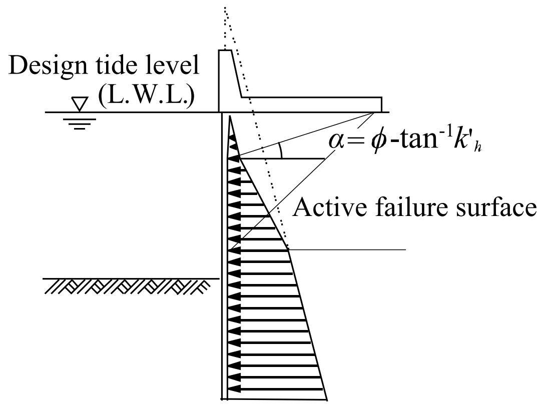


Fig.2.8.5 Earth Pressure Acting on Sheet Pile with Narrow Relieving Platform

(8) Relieving platform piles driven behind sheet piles bear a part of the earth pressure acting on the sheet piles and thereby have the effect of reducing the earth pressure acting on the sheet piles. Since there are many uncertainties in this effect, it is common that this effect is not taken into consideration for performance verification. There are some proposed methods, including a method of determining the distribution of the earth pressure based on the ratio of the flexural rigidity *EI* of the sheet piles to that of the relieving platform piles[69] and method of calculating the earth pressure acting on the sheet pile based on the ratio of the pile diameter to the center interval of piles[70].

(9) The horizontal force transmitted from the sheet piles and acting on the relieving platform may be calculated with the same method as that for the reaction at tie member installation point obtained in accordance with **Part III, Chapter 5**, **2.3.7 Performance Verification of Stability of Sheet Pile Walls as a Whole** by regarding the bottom of the relieving platform as a tie member installation point.

(10) The tractive force by ship and fender reaction force also act on the relieving platform. These external forces should be considered as necessary.

(11) The external forces transmitted from the sheet piles to the relieving platform include the horizontal force and bending moment. However, the transmission of the bending moment may be ignored for the sake of safety because the fixing of the sheet piles to the relieving platform may not be rigid enough.

(12) The earth pressure and residual water pressure acting on the back of the relieving platform can be calculated in accordance with **Part II, Chapter 4, 2 Earth Pressure** and **Part II, Chapter 4, 3.1 Residual Water Pressure**. In the calculation of earth pressure, surcharge should be taken into consideration. In the part below the bottom of relieving platform, the difference between the passive earth pressure acting on the rear and that acting on the front acts as the active earth pressure down to the depth where the two pressures are balanced. This should be added as shown in **Fig. 2.8.6**. The angle of wall friction can be taken to be 15° for active earth pressure and −15° for passive earth pressure.

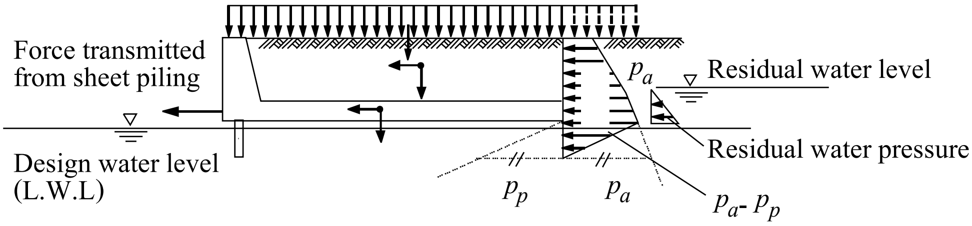


Fig. 2.8.6 External Forces to be considered for Performance Verification of a Relieving Platform

(13) For the self-weight of the relieving platform and the weight of soil and surcharge on the relieving platform, refer to **Part II, Chapter 10**, **2 Self-Weight** and **Part II, Chapter 10**, **3 Surcharge**.

(14) For the dynamic water pressure during action of Level 1 earthquake ground motion, refer to **Part II, Chapter 4**, **3.2 Dynamic Water Pressure**.

(15) When the relieving platform is an L-shaped structure, the earth pressure and residual water pressure acting on the upright section may be calculated in accordance with **Part II, Chapter 4, 2 Earth Pressure** and **Part II, Chapter 4, 3.1 Residual Water Pressure**. The angle of wall friction can be taken to be 15°.

2.8.4 Performance Verification

(1) Performance verification of sheet pile walls

① The embedment length of the sheet pile wall can be examined by assuming that the joint between a sheet pile and the relieving platform is a hinge support, replacing the bottom of the relieving platform with a tie member installation point and applying **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls.**

② Verification of stresses in the sheet pile wall may be conducted in accordance with **Part III, Chapter 5, 2.3 Sheet Piled Quaywalls**, replacing the bottom of the relieving platform with the tie member installation point.

③ In addition to the bending moment due to earth pressure, the bending moment and vertical force transmitted from the relieving platform act on the sheet piles of a sheet pile wall. Normally, the bending moment transmitted from the relieving platform is not taken into consideration because it usually acts in a direction opposite to that of the maximum bending moment that acts on the sheet piles and thus reduces the maximum bending moment. Furthermore, the vertical force transmitted from the relieving platform to the sheet pile wall is normally not taken into consideration when the front row of relieving platform piles is driven in as close to the sheet piles as possible and this significantly reduces the vertical force acting on the sheet piles.

(2) Performance verification of the relieving platform

① A relieving platform should be verified for performance as continuous beams that run in the direction of the quaywall alignment and in the direction perpendicular to the alignment and are supported at heads of relieving platform piles (see **Fig. 2.8.7**). In this case, loads cannot be distributed in the two directions. When the relieving platform is an L-shaped structure, the upright section should be verified for performance as a cantilever beam supported at the bottom slab.

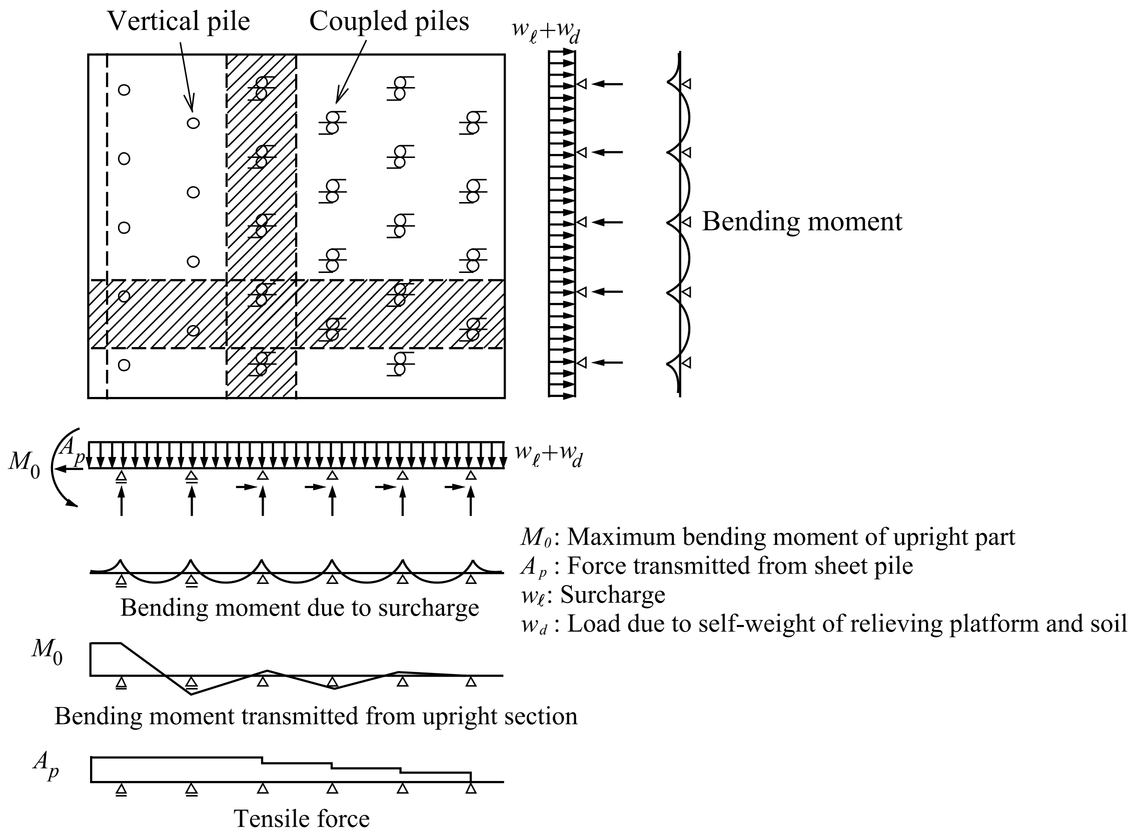


Fig.2.8.7 Continuous Beams Assumed in Performance Verification of a Relieving Platform

② When the relieving platform is an L-shaped structure, the continuous beams that run in the direction of quaywall alignment and in the direction perpendicular to the alignment are subjected to not only the bending moment due to the vertical action alone but also the bending moment transmitted from the upright section of the relieving platform. Therefore, the bending moment at the bottom slab of the relieving platform shall be calculated in consideration of the sum of these bending moments. A convenient way to calculate the bending moment transmitted from the upright section of the relieving platform is to transmit the maximum bending moment at the upright section using the moment distribution method.

③ A great horizontal force from the sheet pile wall acts on the relieving platform but this force can be directly transmitted to relieving platform piles by connecting the sheet piles and the relieving platform piles using tie members. Such a structure may be advantageous in the performance verification of the relieving platform. For the performance verification of tie members used for this purpose, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**. For the calculation of the horizontal force at heads of piles, refer to **(3) Performance Verification of Relieving Platform Piles** below.

④ Normally, relieving platform piles are arranged at short intervals. In principle, reinforcing bars for members of a relieving platform should be arranged at regular intervals without distributing the bending moment to column strips and middle strips.

(3) Performance verification of relieving platform piles

① Performance of relieving platform piles can be verified in accordance with **Part III, Chapter 2, 3.4 Pile Foundations.**

② In principle, relieving platform piles should consist of a combination of coupled piles and vertical piles. The horizontal external force may be borne by the coupled piles alone, and the vertical external force may be borne by the vertical piles alone. It may be assumed that each of the coupled piles bears the horizontal force equally.

③ In the performance verification of relieving platform piles, assessment should be made for the most dangerous state of each pile by varying the surcharge, direction of seismic forces, and tide level within the design condition ranges.

④ In calculating the axial resistance of each of the relieving platform piles, it is desirable to assume that in the ground above the sheet pile active failure plane extending from the seabed, the skin friction does not contribute as the resistance force of the relieving platform piles.

⑤ If it is unavoidable that the relieving platform piles are all composed of vertical piles and the horizontal force is borne by the vertical piles, it is normally assumed in calculating the resistance force normal to their axes that there is no soil above the sheet pile active failure plane extending from the seabed.

⑥ It is desirable to arrange relieving platform piles in such a way as to minimize the vertical force that comes from the relieving platform and acts on the sheet pile wall.

⑦ It is desirable to determine the length of each of relieving platform piles so that they reach almost the same depth.

(4) Analysis of the Stability as Gravity-type Walls

① The examination of the stability of a quaywall with a relieving platform as a whole can be made by assuming that the quaywall with a relieving platform is a kind of gravity-type wall.

② For analyzing the stability of the assumed gravity-type wall, refer to **Part III, Chapter 5, 2.2 Gravity-type Quaywalls**. In this case, the passive earth pressure to the front of the sheet pile can be taken into consideration.

③ A quaywall with a relieving platform may be considered as a gravity-type wall of which the rectangular cross-section is defined by a vertical plane containing the rear face of the relieving platform and a horizontal plane containing the bottom ends of the front side batter piles of the coupled piles, as shown in **Fig. 2.8.8**.

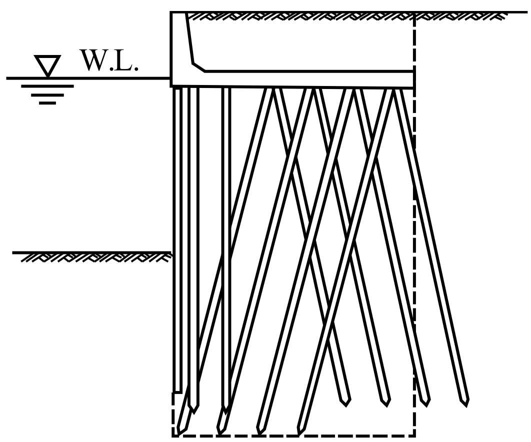


Fig.2.8.8 Virtual Wall as Gravity-type Wall

④ In principle, the frictional resistance acting on the bottom of the gravity-type wall can be assumed to be the product of the total vertical force acting on the wall and tan ϕ when the ground at the wall bottom is made of sandy soil or the product of the cohesion of clayey soil and the area of the wall bottom when the ground at the wall bottom is made of clayey soil. The total vertical force acting on the wall is a weight of the wall not including surcharges and calculated by subtracting buoyancy, and ϕ is the angle of shear resistance of sandy soil.

⑤ In principle, the angle of wall friction to be used in the calculation of earth pressure may be taken to be 15° for active earth pressure, and −15° for passive earth pressure. For clayey soil at the seabed or below, the apparent seismic coefficient to be used in the calculation of earth pressure during earthquake may be calculated in accordance with **Part II, Chapter 4**, **2 Earth Pressure**.

(5) Verification of circular slip failure

① For analysis of circular slip failure, refer to **Part III, Chapter 2, 4.2.1 Stability Analysis by Circular Slip Failure Plane**. In this case, analysis is conducted for circular slip failure passing under the bottom end of the sheet piling. Also, for setting the water level, refer to **Part II, Chapter 2, 3 Tide Levels**.

② When a quaywall with a relieving platform is considered unstable against circular slip failure, it is necessary to take appropriate measures, such as soil improvement and adoption of other type of structure. It is undesirable to increase the embedment length of sheet piles for the purpose of preventing circular slip failure.

2.8.5 Performance Verification of Structural Members

(1) It is necessary to connect a sheet pile wall and relieving platform piles to a relieving platform in such a way as to assure the required safety against stresses that occur at the connections.

(2) In principle, the top of a sheet pile shall be connected to a relieving platform either by embedding the top of the sheet pile into the relieving platform to a sufficient depth and welding reinforcing bars to the sheet pile or by attaching tie rods to convey the horizontal force from the sheet pile to relieving platform piles (see **Fig. 2.8.9**). When attaching tie members, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**.

(3) In principle, relieving platform piles shall be embedded into the bottom slab of the relieving platform to a sufficient depth so that the pile head reaction can be conveyed to the relieving platform. It is advisable to secure the heads of coupled piles by bolting or other means so that the piles can work as a unit (see **Fig. 2.8.9**).

(4) For details of the performance verification of structural members of quaywalls with relieving platforms, refer to those of open-type wharves on vertical piles described in **Part III, Chapter 5**, **5.2.6 Performance Verification of Structural Members**. In particular, for piles subjected to pulling force, it is necessary to thoroughly examine how to fix them to a relieving platform.

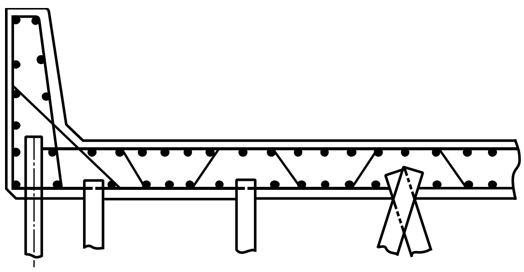


Fig. 2.8.9 Connections of Sheet Pile and Piles to a Relieving Platform

2.9 Embedded-Type Cellular-Bulkhead Quaywalls

[Public Notice] (Performance Criteria of Cellular-Bulkhead Quaywalls)

Article 52

1 The performance criteria of cellular-bulkhead quaywalls shall be as prescribed respectively in the following items:

(1) The following criteria shall be satisfied under the permanent situation, in which the dominating action is earth pressure:

(a) The risk of losing stability due to the shear deformation of the wall body shall be equal to or less than the threshold level.

(b) The risk of impairing the integrity of the members of the cellular-bulkhead quaywalls shall be equal to or less than the threshold level.

(2) The following criteria shall be satisfied under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(a) The risk of occurrence of sliding of the wall body and failure due to the insufficient bearing capacity of the foundation ground shall be equal to or less than the threshold level.

(b) The risk that the amount of deformation of the top of the cells may exceed the allowable limit of deformation shall be equal to or less than the threshold level.

(3) The risk of occurrence of slip failure in the ground shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is self-weight.

(4) The following criteria shall be satisfied by the superstructure of cellular-bulkhead quaywalls under the permanent situation in which the dominating action is earth pressure, and under the variable situation in which the dominating actions are Level 1 earthquake ground motions, ship berthing, and traction by ships.

(a) The risk that the axial force acting in a pile may exceed the resistance force on the basis of the failure of the ground shall be equal to or less than the threshold level.

(b) The risk that the stresses in the piles may exceed the yield stress shall be equal to or less than the threshold level.

(c) The risk of impairing the integrity of the members shall be equal to or less than the threshold level.

2 In addition to the provisions of the preceding paragraph, for the performance criteria of placement-type cellular-bulkhead quaywalls, the risk of occurrence of overturning under the variable situation, in which the dominating action is Level 1 earthquake ground motions, shall be equal to or less than the threshold level.

[Interpretation]

11. Mooring Facilities

(6) Performance Criteria of Cellular-Bulkhead Quaywalls

① **Embedded-type cellular-bulkhead quaywalls** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 52, Paragraph 1 of the Public Notice

(a) Serviceability shall be the required performance for embedded-type cellular-bulkhead quaywalls under the permanent state in which the dominating action is self-weight and earth pressure and under the variable state in which the dominating action is Level 1 earthquake ground motions, ship berthing or traction by ships. The performance verification items for those actions and standard indexes for setting the limit values shall be in accordance with **Attached Tables 11-15** and **11-16**.

Attached Table 11-15 Performance Verification Items and Standard Indexes for Setting Limit Values for the Structural Stability of Cells and the Integrity of Members of Embedded-type Cellular-bulkhead Quaywalls under Different Design Situations

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Index for setting limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="11">26</td><td rowspan="11">1</td><td rowspan="11">2</td><td rowspan="11">52</td><td rowspan="11">1</td><td>1 (a)</td><td rowspan="11">Serviceability</td><td rowspan="4">Permanent</td><td rowspan="4">Earth pressure</td><td rowspan="4">Water pressure, surcharges</td><td>Shear deformation of wall</td><td>Action-to-resistance ratio for the shear deformation of the wall body</td></tr>
<tr><td rowspan="3">1 (b)</td><td>Yielding of cell</td><td>Design yield stress</td></tr>
<tr><td>Yielding arc</td><td>Design yield stress</td></tr>
<tr><td>Joint yielding</td><td>Design yield stress</td></tr>
<tr><td rowspan="2">2 (a)</td><td>Permanent</td><td>Earth pressure</td><td>Self-weight, water pressure, surcharges</td><td rowspan="2">Wall sliding, bearing capacity of foundation ground</td><td>Action-to-resistance ratio for the sliding of the wall body and the bearing capacity of the foundation ground</td></tr>
<tr><td>Variable</td><td>Level 1 earthquake ground motion</td><td>Self-weight, earth pressure, water pressure, surcharges</td><td>Action-to-resistance ratio for the sliding of the wall body and the bearing capacity of the foundation ground</td></tr>
<tr><td rowspan="2">2 (b)</td><td>Permanent</td><td>Earth pressure</td><td>Water pressure, surcharges</td><td rowspan="2">Deformation of cell top</td><td rowspan="2">Residual deformation amount at the top of the quaywall</td></tr>
<tr><td>Variable</td><td>Level 1 earthquake ground motion</td><td>Self-weight, earth pressure, water pressure, surcharges</td></tr>
<tr><td>3</td><td>Permanent</td><td>Self-weight</td><td>Water pressure, surcharges</td><td>Circular slip failure of ground</td><td>Action-to-resistance ratio for circular slip failure</td></tr>
</table>

Attached Table 11-16 Performance Verification Items and Standard Indexes for Setting Limit Values for Superstructure of Embedded-type Cellular-bulkhead Quaywalls under Different Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	52	1	4 (a)	Serviceability	Permanent	Earth pressure	Self-weight, water pressure, surcharges	Axial forces acting on superstructure piles*1)	Action-to-resistance ratio for the supporting capacity of the piles (pushing and pulling)
							Variable	Level 1 earthquake ground motion	Self-weight, earth pressure, water pressure. surcharges		
								Traction of ships			
					4 (b)		Permanent	Earth pressure	Water pressure. surcharges	Yielding of superstructure piles*1)	Design yield stress
							Variable	Level 1 earthquake ground motion	Self-weight, earth pressure, water pressure. surcharges		
								Traction of ships			
					4 (c)		Permanent	Earth pressure	Water pressure. surcharges	Sectional stress of the superstructure cross section	Bending compressive stress
							Variable	Level 1 earthquake ground motion	Self-weight, earth pressure, water pressure. surcharges	Cross-sectional failure of superstructure	Design cross-sectional resistance
								Berthing and traction of ships			

*1) Only for structures having superstructure supporting piles

(b) In addition to these provisions, the provisions regarding Article 22, Paragraph 3 (Scouring and Sand Washing Out) and Article 28 (Performance Criteria for Armor Stones and Blocks) of the Public Notice and their interpretations shall apply as needed.

2.9.1 General

(1) This section is applicable to the performance verification of quaywalls using a steel-sheet-pile cellular-bulkhead structure (hereinafter steel-sheet-pile cellular-bulkhead quaywalls) and quaywalls having a steel plate cellular-bulkhead structure with embedded sections (hereinafter embedded-type steel plate cellular-bulkhead quaywalls).

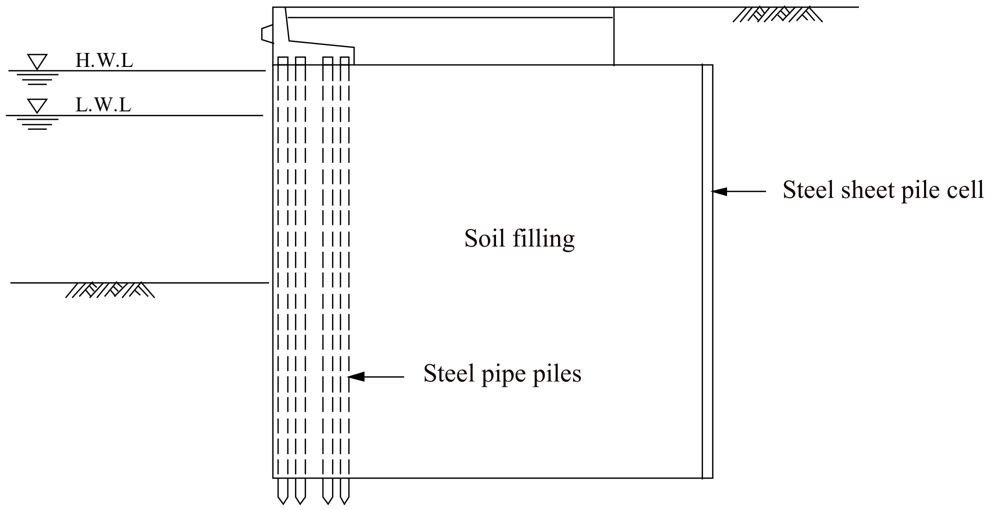
(2) The performance verification method described in this section is mainly based on the results of cellular-bulkhead model tests [71), 72), 73), 74)] conducted on a sandy soil ground with an embedded length ratio of 0 to 1.5 and a ratio of equivalent wall width to wall height of 1 to 2.5. For cases wherein the embedded length ratio is small (i.e., less than 1/8) and the equivalent wall width is small relative to the wall height or cases wherein the quaywall should be constructed on a clay soil ground or ground improved by the sand compaction piles, further examinations (e.g., a numerical analysis that takes into consideration the nonlinear characteristics of the ground) should be preferably

made, in addition to the examination using the performance verification method described in this section, because these cases involve factors that cannot be fully clarified with the method described here.

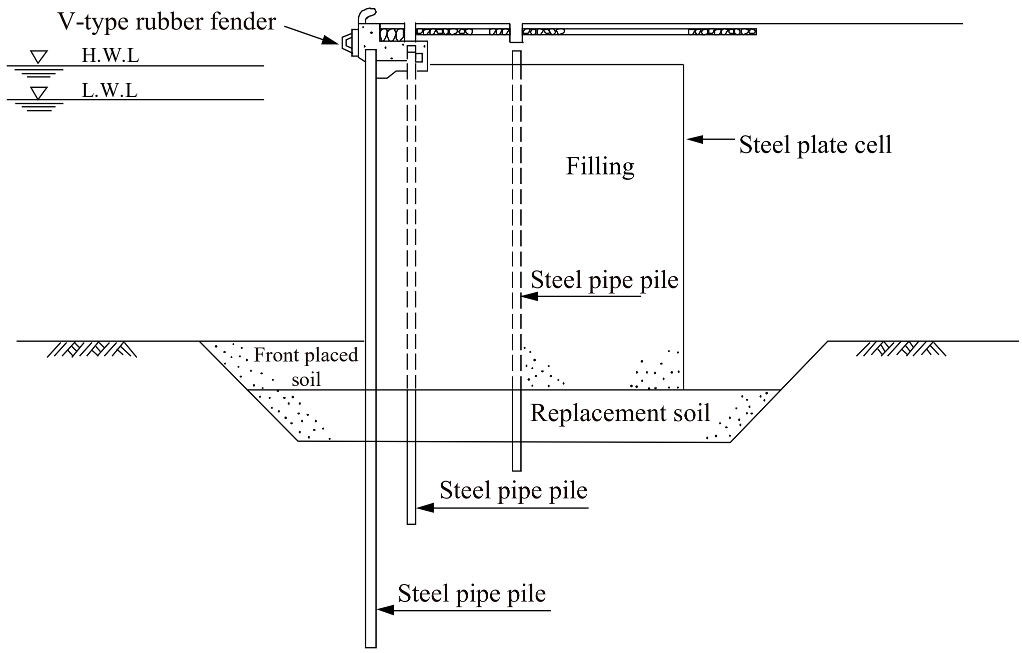
(3) **Fig. 2.9.1** shows examples of the cross section of a steel-sheet-pile cellular-bulkhead quaywall and an embedded-type steel-plate cellular-bulkhead quaywall.

(4) The approaches in **Part III, Chapter 5, 2.9.2 Action** and **Part III, Chapter 5**, **2.9.4 Performance Verification** may be used for simple verification, but it is necessary to be careful when adopting these approaches. Highly accurate methods (model tests and numerical analysis techniques that can simulate mechanisms) should be used for verification.

(5) **Fig. 2.9.2** shows an example of the sequence of performance verification of an embedded-type cellular-bulkhead quaywall. Note that the evaluation of the effects of liquefaction, settlement, and other phenomena caused by earthquake ground motions is not shown in **Fig. 2.9.2**. Therefore, for liquefaction, it is necessary to determine whether there is liquefaction and to consider measures against it appropriately by referring to **Part II, Chapter 7 Ground Liquefaction**. It is possible to verify the variable situation associated with Level 1 earthquake ground motions by using the seismic coefficient method. For high-earthquake-resistance facilities, however, it is desirable to analyze the amount of deformation (e.g., by using the nonlinear seismic response analysis in consideration of the dynamic interaction between the ground and a structure). For embedded-type cellular-bulkhead quaywalls that are not categorized as high-earthquake-resistance facilities, the verification of the accidental situation associated with Level 2 earthquake ground motions can be omitted.

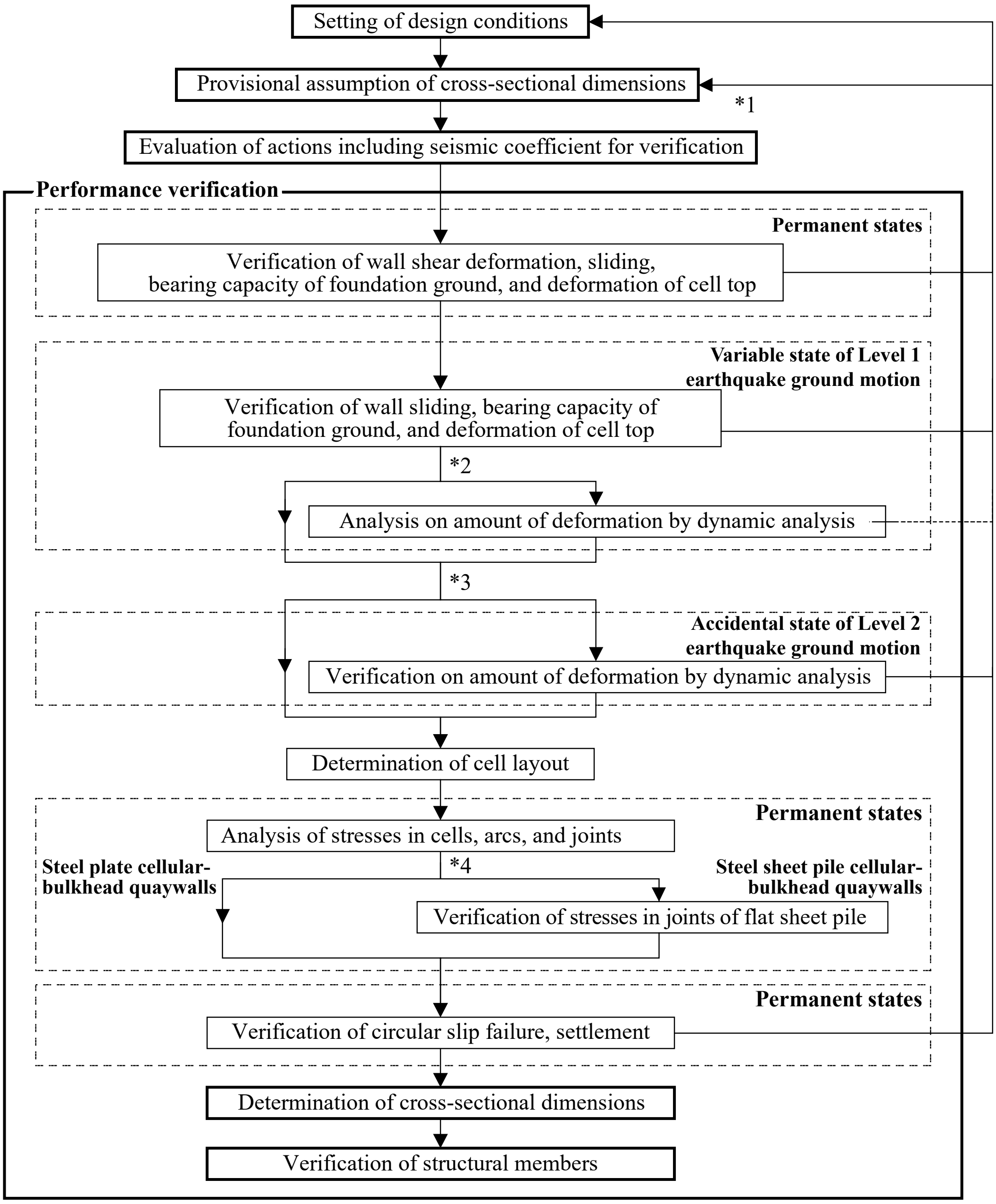


(a) Embedded-type steel sheet pile cellular-bulkhead quaywall



(b) Embedded-type steel plate cellular-bulkhead quaywall

Fig. 2.9.1 Examples of the Cross-section of Embedded-type Cellular-bulkhead Quaywalls



*1: The evaluation of the effect of liquefaction is not shown; therefore, this must be separately considered.
*2: The analysis of the amount of deformation due to Level 1 earthquake ground motion may be performed by dynamic analysis when necessary.
For high-earthquake-resistance facilities, the analysis of the amount of deformation due to Level 1 earthquake ground motion should also be performed by dynamic analysis.
*3: For high-earthquake-resistance facilities, verification is performed for Level 2 earthquake ground motion.
*4: For steel-sheet-pile cellular-bulkhead quaywalls, verification is performed for the joints of a flat sheet pile.

Fig. 2.9.2 Example of the Sequence of Performance Verification of Embedded-type Cellular-bulkhead Quaywalls

(6) It is recommended that the filling material in cells is a sufficient density sand or gravel of good quality. It is not desirable to use clayey soil as filling material. When clayey soil remained in the cells, it is necessary to make a separate examination because the deformation of the cells may become significantly large.

(7) When a foundation for a crane, shed, or warehouse is to be built within a cell, it is desirable to use foundation piles to transmit the load to the bearing stratum.

(8) When constructing steel-sheet-pile cells, the cells should be filled as soon as possible after driving the sheet piles to reduce the time during which the cells are unstable without any filling.

2.9.2 Actions

(1) To calculate the action to be considered in the performance verification of embedded-type cellular-bulkhead quaywalls, refer to **Part II, Chapter 6**, **2 Seismic Action**, **Part II, Chapter 4**, **2 Earth Pressure**, **Part II, Chapter 4**, **3 Water Pressure**, and **Part II, Chapter 10 Self-Weight and Surcharges**.

(2) During the examination of shear deformation of the cell wall body, the rear of the wall may be subjected to active earth pressure (see **Fig. 2.9.3**). The results of the model tests show that the embedded section of the cell is subjected to the action corresponding to the earth pressure at rest because the deformation of the embedded section of the cell is small. According to the results of the shaking table tests, the earth pressure acting on this part works as a resisting force against overturning of the wall. In examining the stability of the entire system, it can be generally considered that the active earth pressure acts on the rear of the wall above the seabed and that the seabed earth pressure acts on the rear of the wall at the seabed or below regardless of depth (see **Fig. 2.9.4**). The characteristic value of the earth pressure at the seabed can normally be calculated using **equation (2.9.1)**.

$$p_{ac} = 0.5\left(\sum wh + q\right), \tag{2.9.1}$$

where

p_{ac} : earth pressure acting on the rear of wall below the seabed (kN/m^2);

w : unit weight of each layer of backfilling (kN/m^3);

h : thickness of each layer of backfilling (m);

q : surcharge (kN/m^2).

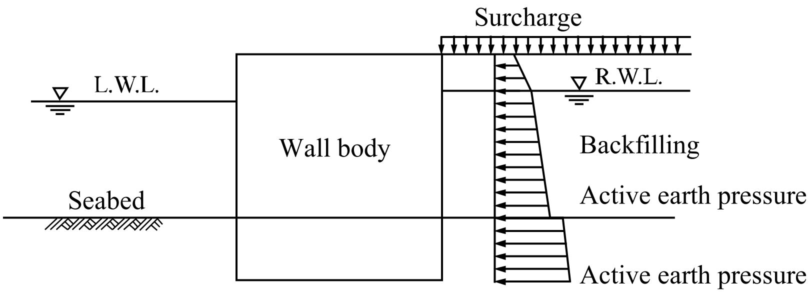


Fig. 2.9.3 Earth Pressure Acting on the Rear of Wall Body for Examination of Shear Deformation

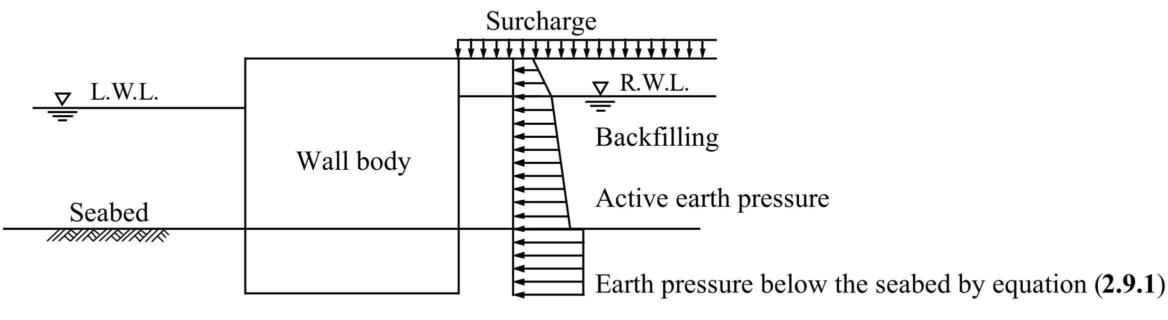


Fig. 2.9.4 Earth Pressure Acting on the Rear of Wall Body for Examination of the Stability as Gravity-type Wall

(3) In principle, the residual water level of the backfilling can be taken at the elevation with the height equivalent to two-thirds of the tidal range above the monthly mean lowest water level (L.W.L.). However, when using a backfilling with low permeability, the residual water level may become higher than this level. Therefore, it is desirable to determine the residual water level on the basis of the results of the investigations of existing structures and similar structures. The residual water level in the filling material in the cells may be set to the same level as that of the backfilling for the wall body. For steel-plate cells, joints are provided only at the connections between the cells and arcs. Therefore, the water blocking capability of steel-plate cells can be considered equivalent to or higher

than that of steel-sheet-pile cells, although there is no data on the actual measurements of the water blocking capability. It is necessary to note that the water level may significantly differ from that at the front of the cells when the ground water level at the rear of a quaywall rises due to rain or for any other reason.

(4) Seismic coefficient for verification used in the performance verification of embedded-type cellular-bulkhead quaywalls

① The characteristic value of the seismic coefficient for the performance verification of embedded-type cellular-bulkhead quaywalls under variable situations associated with Level 1 earthquake ground motion, as well as the allowable value of the amount of deformation set corresponding to the seismic coefficient for verification, shall be appropriately calculated by taking the structural characteristics into consideration.

② The characteristic value of the seismic coefficient for the verification of embedded-type cellular-bulkhead quaywalls can be calculated by using **equation (2.9.2)**. The basic approach described in **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls** shall apply to the calculation. For the filter used for considering frequency characteristics, the reduction coefficient, the equation for calculating the seismic coefficient for verification, and the allowable value of the amount of deformation, refer to **References (Part III)**, **Chapter 1, 1 Details about Seismic Coefficient for Verification**[75].

$$k_h = 1.62\left(\frac{D_a}{D_r}\right)^{-0.58} \cdot \frac{\alpha_c}{g} + 0.04, \tag{2.9.2}$$

where

k_h : seismic coefficient for verification;

D_a : allowable deformation (10 cm);

D_r : referenfedeformation (10 cm);

g : acceleration of gravity (980 cm/s^2).

(5) For the part above the seabed, the seismic coefficient to be used in the calculation of the seismic inertia force that acts on the filling material shall be the seismic coefficient for verification. For the seabed and area below the seabed, this value is reduced linearly in such a way that it becomes 0 at 10 m below the seabed. In principle, the seismic inertia force is not considered for parts deeper than that level (see **Fig. 2.9.5**).

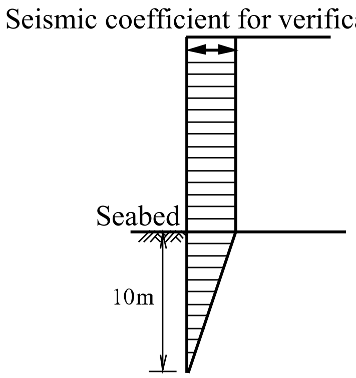


Fig. 2.9.5 Inertia Force Acting on Filling

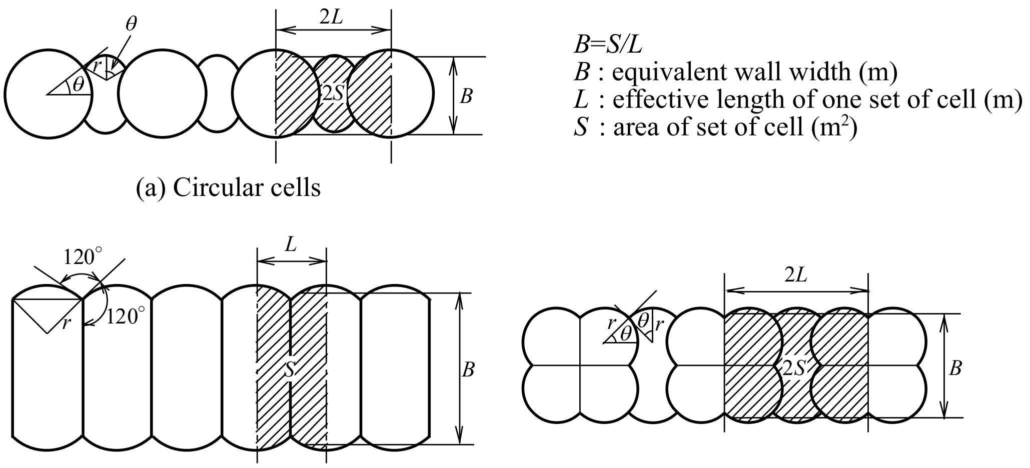
(6) With regard to setting the tide level, refer to **Part II, Chapter 2, 3.6 Design Tide Level Conditions**.

2.9.3 Setting of the Equivalent Wall Width

(1) The equivalent wall width may be used for performance verification. In this case, the equivalent wall width shall be the width of a rectangular virtual wall substituted for the combination of cells and arcs.

(2) The equivalent wall width is the width of a rectangular virtual wall width that is used in place of the combination of cells and arcs to simplify performance verifications (see **Fig. 2.9.6**). The virtual wall is defined in such a way that the area of the horizontal cross section of the virtual wall body becomes the same as that of the actual wall body.

(3) The equivalent wall width is normally determined to satisfy the analysis of the shear deformation of the wall body.



(a) Circular cells

(b) Diaphragm Type Cells

(c) Clover Leaf Type Cells

Fig. 2.9.6 Plain View of Cellular-bulkhead Quaywall Structure and Equivalent Wall Width *B*

2.9.4 Performance Verification

(1) Analysis of the Shear Deformation of the Wall Body

① The cell shell and filling of the cellular-bulkhead quaywall usually act as an integrated structure because the filling is constrained in the cell shell. Therefore, the deformation of the cell wall body may be ignored relative to its displacement, and the overall behavior of the cell wall body may be considered to be the same as that of a rigid body. This has been verified by model tests in which the cell wall body did not show significant deformation under loads larger than the external forces that are expected to act on the cell wall body both under a permanent situation and variable situation associated with Level 1 earthquake ground motion. However, when the diameter of the cell is small or the strength of the filling material is extremely low, it may not be possible to satisfy the assumption that the cell wall body is a rigid body. Therefore, it is necessary to examine the strength of the filling against shear deformation due to loads under a permanent situation to remain the deformation of the cell wall body to a negligible level.

② It is normally possible to analyze the shear deformation of the steel-sheet-pile cellular-bulkhead quaywalls with **equations (2.9.3)** and **(2.9.4)** by using the resistant and deformation moments at the cell bottom and the resistant and deformation moments of the soil within the cells at the seabed. Furthermore, an analysis of the shear deformation of steel-plate cellular-bulkhead quaywalls can be performed using **equation (2.9.4)**. Subscripts k and d in the following equations indicate the characteristic value and design value, respectively. For the calculation of the characteristic values, refer to ③ **Calculation of deformation moment**, ④ **Calculation of the resistant moment at the cell bottom**, and ⑤ **Resistant moment of the filling with respect to the seabed**. An appropriate value of 1.20 or higher may be used as the adjustment factor m, and a value of 1.00 may be used as γ_R and γ_S to simplify the calculations.

$$\left.\begin{aligned} & m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\ & S_k = M_{d_k} \\ & R_k = M_{r_k} \end{aligned}\right\}, \tag{2.9.3}$$

$$\left.\begin{aligned} & m \cdot \frac{S_d}{R_d} \le 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\ & S_k = M'_{d_k} \\ & R_k = M'_{r_k} \end{aligned}\right\}, \qquad \textbf{(2.9.4)}$$

where

M_r : resistant moment at the cell bottom (kN·m/m);

M_d : deformation moment at the cell bottom (kN·m/m);

M'_r : resistant moment of filling soil at the seabed (kN·m/m);

M'_d : deformation moment at the seabed (kN·m/m);

R : resistance term (kN·m/m);

S : load term (kN·m/m);

γ_R : partial factor by which the resistance term is multiplied;

γ_S : partial factor by which the load term is multiplied;

m : adjustment factor.

③ **Calculation of deformation moment**

(a) The deformation moment to be used in the performance verification of steel-sheet-pile cellular-bulkhead quaywalls shall be the moment at the cell bottom or the seabed due to external forces such as active and passive earth pressures and the residual water pressure above the cell bottom or the seabed. The deformation moment for steel-plate cellular-bulkhead quaywalls shall be the moment at the seabed due to external forces such as active and passive earth pressures and the residual water pressure above the seabed.

(b) Earth pressure is considered only in terms of the horizontal component in the calculation of the deformation moment. The vertical component is not taken into consideration. The vertical force of the surcharge that acts on the cell top is not taken into consideration in the calculation of deformation moment. However, the surcharge is taken into consideration in the calculation of active earth pressure (**Fig. 2.9.7**).

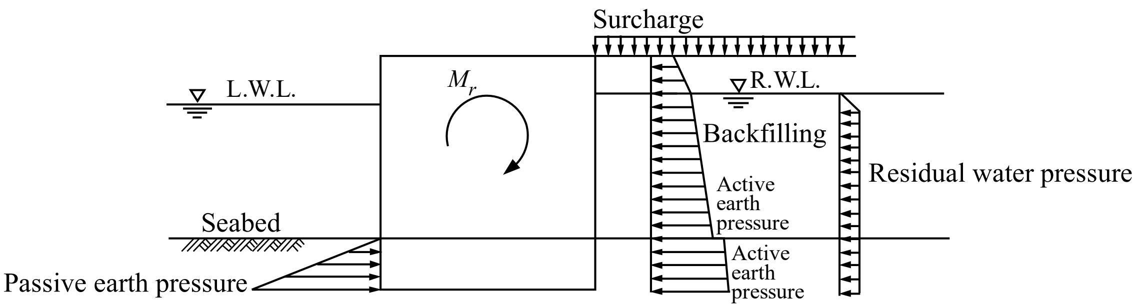


Fig. 2.9.7 Loads and Resisting Forces to be taken into consideration in the Examination of Shear Deformation

④ **Calculation of resistant moment at the cell bottom**

(a) The resistant moment at the cell bottom shall be calculated appropriately in consideration of the structural characteristics of the cell and deformation of the wall.

(b) The result of the model tests[71)] shows that the resistant moment with respect to the cell bottom may be increased by increasing the embedded length ratio D/H (**Fig. 2.9.8**). This can be calculated using **equation (2.9.5)**.

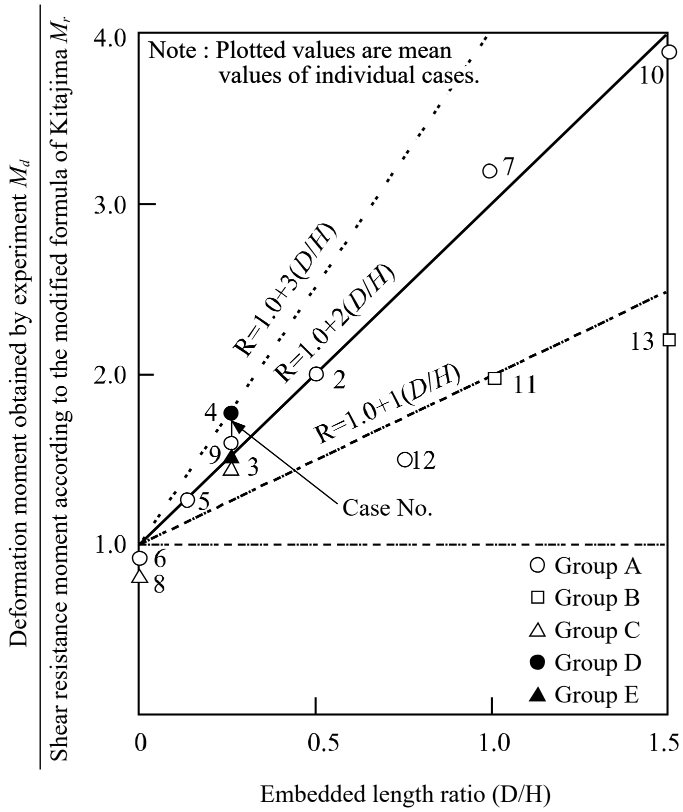


Fig. 2.9.8 Relationship between Resistant Moment and Embedded Length Ratio

$$M_{r_k} = \left(M_{r0_k} + M_{rs_k}\right)\left(1 + \alpha \frac{D}{H}\right), \tag{2.9.5}$$

where

M_r : resistant moment with respect to the cell bottom (kN·m/m);

M_{r0} : resistant moment of the filling with respect to the cell bottom (kN·m/m);

M_{rs} : resistant moment due to the friction force of sheet-pile joints with respect to the cell bottom (kN·m/m);

D : embedded length (m);

H : height from the seabed to the wall top (m) (see **Fig. 2.9.9**);

α : required additional rate against the embedded length ratio (D/H).

It is recommended to use 1.0, which is close to the lowest value found in the test results shown in **Fig. 2.9.8**, for the required additional rate α because the equation given above has been derived on the basis of tests and has not been fully clarified theoretically.

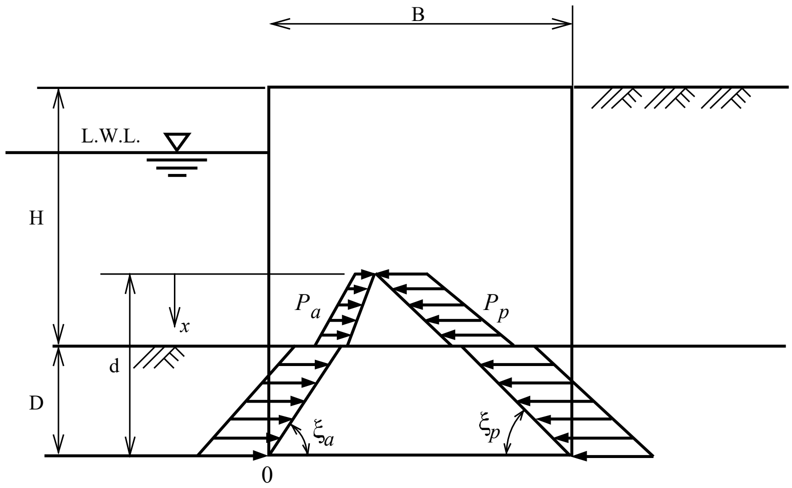


Fig. 2.9.9 Assumed shear surface of filling soil

(c) Equation for Calculating the Resistant Moment of the Filling

In the determination of the resistant moment of the filling at the cell bottom, it is assumed that an active failure surface is generated from the front of the cell bottom, a passive failure surface is generated from the rear, and active and passive earth pressures act on the respective failure surfaces (**Fig. 2.9.9**). The active and passive failure angles, as well as the active and passive earth pressures, may be calculated using Rankine's equations. Subscript k in the equations indicates the characteristic value.

$$\left.\begin{array}{lll} \text{active failure surface} & \zeta_{a_k} = \dfrac{\pi}{4} + \dfrac{\phi_k}{2} & \\ \text{passive failure surface} & \zeta_{p_k} = \dfrac{\pi}{4} - \dfrac{\phi_k}{2} & \\ \text{active earth pressure} & P_{a_k} = K_a w_k h\ , & K_a = \dfrac{1-\sin\phi_k}{1+\sin\phi_k} \\ \text{passive earth pressure} & P_{p_k} = K_p w_k h\ , & K_p = \dfrac{1+\sin\phi_k}{1-\sin\phi_k} \end{array}\right\} \tag{2.9.6}$$

where

ϕ : angle of shear resistance of filling (°);

w : unit weight of the soil (kN/m^3);

h : thickness of the soil layer (m).

The moment caused by the earth pressure acting on the shear surface may be calculated by using **equation (2.9.7)** (see **Fig. 2.9.9**).

$$M_{r0_k} = \int_0^d (P_{p_k} - P_{a_k})(d-x)\frac{2}{3}\tan\phi_k dx \tag{2.9.7}$$

When the geotechnical constants of the ground and those of the filling differ, **equation (2.9.7)** becomes complex as the failure angle, and the earth pressure level varies from one soil layer to another. However, when there is no significant difference in the angle of shear resistance between the ground and filling or when the embedded length ratio is large and the failure surfaces do not reach the filling portion, the following simplified equation may be used. In the equations below, subscript k indicates the characteristic value.

$$M_{r0_k} = \frac{1}{6} w_{0_k} R_0 H_0{}^3, \tag{2.9.8}$$

$$R_{0_k} = \frac{2}{3} \nu_{0_k}{}^2 \left(3 - \nu_{0_k} \cos\phi_k\right) \tan\phi_k \sin\phi_k, \tag{2.9.9}$$

where

w_0 : equivalent unit weight of the filling that assumes that the unit weight is uniform throughout the filling (normally, w_{0k} = 10 kN/m^3 is used);

H_0 : equivalent wall height measured from the cell bottom (the equivalent wall height is employed to calculate the resistant moment due to the filling by using the equivalent unit weight of the filling, and it is calculated by **equation (2.9.10)**);

ϕ : angle of the shear resistance of the filling (°).

$$H_{0_k} = \frac{1}{w_{0_k}} \sum_i w_{i_k} h_i, \tag{2.9.10}$$

w_i : unit weight of the i-th layer of the filling (kN/m^3);

h_i : thickness of the i-th layer from the cell bottom to the top of the quaywall (m).

$$\nu_{0_k} = \frac{B}{H_{0_k}}, \tag{2.9.11}$$

B : equivalent wall width (m).

(d) Equation for Calculating the Resistant Moment due to the Friction Force of the Joints of the Sheet Piles

The resistant moment due to the friction force of the joints may be calculated as follows. In the equations below, subscript k indicates for the characteristic value.

$$M_{rs_k} = \frac{1}{6} w_{0_k} R_{s_k} H_{s_k}{}^3, \tag{2.9.12}$$

$$R_{s_k} = \frac{3}{2} \nu_{sk} f \tan\phi_k, \tag{2.9.13}$$

where

H_{sk} : The equivalent wall height measured from the cell bottom and employed to calculate the resistant moment due to the friction force between the sheet-pile joints when the equivalent unit weight of the filling is used. It is evaluated using **equation (2.9.14)** so that the resultant force of the distributed earth pressure in diagram (a) becomes equal to that of (b) in **Fig. 2.9.10**. In this calculation, 0.5tanϕ can be used as the coefficient of the earth pressure of the filling.

$$H_{s_k} = 2\sqrt{\frac{\sum P_{i_k}}{w_{0_k} \tan\phi_k}}, \tag{2.9.14}$$

P_i : resultant earth pressure of the i–th layer of filling (kN/m) (in this case, the surcharge is ignored);

w_0 : equivalent unit weight of filling (kN/m^3);

ϕ : angle of shear resistance of filling (°).

$$\nu_{s_k} = \frac{B}{H_{s_k}}, \tag{2.9.15}$$

B : equivalent wall width (m);

f : friction coefficient of the sheet-pile interlock (usually, 0.3 is used).

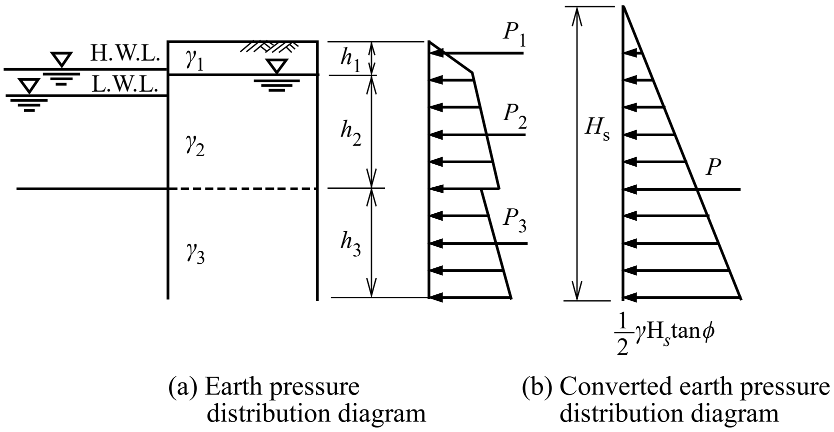


Fig. 2.9.10 Equivalent Wall Height

⑤ **Resistant moment of the filling with respect to the seabed**

(a) The resistant moment with respect to the seabed should be calculated appropriately by taking into consideration the structural characteristics of the cell and the deformation of the wall.

(b) In the calculation of the resistant moment of the filling with respect to the seabed, **equations (2.9.16)** and **(2.9.17)** may be used. Subscript k in the equations indicates the characteristics value:

$$M'_{r_k} = \frac{1}{6} w_{0_k} R_{0_k} H'^{\,3}_{0_k}, \tag{2.9.16}$$

$$R_{0_k}' = v_{0_k}'^{2}(3 - v_{0_k}' \cos\phi')\sin\phi', \tag{2.9.17}$$

where

M_r' : resistant moment of sheet-pile cell with respect to seabed (kN·m/m);

H_0' : equivalent wall height measured from the seabed (the equivalent wall height is employed to calculate the resistant moment due to the filling by using the equivalent unit weight of the filling, and it is evaluated by means of **equation (2.9.18)**);

$$H_{0_k}' = \frac{1}{w_{0_k}} \sum_i w'_{i_k} h'_i \tag{2.9.18}$$

w'_i : unit weight of the filling of the i-th layer above the seabed (kN/m^3);

h'_i : thickness of the i-th layer above seabed between seabed and top of quaywall (m).

$$v_{0_k}' = \frac{B}{H_{0_k}'} \tag{2.9.19}$$

ϕ' : angle of the shear resistance of the filling above seabed (°).

⑥ Increasing the strength of the filling enhances the rigidity of the cell wall. Therefore, the improvement work of filling is effective in increasing the stability of the cell wall.

⑦ Cells containing clayey soil as a filling material are not considered a desirable structure because there are many uncertainties in the behavior of cells and because clayey soil has higher plasticity than sandy soil. Therefore, the use of clayey soil as a filling material should be avoided whenever possible. The results of the analysis using the finite element method show that the stability of a structure made of cells embedded in clayey soil ground depends on the deformation of the ground at the front of the cellular structure and does not depend on the shear deformation of the filling. Therefore, the resistant moments of cells containing clayey soil as a filling may be calculated in the same way as the analysis of those of cells filled with sandy soil.

The resistant moment due to a filling material M_{r0} and the resistant moment due to the friction force between the sheet-pile joints M_{rs} may be calculated by using **equations (2.9.20)** and **(2.9.21)**. Considering that the resistant moments of cells containing clayey soil as a filling material are unknown, it is necessary to examine shear deformation not only at the seabed and at the cell bottom but also on surfaces that are considered dangerous, such as the bottom of a clayey soil layer (**Fig. 2.9.11**). In this manner, the embedded length ratios for the examined surfaces shall be used in the calculation of the embedding effect. In this case, the adjustment factor m to be used in **equations (2.9.3)** and **(2.9.4)** for the verification of shear deformation may be set as 1.20 or larger, and partial factors may be set as 1.00.

$$\left.\begin{aligned} M_{r0_k} &= \int_0^d (P_{p_k} - P_{a_k})(d-x)dx \\ P_{a_k} &= K_a w_k h - 2c_k\sqrt{K_a}, \quad K_a = \frac{1-\sin\phi_k}{1+\sin\phi_k} \\ P_{p_k} &= K_p w_k h + 2c_k\sqrt{K_p}, \quad K_p = \frac{1+\sin\phi_k}{1-\sin\phi_k} \end{aligned}\right\}, \qquad \textbf{(2.9.20)}$$

where

ϕ : angle of shear resistance of filling (°);

c : cohesion of filling (kN/m^2);

w : unit weight of soil (kN/m^3);

h : thickness of soil layer being considered (m).

$$M_{rs_k} = \frac{2}{3}(P_{1_k} + P_{2_k} + P_{3_k}) f_k B, \qquad \textbf{(2.9.21)}$$

where

P_1, P_2, P_3 : resultant force in each layer of the filling (kN/m), as shown in **Fig. 2.9.12**;

w_1, w_2, w_c : unit weight of filling material in each layer (kN/m), as shown in **Fig. 2.9.12**;

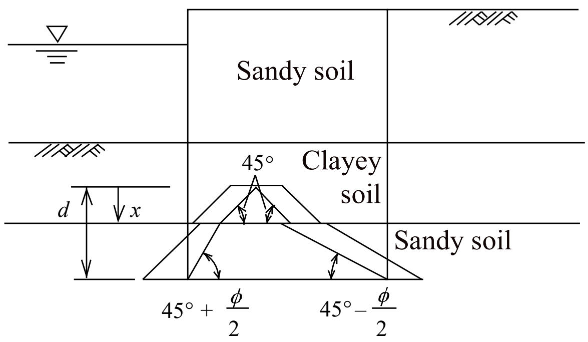
h_1, h_2, h_c : thickness of each layer of the filling (m), as shown in **Fig. 2.9.12**;

K_s : coefficient of earth pressure of sandy soil used for filling (normally, K_s = 0.6 may be used);

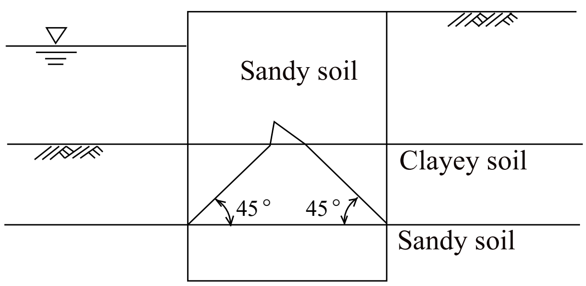
K_c : coefficient of earth pressure of clayey soil used for filling (normally, K_c = 0.5 may be used);

B : equivalent wall width (m);

f : friction coefficient of sheet-pile interlock (normally, f_k = 0.3 may be used).



(a) Cell bottom



(b) Bottom of clayey soil layer

Fig. 2.9.11 Assumption of Shear Surface of Filling

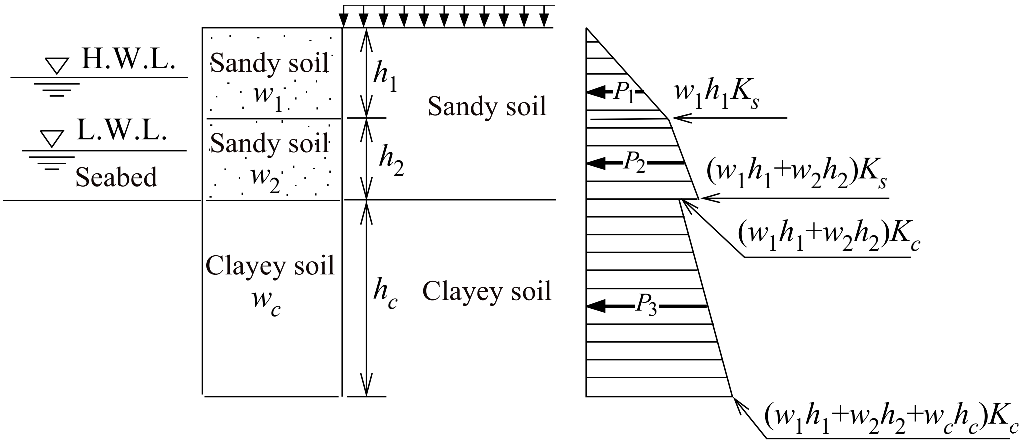


Fig. 2.9.12 Earth Pressure of Filling

(2) Calculation of the amount of deformation of wall body under permanent situations and variable situations associated with Level 1 earthquake ground motion may be carried out based on the following items.

① **General**

(a) In the examination of the stability of the wall as a whole, the subgrade reaction generated against the load and the displacement of the wall are calculated by considering the wall as a rigid body elastically supported by the ground.

(b) Within the elastic range of the ground, the subgrade reaction is calculated as the product of the modulus of subgrade reaction and the displacement. Here, it is considered that the stability of the wall as a gravity wall is obtained when the subgrade reaction and the displacement of the wall do not exceed the respective allowable limits.

② **Modulus of subgrade reaction**

(a) The modulus of the subgrade reaction to be used in the examination of the stability of the wall as a gravity wall shall be set on the basis of the results of soil investigation.

(b) The modulus of subgrade reaction includes the coefficient of lateral subgrade reaction, the coefficient of vertical subgrade reaction, and the horizontal shear modulus at the cell bottom.

(c) The modulus of subgrade reaction may be calculated on the basis of the results of the soil investigation:

1) Coefficient of lateral subgrade reaction

The coefficient of lateral subgrade reaction may be calculated by referring to Yokoyama's diagram[76] shown in **Part III, Chapter 2**, **3.4.7 Calculation of Pile Deflection Using Chang's Method**.

$$k_{CH} = 2000N, \tag{2.9.22}$$

where

k_{CH} : coefficient of lateral subgrade reaction (kN/m³);

N : N value.

The coefficient of lateral subgrade reaction should be calculated for each stratum when the ground consists of the strata of different characteristics.

2) Coefficient of vertical subgrade reaction

For the coefficient of vertical subgrade reaction at the cell bottom, the same value as the coefficient of lateral subgrade reaction at the cell bottom can be used. When the ground consists of the strata of different characteristics, the coefficient of vertical subgrade reaction shall not correspond to the stratum at the cell bottom. However, when there is an extremely soft stratum below the cell bottom, it is necessary to carefully consider the possible effects.

3) Horizontal shear modulus

The horizontal shear modulus at the cell bottom may be calculated by **equation (2.9.23)** using the coefficient of vertical subgrade reaction.

$$k_s = \lambda k_v, \tag{2.9.23}$$

where

k_s : horizontal shear modulus (kN/m^3);

λ : ratio of the horizontal shear modulus to the coefficient of vertical subgrade reaction;

k_v : coefficient of vertical subgrade reaction (kN/m^3).

Past studies suggest the use of λ values in the range of 1/2 to 1/5[77), 78)]. In the case of steel-sheet-pile cellular bulkhead, the value of λ may be set as approximately 1/3.

③ **Calculation of subgrade reaction and wall displacement**

(a) The subgrade reaction acting on the embedded part of the steel-sheet-pile cellular bulkhead and the wall displacement can be calculated on the assumption that the wall subject to the external forces is supported by the horizontal subgrade reaction, vertical subgrade reaction, and horizontal shear reaction at the wall bottom and the vertical frictional resistance along the front and rear of the wall.

(b) Subgrade reaction

1) Horizontal subgrade reaction

Horizontal subgrade reaction may be calculated by **equation (2.9.24)**, but this should not exceed the passive earth pressure intensity calculated in accordance with **Part II, Chapter 4**, **2 Earth Pressure** to prevent the yielding of the ground. The angle of the wall friction used to calculate the passive earth pressure can basically be taken at −15°. **Fig. 2.9.13** illustrates the distribution of the subgrade reaction of a sample case in which the subgrade reaction reaches the passive earth pressure intensity up to a certain depth.

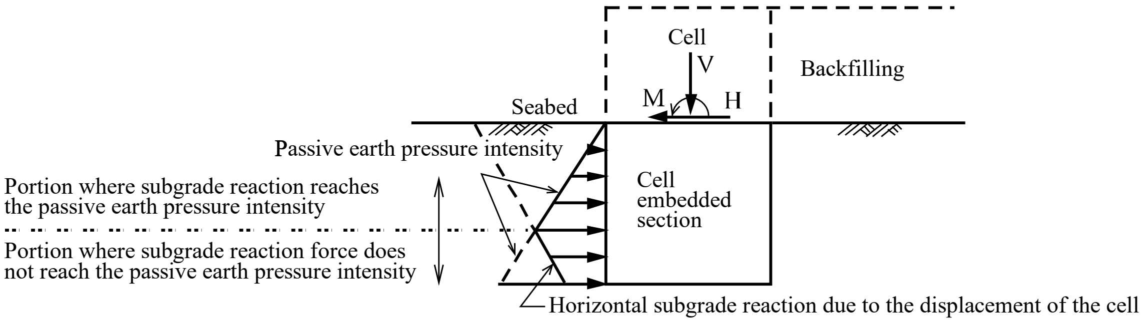


Fig. 2.9.13 Example of Distribution of Horizontal Subgrade Reaction

2) Vertical subgrade reaction

The vertical subgrade reaction at the cell bottom acts in a trapezoidal or triangular distribution. It should be assumed that no tensile stress is generated.

(c) Vertical frictional resistance

It should be assumed that vertical frictional force acts on the front and rear of the wall and the vertical frictional resistance is calculated as the product of the horizontal earth pressure or subgrade reaction and tanδ, where δ denotes the angle of wall friction.

(d) Distribution of external forces

Fig. 2.9.14 shows the standard distribution patterns of the external forces acting on a steel-sheet-pile cellular-bulkhead quaywall.

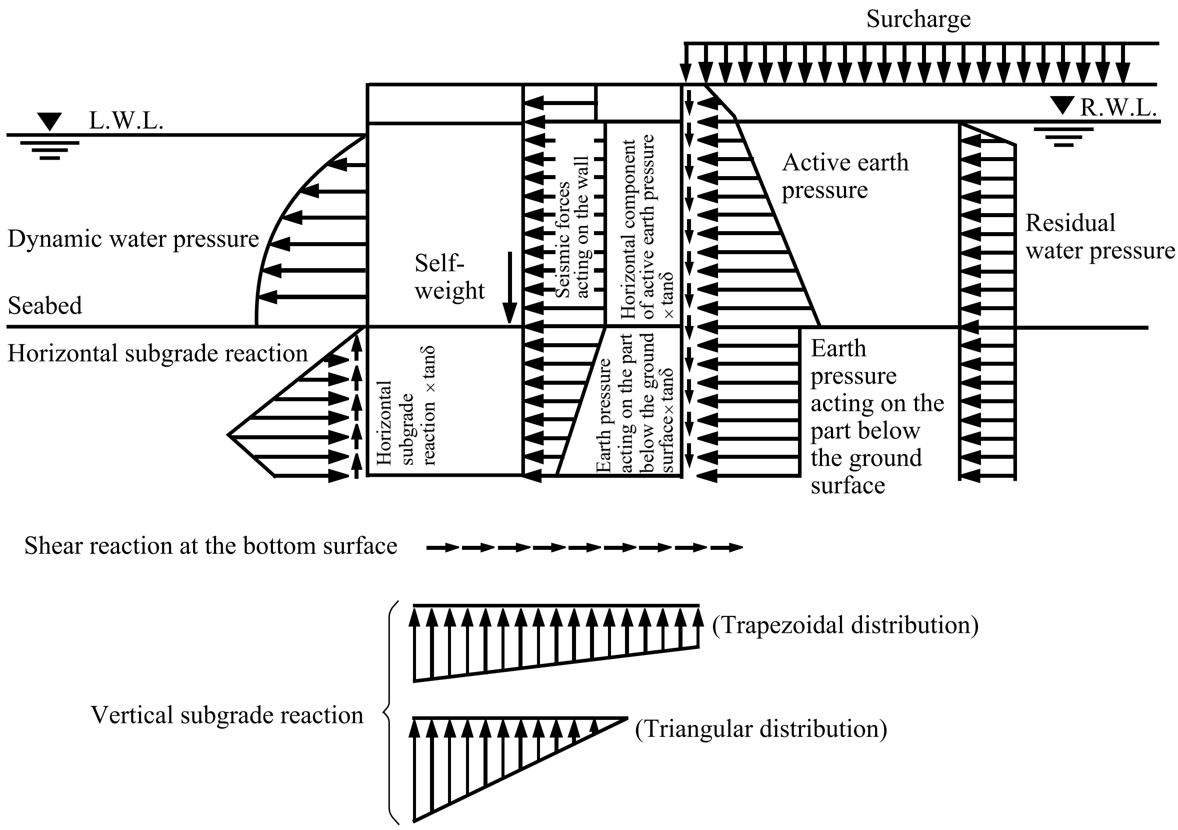


Fig. 2.9.14 Distribution Patterns of External Forces Acting on a Steel Sheet Pile Cellular-bulkhead Quaywall

(e) Displacement modes of cell

As shown in **Fig. 2.9.15**, it is assumed that the cell wall rotates around its center of rotation O, which is horizontally away from the center axis of the cell by distance e and vertically away from the seabed by depth h. When the center of the rotation is located inside the cell, the horizontal subgrade reaction is generated in the rear of the wall for the part below the center of rotation.

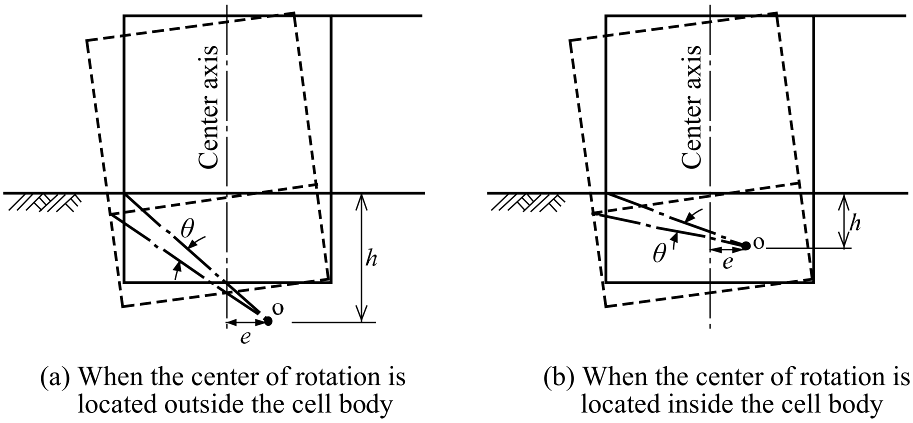


Fig. 2.9.15 Displacement Modes of Cell

(f) Equation for calculating subgrade reaction and wall displacement

Fig. 2.9.16 shows a calculation model for a case in which horizontal force, vertical force, and moment act at the intersection of the ground surface and the center axis of the cell wall and when the ground comprises n layers of soil. The equations for calculating the subgrade reaction and cell wall displacement of the model shown in **Fig. 2.9.16** are as follows. This method does not necessarily accurately calculate the displacement during an earthquake; therefore, caution is needed. In other words, if the embedment length is increased to improve the earthquake-resistant performance, the following methods can overevaluate the deformation in seismic response analysis. For the consistency with seismic response analysis, refer to the related literature[79) 80)].

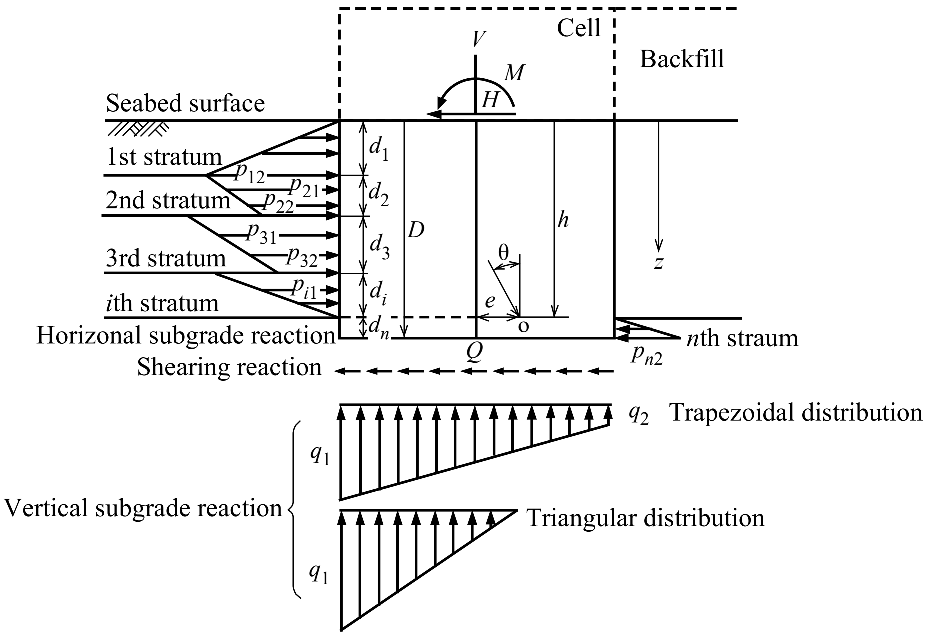


Fig. 2.9.16 Calculation Model

1) When the vertical subgrade reaction acts in a trapezoidal distribution

i) Horizontal subgrade reaction (kN/m^2)

$$\left.\begin{aligned}
p_{12} &= k_{CH_1}\left(h-d_1\right)\theta \\
p_{21} &= k_{CH_2}\left(h-d_1\right)\theta \\
p_{22} &= k_{CH_2}\left(h-d_1-d_2\right)\theta \\
&\vdots \\
p_{i1} &= k_{CH_i}\left(h-\sum_{j=1}^{i-1} d_j\right)\theta \\
p_{i2} &= k_{CH_i}\left(h-\sum_{j=1}^{i} d_j\right)\theta \\
&\vdots \\
p_{n1} &= k_{CH_n}\left(h-\sum_{j=1}^{n-1} d_j\right)\theta \\
p_{n2} &= k_{CH_n}\left(h-\sum_{j=1}^{n} d_j\right)\theta
\end{aligned}\right\}. \qquad \textbf{(2.9.24)}$$

ii) Vertical subgrade reaction (kN/m^2)

$$\left.\begin{aligned} q_1 &= k_v(e+B/2)\theta \\ q_2 &= k_v(e-B/2)\theta \end{aligned}\right\} \tag{2.9.25}$$

iii) Shear reaction force that acts at the wall bottom (kN/m)

$$Q = k_s(h-D)\theta A \tag{2.9.26}$$

iv) Horizontal displacement of the wall (m)

$$\delta = (h-z)\theta \tag{2.9.27}$$

v) Angle of wall rotation (°)

$$\theta = \frac{MK_1 + HK_3}{K_1K_4 - K_2K_3} \tag{2.9.28}$$

vi) Depth of the center of rotation of the wall (m)

$$h = \frac{MK_2 + HK_4}{MK_1 + HK_3} \tag{2.9.29}$$

vii) Distance from the wall center axis to the center of rotation of the wall (m)

$$e = \frac{1}{k_v A}\left\{\frac{V}{\theta} - h\sum_{i=1}^{n} k_{CH_i} d_i \tan|\delta_i| + \sum_{i=1}^{n} k_{CH_i} d_i \left(\sum_{j-1}^{i-1} d + \frac{d_i}{2}\right)\tan|\delta_i|\right\} \tag{2.9.30}$$

where

$$K_1 = \sum_{i=1}^{n} k_{CH_i} d_i + k_s A$$

$$K_2 = \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2}\right)\right\} + k_s AD$$

$$K_3 = \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2} + \frac{B}{2}\tan\delta_i\right)\right\} + k_s AD$$

$$K_4 = \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\frac{d_i^2}{3} + \sum_{j=1}^{i-1} d_j \sum_{j=1}^{i} d_j + \frac{B}{2}\left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2}\right)\tan\delta_i\right)\right\} + k_s AD^2 + \frac{1}{12}k_v A^3$$

The angle of wall friction δ is negative for the strata whose horizontal subgrade reaction acts on the front of the wall and is positive for the strata whose horizontal subgrade reaction acts on the rear of the wall.

2) When the vertical subgrade reaction acts in a triangular distribution

The horizontal subgrade reaction, horizontal wall displacement, angle of rotation, and depth of the center of rotation are expressed in the same form as those in 1).

i) Vertical subgrade reaction (kN/m^2)

$$q_{1k} = k_v\left(e + \frac{B}{2}\right)\theta \tag{2.9.31}$$

ii) Shear reaction that acts at the wall bottom (kN/m)

$$Q_k = k_s(h-D)\theta A', \tag{2.9.32}$$

where

$$A' = e + \frac{B}{2}.$$

iii) Distance between the wall center axis and the center of rotation of the wall (m)

$$e = \sqrt{\frac{2}{k_v}\left\{\frac{V}{\theta} - h\sum k_{CH_i} d_i \tan\left|\delta_i\right| + \sum k_{CH_i} d_i \left(\sum d_j + \frac{d_i}{2}\right)\tan\left|\delta_i\right|\right\}} - \frac{B}{2}, \quad \textbf{(2.9.33)}$$

where

$$K_1 = \sum_{i=1}^{n} k_{CH_i} d_i + k_s A'$$

$$K_2 = \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2}\right)\right\} + k_s A' D$$

$$K_3 = \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2} + \frac{B}{2}\tan\delta_i\right)\right\} + k_s A' D$$

$$K_4 = \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\frac{d_i^2}{3} + \sum_{j=1}^{i-1} d_j \sum_{j=1}^{i} d_j + \frac{B}{2}\left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2}\right)\tan\delta_i\right)\right\} + k_s A' D^2 + \frac{1}{6} k_v A'^2 \left(B - e\right)$$

.

The angle of wall friction δ should be negative for the strata whose horizontal subgrade reaction acts on the front of the wall and is positive for the strata whose horizontal subgrade reaction acts on the rear of the wall.

The notations used in Equations in 1) and 2) are as follows:

- V : vertical force acting on the wall (kN/m);
- H : horizontal force acting on the wall (kN/m);
- M : moment acting on the center of the wall at the level of ground surface (kN·m/m) (provided external forces that act on the wall are those for the unit length in the direction along the face line of wall);
- D : embedded length (m);
- d_i : thickness of each soil layer of the embedded ground (m);
- B : equivalent wall width (m);
- k_{CHi} : coefficient of the lateral subgrade reaction of each layer of the embedded ground (kN/m^3);
- k_v : coefficient of vertical subgrade reaction at wall bottom (kN/m^3);
- k_s : horizontal shear modulus at wall bottom (kN/m^3);
- A : area of the wall bottom per unit length of the wall in the direction along the face line of wall (m^2/m);
- A' : area of the wall bottom per unit length of the wall in the direction along the face line of wall when the value of the vertical subgrade reaction is positive (m^2/m);

④ **Verification of the tilt angle of the wall body**

The allowable value of the tilt angle of the wall body is set by reference to relationships between the amount of deformation of the tops and the amount of damage obtained from earthquake damage reports from the past.[81)] It can be verified that the tilt angle of the wall body calculated by the method described above is equal to or less than the allowable value.

(3) Analysis of Bearing Capacity of Grounds

① For the analysis of the vertical bearing capacity of the grounds at the position of the wall bottom, refer to **Part III, Chapter 2**, **3.2.5 Bearing Capacity for Eccentric and Inclined Actions**.

② When the bearing capacity of shallow foundation for eccentric and inclined load is analyzed by using Bishop's method, the soil above the wall bottom is normally treated as a surcharge.

③ The vertical components of the earth pressure acting on the front and rear of the wall that should be taken into consideration include the following: (a) vertical component of the active earth pressure, (b) friction force due to the earth pressure acting on the embedded section, (c) vertical component of the passive earth pressure, and (d) vertical component of subgrade reaction. The vertical component of earth pressure is considered a positive force when it acts in the same direction as that of the cell weight.

④ When the angle of shear resistance of the soil above the wall bottom is different from that below the wall bottom, it is recommended to use the smaller value as the angle of shear resistance at the wall bottom.

(4) Examination against the Sliding of the Wall

① For the examination of wall stability against sliding, refer to the examination on wall sliding in **Chapter 5**, **2.2 Gravity-Type Quaywalls**.

② The sliding of the wall can be examined using **equation (2.9.34)**. In this equation, γ represents the partial factor for its subscript, and subscripts *d* and *k* stand for the design value and characteristic value, respectively. The values shown in **Table 2.9.2** may be used for the partial factors in the following equation. The mark "－" shown in **Table 2.9.2** indicates that the numerical value in parentheses may be used to simplify the verification calculations.

$$\left.\begin{aligned} & m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\ & S_k = k_s \delta b \\ & R_k = \left(W_k + P_{v_k}\right) \tan \phi_k \end{aligned}\right\}, \qquad \textbf{(2.9.34)}$$

where

W : weight of the wall (kN/m);

P_v : vertical component of earth pressure acting on the front and rear of the wall (kN/m);

ϕ : angle of shear resistance of the soil at wall bottom (°);

k_s : horizontal shear modulus at cell bottom (kN/m^2);

δ : cell bottom displacement (m);

b : distribution span of vertical subgrade reaction (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor by which the resistance term is multiplied;

γ_S : partial factor by which the load term is multiplied;

m : adjustment factor.

Table 2.9.2 Partial Factors to be used for Performance Verification of Sliding of Wall

Object of verification	Partial factor by which the resistance term is multiplied γ_R	Partial factor by which the load term is multiplied γ_S	Adjustment factor m
Sliding of the wall (Permanent state)	– (1.00)	– (1.00)	1.20
Sliding of the wall (Variable state of Level 1 earthquake ground motion)	– (1.00)	– (1.00)	1.00

③ The wall weight can be considered a weight that does not include surcharges and can be calculated by subtracting buoyancy.

④ The vertical components of the earth pressure acting on the front and rear of the wall that should be taken into consideration include the following: (a) vertical component of the active earth pressure, (b) vertical component of the passive earth pressure, and (c) vertical component of subgrade reaction. The vertical component of earth pressure is considered a positive force when it acts in the same direction as that of the wall weight.

⑤ When the angle of shear resistance of the soil above the wall bottom is different from that below the wall bottom, it is recommended to use the smaller value as the angle of shear resistance at the wall bottom.

(5) Verification of the Amount of Displacement during the Action of Earthquake Ground Motion by Using the Finite Element Method

For the modeling of cellular-bulkhead quaywalls in the seismic response analysis, refer to related **References 82)** and **83)**.

(6) Verification of Stability against Circular Slip Failure

① When the ground is soft, the examination of stability against circular slip failure shall be made. In the case of cellular-bulkhead quaywalls, it may be assumed that the wall is a rigid body; therefore, the circular slip surface does not go through the inside of the wall.

② For the examination of stability against circular slip failure, refer to **Chapter 5**, **2.2 Gravity-Type Quaywalls**.

(7) Cell layout

① The cells shall be arranged to make the area equal to the area of the wall with the equivalent width obtained by the examination of shear deformation of the wall body and the calculation of subgrade and wall deformation described in (1) and (2) above.

② It is common to use circular cells when examining cells in plain view. The following points should be considered when arranging circular cells.

(a) The wall with the equivalent wall width may be substituted with circular cells by using **equation (2.9.35)** in such a way that the area of the cross section of the circular cells becomes the same as that of the actual wall (**Fig. 2.9.17**).

$$\left.\begin{aligned} S_1 &= \bigtriangledown ABC2 = \pi R^2 \frac{\theta}{360} 2 = \frac{\pi}{180} R^2 \theta \\ S_2 &= \triangle ACD2 = \frac{1}{2} \overline{AD}\,\overline{CD2} = \frac{R^2}{2} = \sin 2\theta_1 \\ S_3 &= \square CC'D'D = \overline{CD}\,\overline{CC'}2 = 2Rr\cos\theta_1 \sin\frac{\theta_2}{2} \\ S_4 &= \frown CGC' = \bigtriangledown ECGC' - \bigtriangledown ECC' \\ &= \pi r^2 \frac{\theta}{360} - \frac{1}{2} \overline{CC'}\,\overline{EF} = \left(\frac{\pi\theta_2}{360} - \frac{1}{2}\sin\theta_2\right) r^2 \end{aligned}\right\}, \qquad \textbf{(2.9.35)}$$

where

$$\theta + \frac{\theta_2}{2} = 90°$$

$$S = (S_1 + S_2 + S_3 + S_4) \times 2$$

$$\therefore B = \frac{S}{L}$$

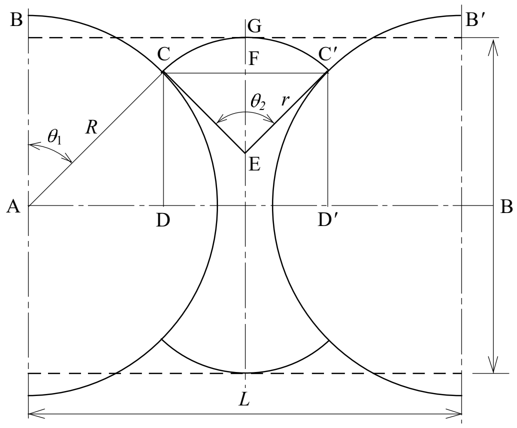


Fig. 2.9.17 Cell Area and Equivalent Wall Width

(b) Cells should be arranged evenly along the total length of the face line of the quaywall wherever possible. In general, it is advisable to set the cell center interval 10% to 15% longer than the cell diameter.

(c) Arcs should be arranged in such a way that they are connected perpendicularly to the wall of the cells. The radius of the arc should be made smaller than that of the cell.

(d) In general, the front tips of the arcs tend to shift forward during and/or after the filling work. Therefore, it is advisable to arrange arcs in such a way that their front surface is located approximately 100 to 150 cm inside the front face line of the cell walls. It is also advisable to arrange cells in such a way that their front face line is located approximately 30 cm inside the design face line of the quaywall.

(8) Analysis of Plate Thickness

① The analysis of the plate thickness of the cells and the arcs is normally performed using **equation (2.9.36)**. In the following equation, subscripts k and d indicate the characteristic value and design value respectively. The values shown in **Table 2.9.3** may be used for the partial factors in the equation. The mark "－" shown in **Table 2.9.3** indicates that the numerical value in parentheses may be used to simplify the verification calculations.

$$\left.\begin{array}{l} m \cdot \dfrac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\ S_k = \dfrac{T_k}{t} \\ R_k = \sigma_{y_k} \end{array}\right\}, \qquad \textbf{(2.9.36)}$$

where

T : tension force acting on the cell (N/mm);

σ_y : yield stress of the cell shell and the arc (N/mm^2);

T : plate thickness of the cell shell and the arc (mm);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor by which the resistance term is multiplied;

γ_S : partial factor by which the load term is multiplied;

Table 2.9.3 Partial Factors to be used for Performance Verification of Plate Thickness of Cells and Arcs

Object of verification	Partial factor by which the resistance term is multiplied γ_R	Partial factor by which the load term is multiplied γ_S	Adjustment factor m
Material yield of cells and arcs (Permanent state)	– (1.00)	– (1.00)	1.67
Material yield of cells and arcs (Variable state of Level 1 earthquake ground motion)	– (1.00)	– (1.00)	1.12

Furthermore, the tensile force acting on the cell may be calculated using **equation (2.9.37).**

$$T_k = \left\{ \left(w_{0_k} H_0' + q_k\right) K_{i_x} + \rho_0 g h_{w_k} \right\} R, \tag{2.9.37}$$

where

T : tensile force acting on the cell (kN/m);

K_i : coefficient of earth pressure of filling;

w_0 : equivalent unit weight of filling (kN/m^3);

$\rho_0 g h_w$: buoyancy due to the difference in water level within the cell and on the front surface (kN/m);

H_0' : equivalent wall height (m);

R : radius of cell (m);

q : surcharge (kN/m^2);

② The equivalent wall height H_0' can be calculated using **equation (2.9.18)** for the calculation of the resistant moment in (1) above.

③ When materials with a large angle of shear resistance, such as gravel, are used for the filling or when no compaction is performed, the characteristic value of the coefficient of earth pressure of filling for the cells can be normally set as 0.6. When the filling is to be compacted, tanϕ can be used as the characteristic value of the coefficient of the earth pressure of filling because the internal pressure of the cell and the angle of shear resistance of the filling become larger. The characteristic value of the coefficient of earth pressure for filling for the arcs can be taken at 1/2tanϕ. This setting is based on the following knowledge obtained from the results of the model tests and field measurements of embedded-type steel-plate cellular blocks[84)]: when the ratio of the cell center interval to the cell diameter is 1.5 or less, the coefficient of earth pressure for filling for the arcs is 1/2 or less of that of the cells.

④ In determining the plate thickness of the cell shells and the arcs of the steel-plate cellular-bulkhead quaywalls, the fabrication, construction, and maintenance aspects must be considered sufficiently. If a corrosion allowance is considered for the cell shells and arcs, the corrosion allowance shall be added to the plate thickness obtained from **equation (2.9.36)** to obtain the plate thickness. **equation (2.9.38)** has been proposed as a method of obtaining the plate thickness of the cell shells necessary for the stresses during driving from tests on the buckling of cylindrical cells and from the construction experience of the past.[85)]

$$t \geq 0.032\left(R\overline{N}D'/E\right)^{0.5}, \tag{2.9.38}$$

where

t : plate thickness of the cell shell (mm);

E : young's modulus of the steel material (kN/mm^2);

R : radius of the cell shell (cm);

$\bar{N}$: average N value of the soils into which the cell is driven;

D' : depth of drive of the cell (cm).

Furthermore, the minimum plate thickness of the cell shell for which there is experience of driving in the past is 8 mm; therefore, it is desirable that the minimum plate thickness is approximately 8 mm.

(9) Verification of T-Shaped Sheet Piles of the Steel-Sheet-Pile Cellular-Bulkhead Quaywalls

① Normally, cells and arcs are connected by using T-shaped sheet piles. A T-shaped sheet pile is a deformed sheet pile to join the cell to arcs (see **Fig. 2.9.18**).

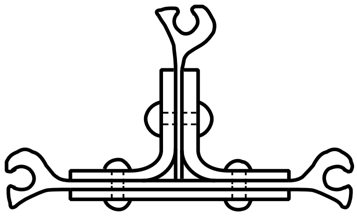


Fig. 2.9.18 T–Shaped Sheet Pile

② The structure of a T-shaped sheet pile shall have sufficient safety against the sheet-pile interlock tension acting on cells and arcs. The standard structures of T-shaped sheet pile are shown in **Figs. 2.9.19** and **2.9.20**.

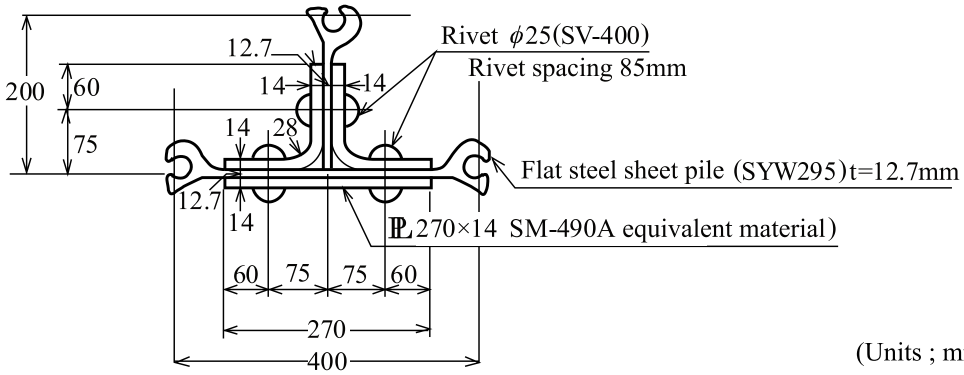


Fig. 2.9.19 Standard Cross Section of T-shaped Sheet Pile for Rivet Connection with Rivet Intervals of 85 mm

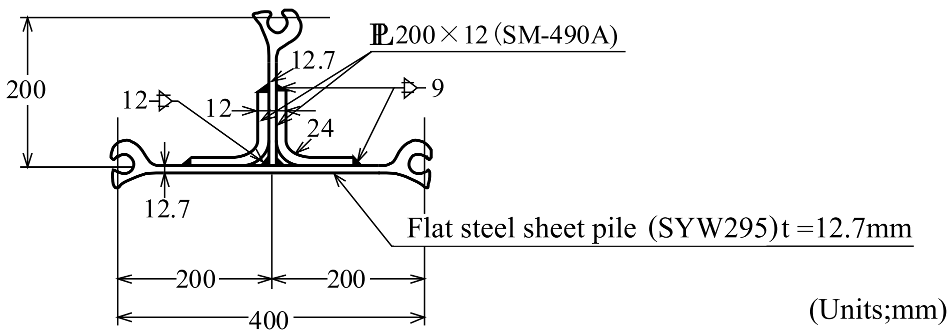


Fig. 2.9.20. Standard Cross Section of T-shaped Sheet Pile for Welding Connection

③ The strength of the cross sections shown in **Figs. 2.9.19** and **2.9.20** has been confirmed by a breaking test where the tensile strength of the joint of the sheet pile in a cell is 3,900 kN/m, and the arc diameter is 2/3 or less of the cell with a tensile strength of 2,600 kN/m. The rivet and welding joints for tests were made in a workshop.

④ **Figs. 2.9.19** and **2.9.20** show the standard cross sections for a flat steel sheet pile with a thickness *t* = 12.7 mm.

⑤ The strength of the cross sections shown in **Figs. 2.9.21** and **2.9.22** has been confirmed by a breaking test where the tensile strength of the joint of the sheet pile in a cell is 5,900 kN/m, and the arc diameter is 2/3 or less of the cell.

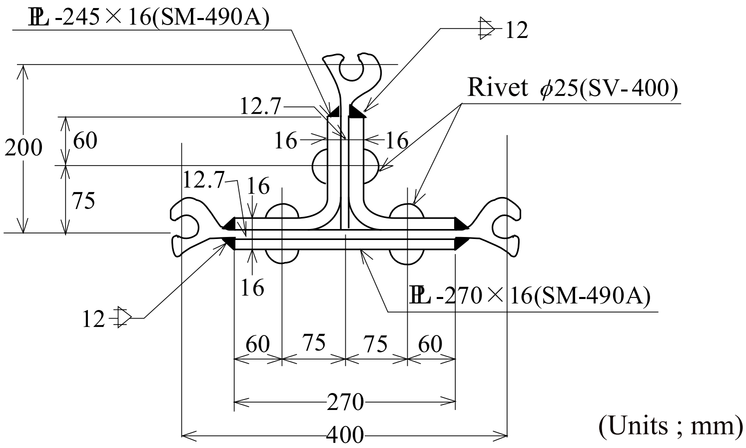


Fig. 2.9.21 Cross Section of T-shaped Sheet Pile for Rivet and Welding Connection with Rivet Intervals of 85 mm

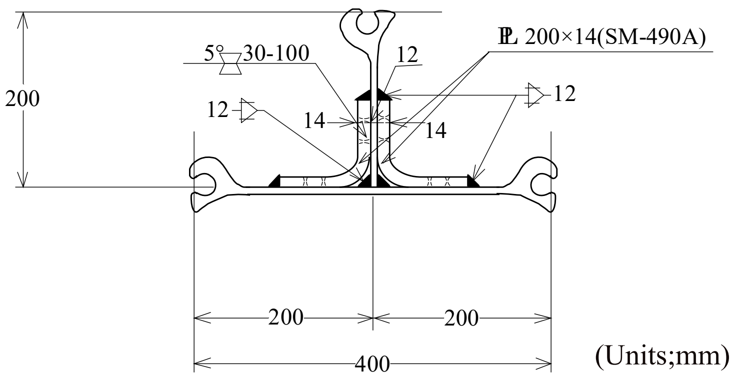


Fig. 2.9.22 Cross Section of T-shaped Sheet Pile for Welding Connection

(10) Joints and Stiffeners for Steel Plate Cellular-Bulkhead Quaywalls

① The joints of cells and arcs shall have a safe structure that resists the maximum horizontal tension acting on the arcs. Cell shells and arcs shall have safe structures that resist stresses that can occur during fabrication, transportation, and construction. The joints of cells and arcs must have a structure that is safe enough to resist tensions acting on the arcs, does not interfere with driving of arcs, and prevents filling and backfilling materials from leaking out of the arcs.

② **Fig. 2.9.23** shows an ordinary shape of a joint.

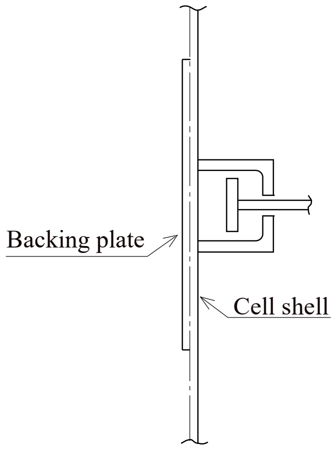


Fig. 2.9.23 Example of Joint Structure

③ To protect cell shells and arcs from stresses that can occur during fabrication, transportation, and construction, it is advisable to equip them with vertical stiffeners (longitudinal ribs), horizontal stiffeners (lateral ribs), and stiffeners that add strength to the top and bottom ends.

2.9.5 Performance Verification of Structural Members

(1) Performance Verification of Superstructure Bearing Piles

① Piles that support the superstructure may be verified for performance as piles subjected to a vertical force, a horizontal force, or a moment.

② Superstructures shall normally be supported by piles alone.

③ For actions on superstructures, refer to **Chapter 5**, **2.8 Quaywalls with Relieving Platforms**.

④ Normally, a horizontal force acting on a superstructure is not directly conveyed to the filling but is conveyed to piles first. It is then conveyed to the filling as a horizontal resistance of the piles. Therefore, piles that support the superstructure may be verified as piles subjected to a vertical force, a horizontal force, or a moment.

⑤ It is common to use vertical piles for supporting a superstructure. The pile-head moment may act on pile heads depending on the degree of constraint to the superstructure. In the performance verification of vertical piles, the superstructure bottom may be considered the ground surface compared with relieving platform piles for which the failure surface under active earth pressure is considered the ground surface.

⑥ For the performance verification of piles, refer to **Part III, Chapter 2**, **3.4 Pile Foundations**.

(2) Performance Verification of Superstructures

① Calculations for the arrangement of reinforcing bars must be made appropriately for the following parts of a superstructure:

(a) Upright walls

(b) Slabs

② The upright walls of a superstructure may be verified for performance as cantilevers supported by slabs and subjected to actions of earth pressure and residual water pressure.

③ Superstructure joints should be positioned at the center of a cell.

④ The rear end of a superstructure should be extended to approximately 1.0 m behind T-shaped sheet piles.

⑤ For the performance verification of slabs, refer to **Part III, Chapter 5**, **2.8 Quaywalls with Relieving Platforms**, excluding the descriptions about the horizontal force transmitted from sheet piles.

⑥ It is advisable to apply concrete jacketing to the upper part of the sheet piles at the front of the cells to prevent sand leakage and to protect corrosion.

2.10 Placement-Type Cellular-Bulkhead Quaywalls

[Public Notice] (Performance Criteria of Cellular-Bulkhead Quaywalls)

Article 52

2 In addition to the provisions of the preceding paragraph, for the performance criteria of placement-type cellular-bulkhead quaywalls, the risk of occurrence of overturning under the variable situation, in which the dominating action is Level 1 earthquake ground motion, shall be equal to or less than the threshold level.

[Interpretation]

11. Mooring Facilities

(6) Performance Criteria of Cellular-Bulkhead Quaywalls

② **Placement-type cellular-bulkhead quaywalls** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 52, Paragraph 2 of the Public Notice

(a) The performance criteria and interpretation of embedded-type cellular-bulkhead quaywalls shall apply correspondingly to placement-type cellular-bulkhead quaywalls, and the following provision shall apply to placement-type cellular-bulkhead quaywalls.

(b) Serviceability shall be the required performance for placement-type cellular-bulkhead quaywalls under a variable situation in which the dominating action is Level 1 earthquake ground motions. The performance verification items for those actions and the standard indexes for setting limit values shall be in accordance with **Attached Tables 11-17**.

Attached Table 11-17 Performance Verification Items and Standard Indexes for Setting Limit Values for Placement-type Cellular-bulkhead Quaywalls under Different Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	52	2	—	Serviceability	Variable	Level 1 earthquake ground motion	Self-weight, earth pressure, water pressure. surcharges	Overturning of wall	Action-to-resistance ratio for overturning

(c) In addition to these provisions, provisions regarding Article 22, Paragraph 3 (Scouring and Sand Washing Out) and Article 28 (Performance Criteria for Armor Stones and Blocks) of the Public Notice and their interpretations shall apply as needed.

2.10.1 General

(1) This section is applicable to the performance verification of placement-type cellular-bulkhead quaywalls. The performance verification method described in this section may also be applied to the performance verification of revetments using this structure.

(2) Placement-type cellular-bulkhead quaywalls are cellular-bulkhead quaywalls without an embedded section. In many cases, these quaywalls are constructed on strong foundation ground whose bearing capacity is considered sufficiently large or on ground that has been improved to have sufficient bearing capacity.

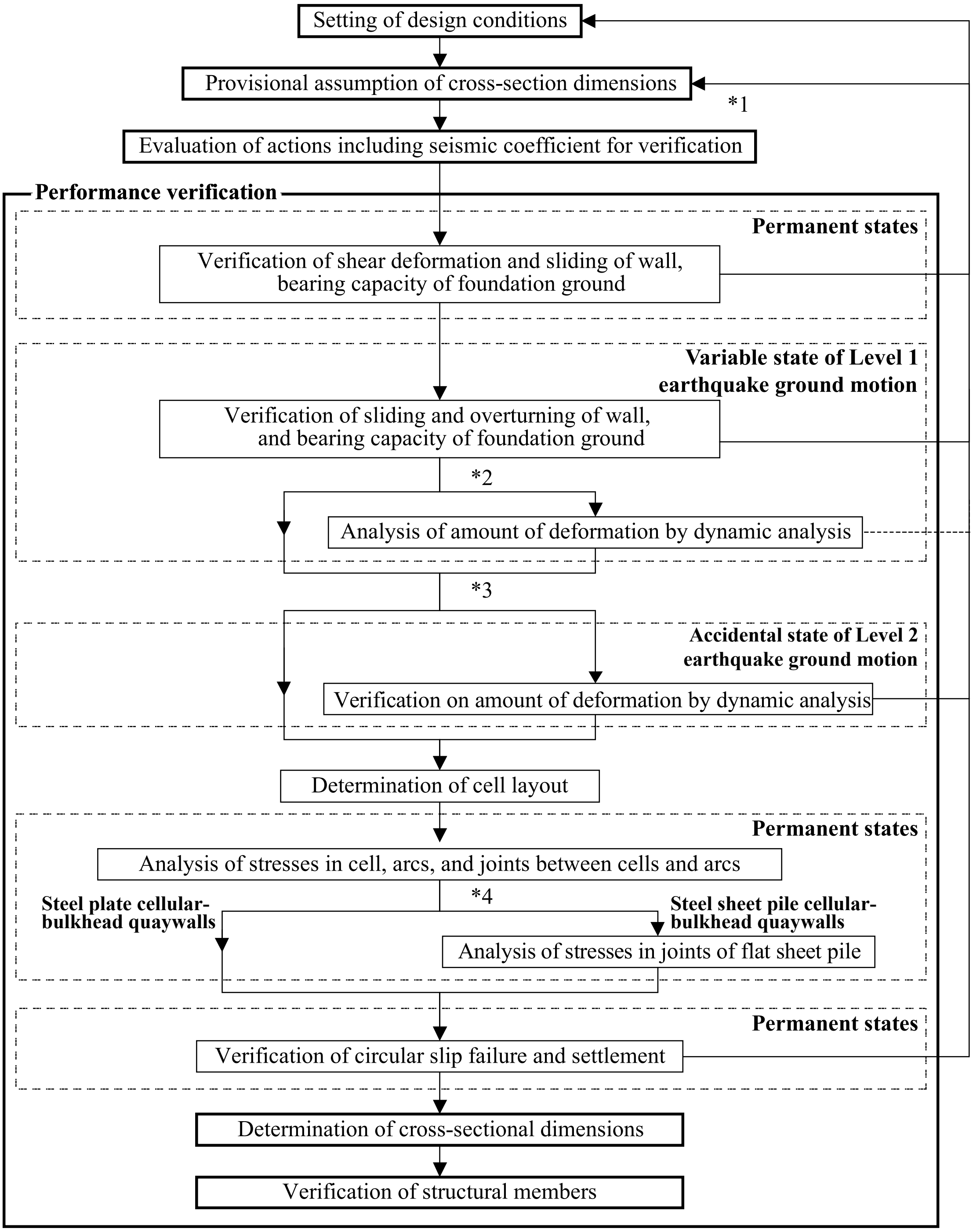
(3) The approaches given in **Part III, Chapter 5**, **2.10.2 Actions** and **Part III, Chapter 5**, **2.10.4 Performance Verification** are simplified approaches, and it is necessary to be careful when adopting these approaches. Highly precise methods (model tests and numerical analysis techniques that can simulate mechanisms) should be used for verification.

(4) **Fig. 2.10.1** shows an example of the sequence of performance verification of placement-type cellular-bulkhead quaywalls. Note that the evaluation of the effects of liquefaction, settlement, and other phenomena caused by earthquake ground motions is not shown in **Fig. 2.10.1**. Therefore, for liquefaction, for example, it is necessary to determine whether there is liquefaction and to consider measures against it appropriately by referring to **Part II, Chapter 7 Ground Liquefaction**. It is possible to verify the variable situation associated with Level 1 earthquake ground motions by using the seismic coefficient method. However, for high-earthquake-resistance facilities, it is desirable to analyze the amount of deformation by using the nonlinear seismic response analysis in consideration of dynamic interaction between the ground and a structure. For placement-type cellular-bulkhead quaywalls that are not categorized as high earthquake-resistance facilities, the verification of the accidental situation associated with Level 2 earthquake ground motions can be omitted.

(5) For other general matters, refer to **Part III, Chapter 5**, **2.9.1 General**.

2.10.2 Actions

For the action on placement-type cellular-bulkhead quaywalls, refer to **Part III, Chapter 5**, **2.9 Embedded-type Cellular-bulkhead Quaywalls**. The characteristic value of seismic coefficient for verification used in the performance verification of placement-type cellular-bulkhead quaywalls under variable situations associated with Level 1 earthquake ground motion shall be appropriately calculated by taking into consideration the structural characteristics. For the purpose of convenience, the characteristic value of seismic coefficient for the verification of placement-type cellular-bulkhead quaywalls may be calculated in accordance with **Part III, Chapter 5**, **2.2 Gravity-type Quaywalls**.



*1: The evaluation of the effect of liquefaction is not shown; therefore, this must be separately considered.
*2: Analysis of the amount of deformation due to Level 1 earthquake ground motion may be performed by dynamic analysis when necessary.
For high-earthquake-resistance facilities, the analysis of the amount of deformation due to Level 1 earthquake ground motion should also be performed by dynamic analysis.
*3: For high-earthquake-resistance facilities, verification is performed for Level 2 earthquake ground motion.
*4: For steel-sheet-pile cellular-bulkhead quaywalls, verification is performed for the joints of a flat sheet pile.

Fig. 2.10.1 Example of the Sequence of Performance Verification of Placement-type Cellular-bulkhead Quaywalls

2.10.3 Setting of Cross-Sectional Dimensions

The width of the wall structure used in performance verification may be the equivalent wall width, which is an imaginary wall width obtained by replacing cells and arcs with a rectangular wall structure. For the equivalent wall width, refer to **Part III, Chapter 5, 2.9 Embedded-type Cellular-bulkhead Quaywalls**.

2.10.4 Performance Verification

(1) Examination of the Shear Deformation of the Wall

① The examination of the shear deformation of the wall body shall be made in accordance with the performance verification methods described in **Part III, Chapter 5**: **2.9 Embedded-type Cellular-bulkhead Quaywalls**. The resistant moment shall be calculated appropriately in consideration of the structural characteristics of the cellular-bulkhead and the deformation of the wall. The deformation moment to be used in the verification shall be the moment at the seabed due to external forces acting on the wall body above the seabed, including active earth pressure and residual water pressure.

② When the deformation of the steel plate cells is not allowed, i.e., when the horizontal displacement of the cell top is approximately less than 0.5% of the cell height, the resistant moment against deformation can be normally calculated using **equations (2.10.1)** and **(2.10.2)**. Subscript k in the following equations means the characteristic value.

$$M_{rd_k} = \frac{1}{6} w_{0_k} H_d'^3 R \qquad \textbf{(2.10.1)}$$

$$R = \nu^2 (3 - \nu \cos\phi_k) \sin\phi_k \qquad \textbf{(2.10.2)}$$

where

M_{rd} : resistant moment of the cell (kN･m/m);

H'_d : equivalent wall height used in the examination of cell deformation (m);

w_0 : equivalent unit weight of filling (kN/m^3) (normally, w_0 = 10 kN/m^3);

v : ratio of the equivalent wall width to the equivalent wall height used in the examination of cell deformation $v = B/H_d'$

B : equivalent wall width (m);

ϕ : angle of shear resistance of filling (°).

③ In the calculation of resistant moment, the equivalent wall height of the cell H'_d is calculated by **equation (2.10.3)**. The height H'_d is the height above the seabed.

$$H'_d = \left(w'_k / w_{0_k}\right) H_w + \left(w_{tk} / w_{0_k}\right)\left(H_d - H_w\right) \qquad \textbf{(2.10.3)}$$

where

H'_d : height from the seabed to the top of the quaywall (m);

H_w : height from the seabed to the residual water level (m);

w_t : wet unit weight of the filling above the residual water level (kN/m^3);

w' : saturated unit weight of saturated filling (kN/m^3);

w_0 : equivalent unit weight of the filling (kN/m^3) (normally, w_0= 10 kN/m^3).

In the calculation of the equivalent wall height H'_d, a surcharge may be ignored similar to the case of resistant moment calculation discussed for the performance verification in **Part III, Chapter 5**, **2.9 Embedded-type Cellular-bulkhead Quaywalls**.

④ When the filling material can be regarded as uniform, the height H_d of the quaywall top above the seabed can be used in place of the equivalent wall height H'_d of **equation (2.10.1)**.

(2) Examination of the Sliding of the Wall

For the examination of sliding, refer to Part III, Chapter 5, 2.9 Embedded-type Cellular-bulkhead Quaywalls.

(3) Examination of the Overturning of the Wall

① In the calculations to examine the stability of a cell wall body against overturning, the stability of the cell shall be examined against the external forces acting above the cell bottom, including earth pressure, residual water pressure, and earthquake ground motion.

② For the performance verification for overturning, **equation (2.10.4)** can normally be used. In the equation, subscripts k means the characteristic and d means design values, respectively. The values shown in **Table 2.10.1** may be used for the partial factors in the following equation. The mark "—" shown in **Table 2.10.1** means that the numerical value in parentheses may be used to simplify the verification calculations.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$

$$S_k = M_{dk} \qquad \textbf{(2.10.4)}$$

$$R_k = M_{rdk}$$

where

M_{rd} : resistant moment against the overturning of the steel plate cell (kN·m/m);

M_d : deformation moment of the cell bottom (kN·m/m).

Table 2.10.1 Partial Factors to be Used for Performance Verification of Overturning of Wall

Object of verification	Partial factor by which the resistance term is multiplied γ_R	Partial factor by which the load term is multiplied γ_S	Adjustment factor m
Overturning of the wall (Variable state of Level 1 earthquake ground motion)	– (1.00)	– (1.00)	1.10

③ The resistant moment of the cell against overturning can be calculated using **equations (2.10.5)** and **(2.10.6)**.

$$M_{rd_k} = \frac{1}{6} w_{0_k} H_d'^3 R_t \qquad \textbf{(2.10.5)}$$

$$\left.\begin{aligned} R_t &= \nu'^2 (3 - \nu' \cos\phi_k)\sin\phi_k + 3(\alpha^2 + \beta^2) + 6\nu\beta \\ \alpha &= K_a \tan\delta_k \\ \beta &= K_a \tan\delta_k (\nu'/2)(4 - \nu' \cos\phi_k)\tan\phi_k \tan\delta_k \\ \nu' &= \nu - (\alpha + \beta) \end{aligned}\right\} \qquad \textbf{(2.10.6)}$$

where

M_{rd} : resistant moment of the steel cell plate against overturning (kN·m/m);

H' : equivalent wall height of the cell to obtain the resistant moment against overturning (m);

ν : ratio of the equivalent wall width to the equivalent wall height of the cell $\nu = B/H'$

B : equivalent wall width of the cell (m);

δ : angle of wall friction of the filling material (°) (normally, δ = 15° is used);

K_a : coefficient of the active earth pressure of the filling material.

④ The equivalent wall height H' used to calculate the resistant moment against the overturning of the cell can be calculated using **equation (2.10.7).**

$$H' = \left(w'_k / w_{0_k}\right) H_w + \left(w_{tk} / w_{0_k}\right)\left(H_d - H_w\right), \qquad \textbf{(2.10.7)}$$

where

H' : equivalent wall height of the cell used to calculate the resistant moment against overturning (m);

H_d : distance from the cell bottom to the top of the quaywall (m);

H_w : distance from the cell bottom to the residual water level (m);

w : wet unit weight of the filling above the residual water level (kN/m^3);

w' : saturated unit weight of saturated filling (kN/m^3);

w_0 : equivalent unit weight of filling (kN/m^3); normally, w_0= 10 kN/m^3.

⑤ In general, the filling of a cell used as a mooring facility is not uniform because the major portion of such filling is under the water and is subjected to buoyancy. Therefore, the equivalent wall height is used here in the calculation of the resistant moment of the cell against deformation. When the filling material can be considered uniform, the total wall height of the cell H may be used in the same calculation in place of the equivalent wall height H' of **equation (2.10.7)**.

Although the actions of the filling against overturning are not uniform,[85)] given that the main part of the filling's resistance is the hanging effect, the margin of error is minimal, and safety is ensured even when the ratio of the equivalent wall width to the equivalent wall height v is used as that in **equation (2.10.6)**. In this case, a surcharge can be ignored.

⑥ The overturning moment is the moment at the cell bottom due to the external forces acting above the bottom. The equivalent wall height of the cell H' used in the calculation of the resistant moment should be the height above the cell bottom.

(4) Examination of Bearing Capacity on Cell Front Toe

① The maximum front toe reaction force on the cell shell front toe shall be calculated appropriately in consideration of the effect of the filling material acting on the front wall of the cell.

② The maximum front toe reaction force on the cell shell front toe may be obtained from **equation (2.10.8)**. Subscript k means the characteristic value.

$$V_{t_k} = \frac{1}{2} w_k H^2 \tan^2 \phi_k \qquad \textbf{(2.10.8)}$$

where

V_t : maximum front toe reaction force on the cell shell front toe (kN/m);

w : unit weight of filling (kN/m^3);

H : total wall height of the cell (m);

ϕ : angle of the shearing resistance of the filling (°).

Equation (2.10.8) calculates the weight of the filling weighing down on the front wall, with the product of the coefficient of earth pressure of the filling and the wall surface friction coefficient given by $\tan^2 \phi$. Therefore, when the filling is not uniform, it is necessary to perform the calculation for the same domain as the earth pressure calculation.

③ The wall height H should normally be considered the height of the cell top above the cell bottom. However, when the superstructure of the cell is supported by foundation piles, it may be considered the height of the bottom of the superstructure above the cell bottom.

④ **Equation (2.10.8)** represents the cell shell front toe reaction force when the overturning moment is roughly equal to the overturning resistant moment of **equation (2.10.5)**. Without the occurrence of overturning, the reaction force is smaller than the value obtained from **equation (2.10.8)**. According to a model test, the maximum front toe reaction force V_t is nearly proportional to the overturning moment.[86] Therefore, a reaction force without the occurrence of overturning should be calculated using **equation (2.10.9)**.

$$V_k = Vt_k (M_k / M_{r0_k}), \qquad \textbf{(2.10.9)}$$

where

V : front toe reaction force of the cell shell corresponding to overturning moment M (kN/m);

M : overturning moment (kN·m/m);

M_{r0} : resistant moment against overturning (kN·m/m).

Hence, the use of a larger cell shell radius makes the cell safer against overturning by increasing the resistant moment M_{r0} while reducing the front toe reaction force V.

⑤ For the bearing capacity of the ground, refer to the bearing capacity in **Part III, Chapter 2**, **3.2 Shallow Spread Foundations**.

⑥ When providing a footing at the cell shell bottom to reduce the reaction force of the foundation, it is favorable to locate the footing outside the cell shell.[85]

(5) Examination of Plate Thickness

① Examination of the plate thickness of the cells and arcs may be performed in accordance with the examination of plate thickness given for the performance verification in **Part III, Chapter 5, 2.9 Embedded-type Performance Verification of Cellular-bulkhead Quaywalls**.

② Forces acting on the cell shell include the horizontal tension due to filling, the compressive stress due to subgrade reaction near the front toe bottom, and the shear stress due to deformation moment near the side wall. However, the compressive stress due to the front toe reaction force does not pose a problem for stability because it is much smaller than the tensile stress and because there is a difference in the point where the maximum stress occurs. Care should be taken when providing a footing outside the cell bottom because a significantly large stress will occur owing to the bending moment. In a model test, no buckling occurred at the front toe bottom before the occurrence of overturning failure[85]. According to the results of an experiment on a cell shell with a small thickness, no local buckling occurred near the bulkhead, but a significant shear stress occurred at the side wall at the time of overturning because the resistance against overturning was dominated by the hanging effect of filling. However, the hanging effect of filling was not large before the occurrence of deformation. Therefore, it is considered that the shear stress of the cell shell is relatively small. The effect of the shear stress may be ignored because it does not matter if the stress at the cell shell exceeds the limit value at the time of overturning.

③ From the point of view of cell shell stiffness and corrosion, a minimum cell shell thickness should be more than 6 mm is necessary.

2.10.5 Performance Verification of Structural Members

For the performance verification of the structural members of placement-type cellular-bulkhead quaywalls, refer to the performance verification of the structural members in **Part III, Chapter 5**, **2.9 Embedded-type Cellular-bulkhead Quaywalls**.

2.11 Upright Wave-Absorbing-Type Quaywalls

2.11.1 General

(1) This section is applicable to upright wave-absorbing-type quaywalls, but it may also be applied to the performance verification of revetments.

(2) The upright wave-absorbing-type quaywall shall be structured so as to have the required capability of wave energy dissipation and shall be located at strategic positions to enhance the calmness within the harbor.

(3) Waves within a harbor are the result of the superposition of the waves entering the harbor through the breakwater openings, transmitted waves over the breakwaters, wind generated waves within the harbor, and reflected waves inside the harbor. By using the quaywalls of the wave-absorbing type, the reflection coefficient can be reduced to 0.3 to 0.6 from 0.7 to 1.0 of the upright walls. To improve the harbor calmness, it is important to design the alignments of breakwaters in a careful manner. The suppression of reflected waves through the provision of wave energy absorbing structures within the harbor is also an effective means of improving calmness. It is effective to apply energy absorbing structures to reflecting surfaces, particularly those that directly receive waves entering the harbor through the breakwater openings and those that receive waves coming from many directions.

(4) When it is necessary to improve harbor calmness for small craft facilities, energy absorbing structures should be installed in mooring facilities for small crafts and facilities that direct reflected waves toward such areas.

(5) Determination of Structural Type

① Upright wave-absorbing type quaywalls include upright wave-absorbing block type and upright wave-absorbing caisson type. An appropriate type of structure shall be selected depending on the scale of the mooring facility and wave conditions at the location.

② Upright wave-absorbing block type quaywalls are constructed by stacking layers of various shapes of wave-absorbing blocks. This type is normally used to build relatively small quaywalls. The quaywall width is determined by stability calculation as a gravity-type quaywall.

③ Upright wave-absorbing caisson-type quaywalls include slit-wall caisson type and perforated-wall caisson type. This type is normally used to build large quaywalls. The wave-absorbing performance can be enhanced by optimizing the aperture rate of the front slit wall, the water chamber width, and the others parameters for the given wave conditions.

④ An upright wave-absorbing-type quaywall normally consists of a permeable front wall, a water chamber, and an impermeable rear wall. To improve harbor calmness, this type of quaywall is designed to reduce the reflection rate via the energy losses mainly caused by the horizontal jet flow of water passing through the front wall, the resistance due to roughness inside the structure, and the occurrence of phase difference. The wave conditions to be considered in performance verification may include extreme waves for the examination of the stability of a facility and regular or extreme waves for the examination of the wave-absorbing performance.

⑤ The reflection coefficient is preferably determined by means of a hydraulic model test whenever possible, but it may also be determined in accordance with **[Facilities]** in **Chapter 4: 3.4 Gravity-Type Breakwaters (Breakwater Covered with Wave-dissipating Blocks)** and **[Facilities]** in **Chapter 4**: **3.5 Gravity-Type Breakwaters (Upright Wave-absorbing Block Breakwaters)**. **Figs. 2.11.1** and **2.11.2** show an example of the results of the model tests on slit-wall caissons and perforated-wall caissons with round holes[87) 88) 89)].

⑥ It is recommended that the crown height of the wave-dissipating work of an upright wave-absorbing block-type quaywall is set as high as 0.5 times the significant wave height or more above the monthly mean highest water level, and the bottom height of the wave-dissipating work is set twice as deep as the significant wave height or more below the monthly mean lowest water level.

⑦ The area of wave-dissipating works for upright wave-absorbing caisson-type quaywalls may be determined in the same way as that for upright wave-absorbing block-type quaywalls. It is advisable to examine the effects of ceiling slabs and air holes on the reflection rate by conducting a hydraulic model test.

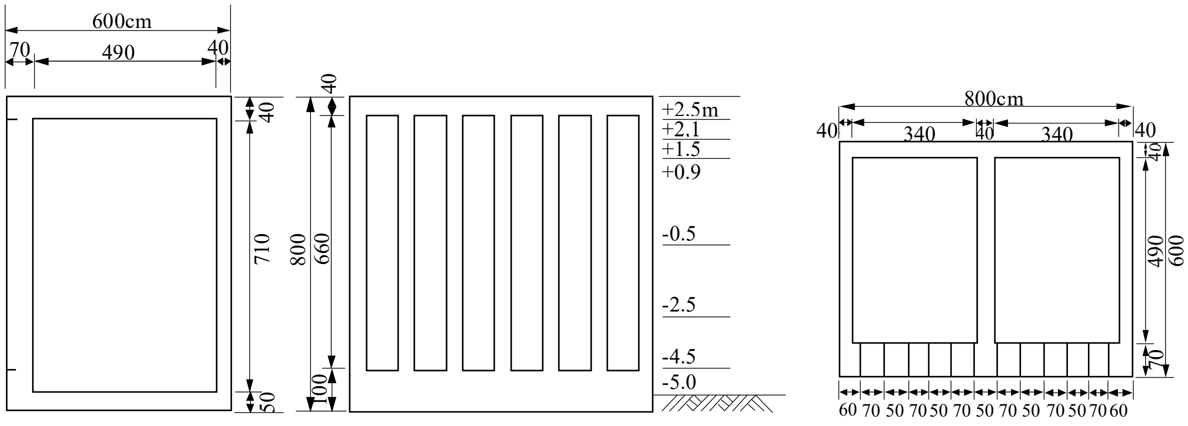


Fig. 2.11.1(a) Slit-wall Caisson Type Wave-absorbing Quaywall

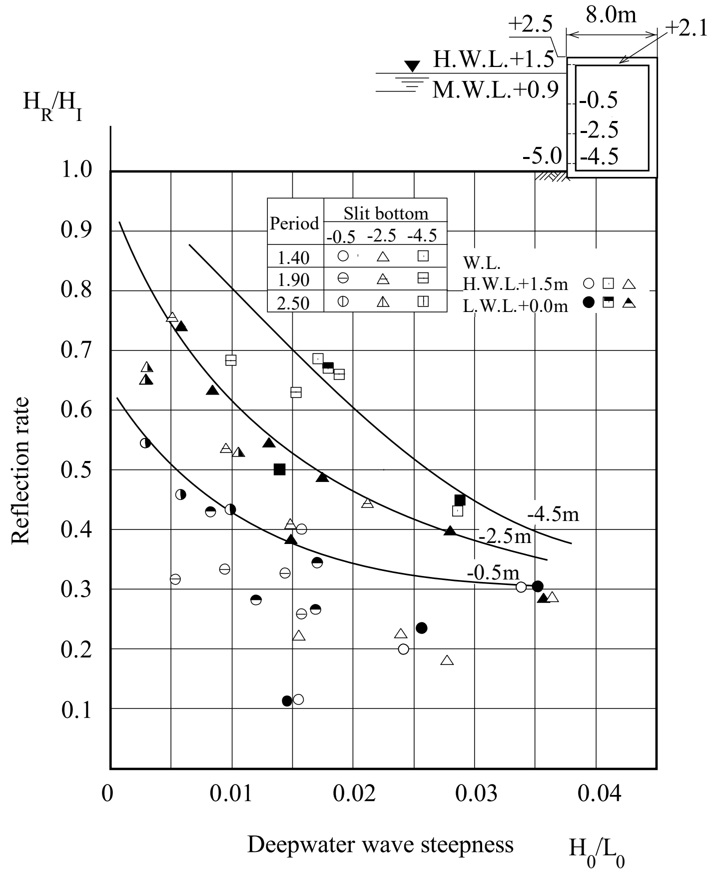


Fig. 2.11.1(b) Slit-wall Caisson Type Wave-absorbing Quaywall
(Relationship between Reflection Rate and Slit Length without Filling Blocks)

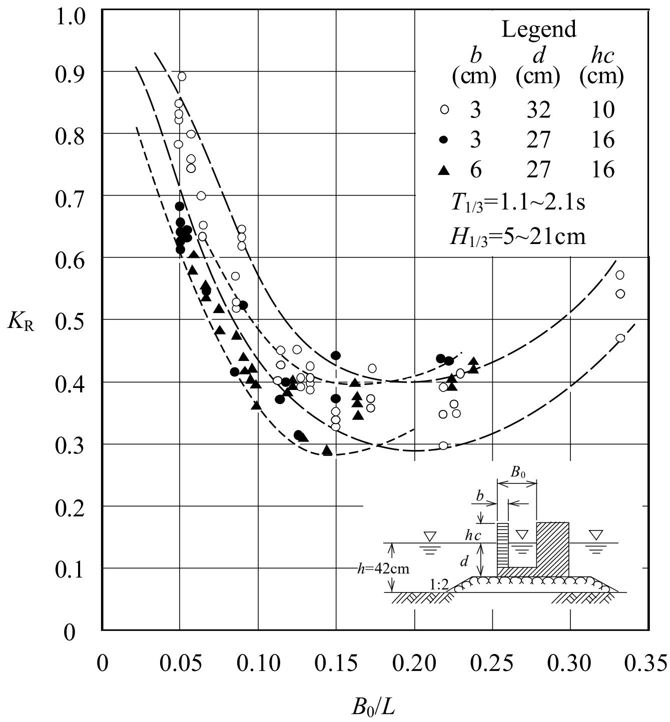


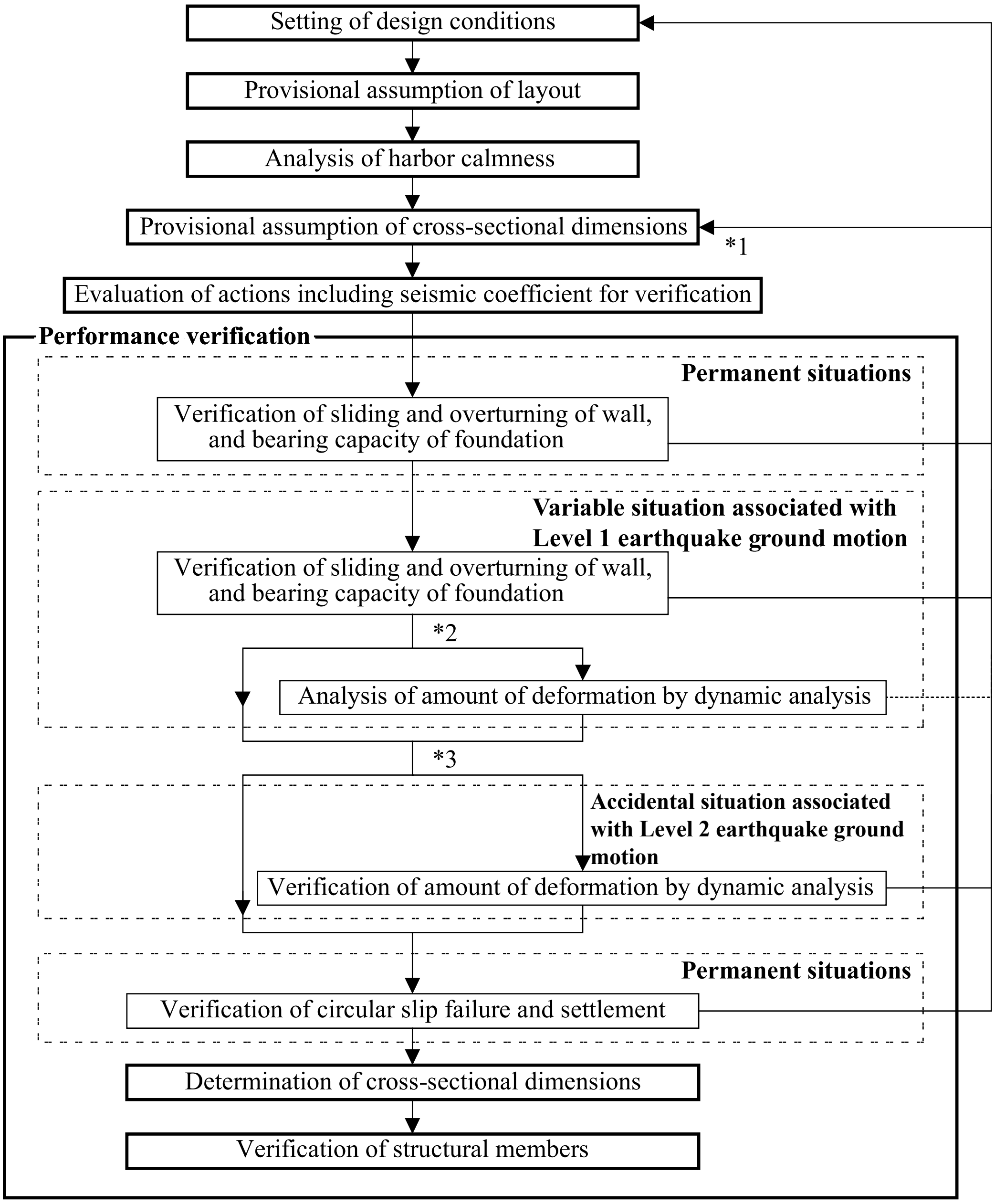
Fig. 2.11.2 Random Wave Reflection Rate of Perforated-wall Caisson with Round Holes[87)]

2.11.2 Performance Verification

(1) **Fig. 2.11.3** shows an example of the sequence of the performance verification of upright wave-absorbing-type quaywalls. The evaluation of the effects of liquefaction, settlement, and other phenomena caused by earthquake ground motions is not shown in **Fig. 2.11.3**. Therefore, for liquefaction, it is necessary to determine whether there is liquefaction and to consider measures against it appropriately by referring to **[Actions and Material Strength Requirements]** in **Chapter 7: Ground Liquefaction**. It is possible to verify the variable situation associated with Level 1 earthquake ground motions by using the seismic coefficient method. However, for high earthquake-resistance facilities, it is desirable to analyze the amount of deformation by using nonlinear seismic response analysis in consideration of the dynamic interaction between the ground and a structure. For upright wave-absorbing-type quaywalls that are not categorized as high earthquake-resistance facilities, the verification of the accidental situation associated with Level 2 earthquake ground motions can be omitted.

(2) The characteristic value of the seismic coefficient for the verification used in the performance verification of upright wave-absorbing-type quaywalls for the variable situations associated with Level 1 earthquake ground motion shall be appropriately calculated by taking the structural characteristics into consideration. For convenience, the characteristic value of the seismic coefficient for the verification of upright wave-absorbing-type quaywalls may be calculated in accordance with that for the gravity-type quaywalls shown in **Chapter 5**: **2.2 Gravity-type Quaywalls**.

(3) It is advisable to examine the wave-absorbing performance by conducting a hydraulic model test. When doing so, it should be noted that the wave-absorbing performance varies depending on the tide level, in addition to the characteristics of incident waves.



*1: The evaluation of the effect of liquefaction is not shown; therefore, this must be separately considered.

*2: The analysis of the amount of deformation due to Level 1 earthquake ground motion may be performed by dynamic analysis when necessary.
For high-earthquake-resistance facilities, the analysis of the amount of deformation due to Level 1 earthquake ground motion should also be performed by dynamic analysis.

*3: For high-earthquake-resistance facilities, verification is performed for Level 2 earthquake ground motion.

Fig.2.11.3 Example of the Sequence of Performance Verification of Upright Wave-absorbing Type Quaywalls

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3 Mooring Buoys

[Ministerial Ordinance] (Performance Requirements for Mooring Buoys)

Article 27

1 The performance requirements for mooring buoys shall be as prescribed respectively in the following items:

(1) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable the safe mooring of ships.

(2) Damage, etc. due to the actions of variable waves, water flows, traction by ships, etc. shall not impair the function of the mooring buoys, and shall not adversely affect the continuous use of the mooring buoys.

2 In addition to the provisions of the preceding paragraph, the performance requirements for mooring buoys in the place where there is a risk of having serious impact on human lives, property, or socioeconomic activity by damage to the mooring buoys shall be such that the structural stability of the mooring buoys is not seriously affected even in cases where the function of the mooring buoys is impaired by design tsunamis, accidental waves, etc.

[Public Notice] (Performance Criteria of Mooring Buoys)

Article 53

1 The performance criteria for mooring buoys shall be as prescribed respectively in the following items:

(1) The buoy shall have the necessary freeboard in consideration of the usage conditions.

(2) The mooring buoy shall have the dimensions necessary for the containment of the swinging area of moored ships within the allowable dimensions.

(3) The following criteria shall be satisfied under the variable situation, in which the dominating actions are variable waves, water flows, and traction by ships.

(a) The risk of impairing the integrity of the anchoring chains of floating bodies, ground chains, and sinker chains shall be equal to or less than the threshold level.

(b) The risk of losing the stability of the buoy due to tractive forces acting on mooring anchors, etc. shall be equal to or less than the threshold level.

2 In addition to the provisions of the preceding paragraph, the performance criteria of the mooring buoys for which there is a risk of serious impact on human lives, property, or socioeconomic activity by damage to the facilities shall be such that the degree of damage under the accidental situation in which the dominating actions are design tsunamis or accidental waves is equal to or less than the threshold level.

[Interpretation]

11. Mooring facilities

(7) Performance criteria of mooring buoys (Article 27 of the Ministerial Ordinance and the interpretation related to Article 53 of the Public Notice)

① The performance requirement for mooring buoys shall be serviceability. The serviceability mentioned here shall mean that the mooring buoy concerned has the necessary freeboard in consideration of the usage conditions as well as the dimensions required for containment of the swinging area of moored ships within the allowable range.

② In setting the freeboard, the expected usage conditions of the facilities concerned shall be appropriately taken into account. Further, in setting the dimensions of mooring buoys, the structure and sectional dimensions of the facilities shall be set so that the containment of the swinging area of floating bodies is appropriately considered in compliance with their expected usage conditions.

③ In addition to the above, the performance requirement for mooring buoys under the variable situation in which the dominating actions are variable waves, water flows and/or tractive forces by ships shall be serviceability. The performance verification items and the standard indexes for the determination of the limit values to such actions shall be as shown in **Attached Table 11-18**.

Attached Table 11-18 Performance Verification Items and Standard Indexes for Determination of Limit Values of Mooring Buoys in Each Design State (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
27	1	2	53	1	3a	Serviceability	Variable	Variable waves [water flow] [traction by ships]	Self-weight, water pressure, water flow	Yield of anchoring chains of floating body, ground chains, or sinker chains	Design yield stress
					3b					Stability of mooring anchors, etc.	Resistance force of mooring anchors, etc. (horizontal, vertical)

* Items in [ ] denote replacing the dominating action according to the design state.
* The stability verification of mooring anchors, etc. refers to verifying that the tensile force acting on such facilities does not exceed their resistance force.

④ The term "mooring anchors, etc." shown in **Attached Table 11-18** is used as a general term for equipment placed on the seabed to retain floating bodies and includes sinkers and the like in addition to mooring anchors.

⑤ The performance requirement for mooring buoys under the accidental situation in which the dominating actions are design tsunamis and/or accidental waves shall be safety. Further, the performance verification item and the standard index for the determination of the limit value to the actions shall be as shown in **Attached Table 11-19**. In addition, to proceed with the performance verification of mooring buoys by referring to **Attached Table 11-19**, the standard indexes for the determination of the limit values shall be appropriately designated based on the structure types.

Attached Table 11-19 Performance Verification Item and Standard Index for Determination of Limit Value of Mooring Buoys in Facilities Prepared for Accidental Incidents under Accidental Situations

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non- dominating action		
27	2	–	53	2	–	Safety	Accidental	Design tsunami [Accidental waves]	Self-weight, water pressure, water flow	Stability of mooring system	–

* Items in [ ] denote replacing the dominating action according to the design state.

3.1 Fundamentals of Performance Verification

(1) The mooring buoy shall secure appropriate stability under the mooring method, the natural conditions at the site, the principal dimensions of the design ships etc.. The mooring buoy here shall be the facilities for mooring ships other than cargo ships and the like that do not involve cargo handling work when moored. In addition, for mooring buoys at which moored ships involve handling work of hazardous cargo such as oil pipelining, **Reference (Part III), Chapter 2, 5.8 Design of Floating Mooring Facilities**, etc. shall be referred to.

(2) Mooring buoys are structurally categorized into three types: sinker type, anchor chain type, and anchored sinker type. The sinker type mooring buoy comprises a floating body, an anchoring chain of a floating body, and a sinker; a mooring anchor is not used, as shown in **Fig. 3.1.1(a)**. The anchor chain type mooring buoy comprises a floating body, an anchor chain, and a mooring anchor; it does not have a sinker, as shown in **Fig. 3.1.1(b)**. Although the construction cost of this type is lower than that of the other types, it is not generally suitable for cases where the area of the mooring basin is limited, because the radius of a moored ship's swinging motion becomes large. The anchored sinker type mooring buoy comprises a floating body, an anchoring chain of a floating body, a ground chain, a sinker chain, a mooring anchor, and a sinker as shown in **Fig. 3.1.1(c)**. Mooring buoys of this type are being used widely in ports and harbors. This type of buoy can be used even when the area of the mooring basin is limited because the radius of a moored ship's swinging motion can be reduced by increasing the weight of the sinker.

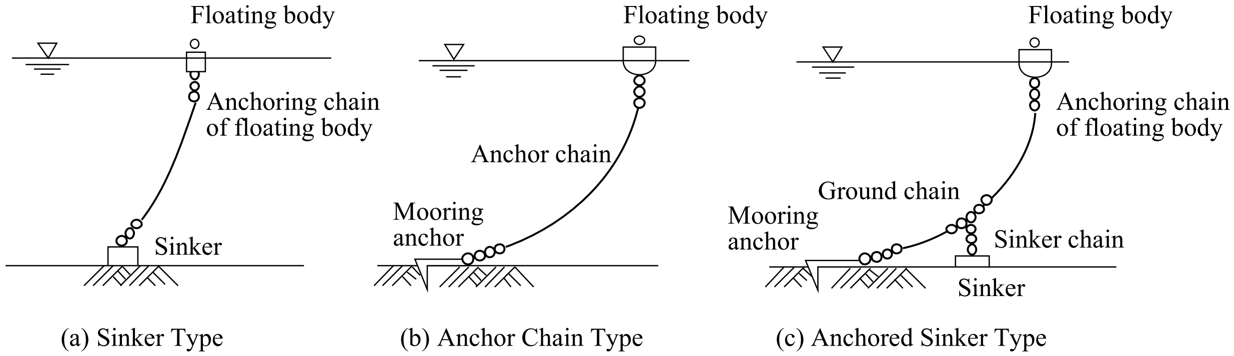


Fig. 3.1.1 Types of Mooring Buoys

(3) The procedure for performance verification of mooring buoys is shown in **Fig. 3.1.2** as an example. Here, the mooring system shall comprise every part of a mooring buoy, and for example, that of the anchored sinker type mooring buoy shall mean each part of a floating body, an anchoring chain of a floating body, a ground chain, a sinker chain, a mooring anchor and a sinker.

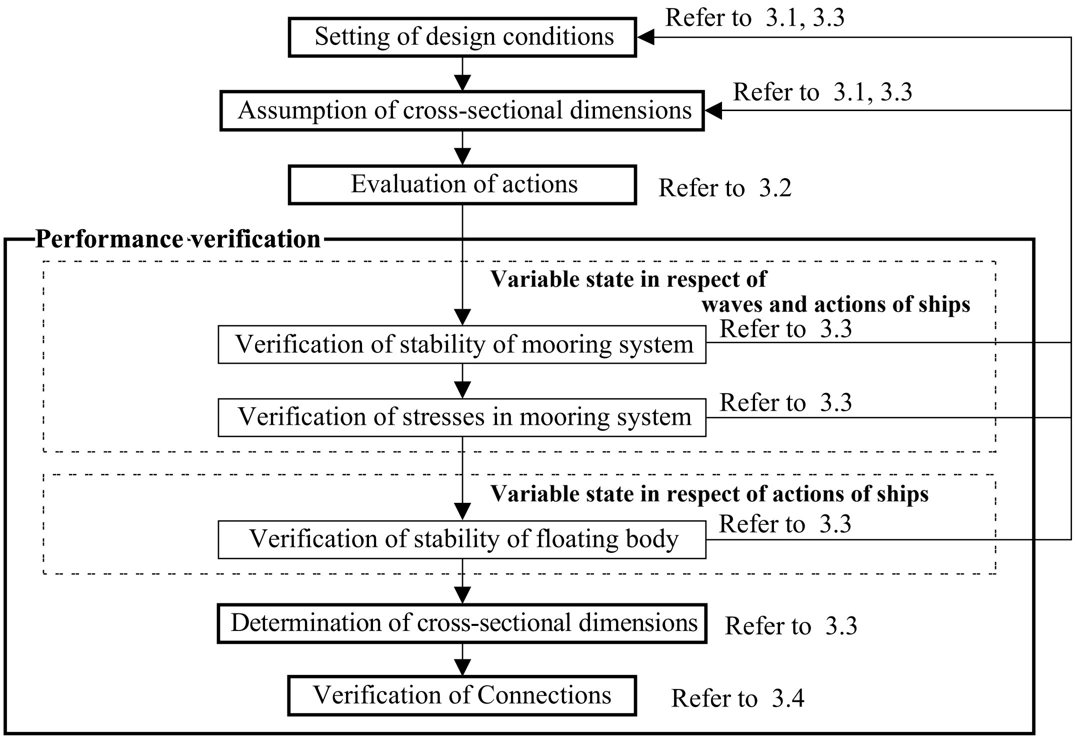
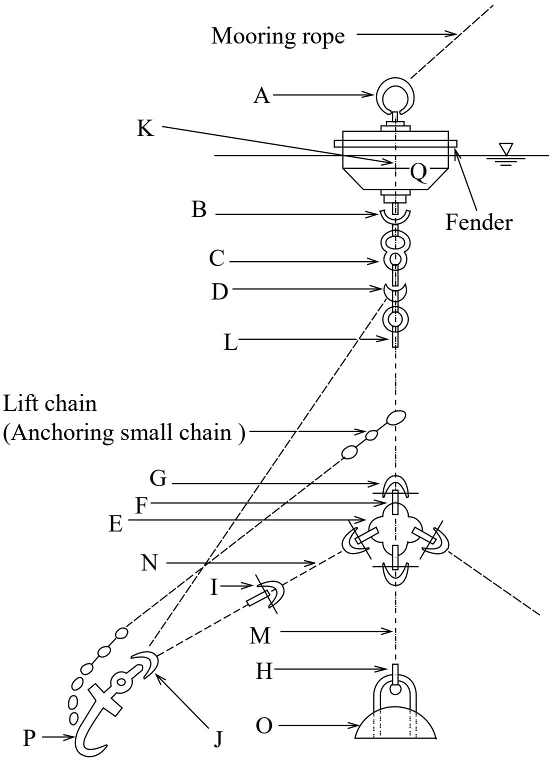


Fig. 3.1.2 Example of Performance Verification Procedure for Mooring Buoys

(4) **Fig. 3.1.3** shows an example of the structure of members of a mooring buoy.



A: Harp shackle (mooring ring) or quick release hook	1
B: Anchor shackle	1
C: Swivel piece	1
D: Joining shackle	1
E: Mooring piece	1
F: Long ring	2
G: Joining shackle	2
H: Anchor shackle	1
I: Joining shackle	2
J: Anchor shackle	2
K: Chain	1
L: Main chain (Anchoring chain of floating body)	1
M: Sinker chain	1
N: Chain or ground chain	4
O: Sinker or pan	1
P: Anchor or screw anchor	1
Q: Buoy	1

Fig. 3.1.3 Example of Structure of Members of Mooring Buoy

(5) The performance verification method for mooring buoys shown in this section shall apply to those categorized as the anchored sinker type from among the types mentioned above. Moreover, since the sinker type and the anchor chain type can be regarded as a simplified anchored sinker type, the same performance verification provisions can be applied to their performance verifications as well.

(6) The tensile force or other force acting on the structural members of a mooring buoy, i.e., an anchoring chain of a floating body and a ground chain, is to be determined in accordance with the shape and/or the weight of each structural member, so that changing even the shape of one structural member can result in a change in all the values. Therefore, to examine the performance verification of a mooring buoy in an economical way, it is necessary to decide the most optimal structure in such a way that the shape of each member is assumed beforehand, the tensile force or other force at this stage is calculated, and then the shape of each member is modified one after another in trial calculations.

(7) The mooring of ships with the mooring buoy can be categorized into single buoy mooring and dual buoy mooring. In the case of the single buoy mooring system, the tractive force acting on the mooring buoy is small, but the area of the mooring basin required is large. In contrast, since the dual buoy mooring system applies two or more mooring buoys to moor a ship, the area of one ship's mooring basin can be made small as it is almost stabilized in the bow to stern direction; however, a large tractive force will act on the buoys. Further, in the case of the dual buoy mooring system, it is desirable that the mooring buoys be arranged so as to be in parallel with the direction of the wind and/or water flow to reduce the tractive force because the ship's swinging motion is small.

(8) The performance verification of the stability of mooring anchors, etc. for the case in which the dominating actions are tsunamis and/or accidental waves must be examined with the drifting of a mooring buoy or a moored ship taken into account, which may be caused by such a tsunami or accidental waves, in order to ensure that it will not make a serious impact on the surroundings.

3.2 Actions

(1) In principle, the tractive force acting on a mooring buoy shall be set considering structural characteristics of the mooring buoy in accordance with the provisions in **Part II, Chapter 8, 2.4 Actions Caused by Traction of Ships**. When setting the tractive force, consideration should be given to the effects of wind, water flows, and waves. However, it should be noted that these are dynamic actions, and thus there are many uncertainties in their relationship with the tractive forces. Therefore, it is preferable that the tractive force acting on a mooring buoy be determined considering the actions that are exerted upon moored ships, such as wind, water flows, and waves, and by referring to the existing tractive force data on buoys of a similar type.

(2) When the motions of a buoy due to wave action are not negligible in terms of the stability of a mooring buoy, it is desirable that the effects of such motions be considered in the calculation of the wave force and/or the resistance force.

(3) When a dynamic analysis of a mooring buoy is performed, the response characteristics vary widely depending on how the waves are applied; in an analysis to which regular waves are applied, the motions of the mooring buoy would be generally either overestimated or underestimated. Therefore, random waves with spectral characteristics shall be employed in the analysis.

(4) In the case of single buoy mooring, the moored ship will enter into a swinging motion. The results of past hydraulic model tests show that the swing angle, the intersection angle between the wind direction and the ship's longitudinal axis, is about 30° at maximum, varying widely with the anchor chain length, the wind velocity, etc.[1] **Reference 2)** can be used as a reference for single buoy mooring.

(5) When the wave height is large under the variable situation related to the waves, a tensile force would act, making an impact upon a mooring buoy. To reduce the impact of the tensile force, it is desirable that an elastic chain be used for part of the mooring system.[3]

(6) **Table 3.2.1** shows examples of design conditions and corresponding tractive forces on mooring buoys.

Table 3.2.1 Examples of Design Conditions for Mooring Buoys

Design ship DWT (ton)	Mooring method	Wind velocity (m/s)	Tidal current (m/s)	Wave height (m)	Tractive force (kN)
1,000	Single buoy	50	0.5	2.0	185
3,000	Ditto	50	0.5	4.0	409
15,000	Ditto	15	0.51	0.7	245
20,000	Ditto	20	1.0	–	589
130,000	Ditto	60	0.67	10.0	1,370
260,000	Ditto	25	0.51	3.0	1,840
30,000	Dual buoy	15	–	–	1,490
100,000	6-points	20	–	1.5	1,470

3.3 Performance Verification of Each Part of a Mooring Buoy

(1) General

For the sizes and materials strength, etc. of each part of a mooring buoy, including the mooring anchor, sinker, sinker chain, ground chain, anchoring chain of the floating body, and the floating body itself, **Part III, Chapter 5, 6 Floating Piers** can be referred to; further, these factors shall be set appropriately in accordance with the tractive force of ships, the structure of the mooring buoy, the mooring method, and the like.

(2) Mooring Anchor

① Normally, three mooring anchors are attached to a mooring buoy. In the performance verification of a mooring buoy, however, it shall be assumed that only one of the three anchors is set to resist the horizontal force. The arrangement of the mooring anchors must be designed in such a way that the buoy would not capsize even if one of the anchor chains were to be broken.

② The horizontal force acting on a mooring buoy shall be resisted only by the mooring anchors. For the anchor holding power of mooring anchors, **Part III, Chapter 5, 6 Floating Piers** shall be referred to. There are cases in which the anchor holding power of a portion of the ground chains that is in contact with the seabed is taken into account; it is however desirable to consider in the performance verification that only the mooring anchors will resist the horizontal force because the interaction mechanism of the mooring anchor with the ground chain has some unclear points and the resistance force of such chains is quite small compared to that of the mooring anchor. Single fluke stock anchors are often used for mooring anchors in port facilities. Further, buried anchors[4)] are also used in place of single fluke stock anchors.

③ The anchor holding power of mooring anchors varies widely depending on the ground conditions of the seabed, the topography, the shape of the mooring anchors, etc., so that these conditions should be properly taken into consideration.

(3) Sinker and Sinker Chain

① Normally the length of a sinker chain ranges from 3 to 4 m. It is preferable not to use an excessively long sinker chain because it allows a large range for the upward movement of the sinker and increases the risk of tangling of the chain and thus the risk of abrasion and accidental breaking of the chain. Generally, the sinker chain should be of the same diameter as that of the anchoring chain of the floating body.

② The vertical and horizontal forces acting on the sinker can be calculated generally based on the tension of the anchoring chain of the floating body and the distance of the horizontal movement of the floating body, using the following **equation (3.3.1)**. [5)]

$$\left.\begin{aligned} P_V &= T_A \sin\theta_1 = (T_C - wl)\sin\theta_1 \\ P_H &= T_A \cos\theta_1 = (T_C - wl)\cos\theta_1 \end{aligned}\right\} \qquad \textbf{(3.3.1)}$$

where

P_V, P_H : vertical and horizontal forces acting on the sinker, etc. respectively (kN)

θ_1 : angle that the anchoring chain of the floating body makes with the horizontal plane at the sinker attachment point (º)

T_A : tension of the anchoring chain of the floating body at the sinker attachment point (kN)

T_C : tension of the anchoring chain of the floating body at the floating body attachment point (kN)

w : weight of the anchoring chain of the floating body per unit length in water (kN/m)

l : length of the anchoring chain of the floating body (m)

Further, θ_1 can be obtained by solving the following equations.

$$\left.\begin{aligned} l &= \frac{T_A \cos\theta_1}{w}(\tan\theta_2 - \tan\theta_1) \\ \Delta K &= \frac{T_A \cos\theta_1}{w}\left\{\sinh^{-1}(\tan\theta_2) - \sinh^{-1}(\tan\theta_1)\right\} \end{aligned}\right\} \qquad \textbf{(3.3.2)}$$

where

ΔK : distance of horizontal movement of the floating body (m)

θ_2 : angle that the anchoring chain of the floating body makes with the horizontal plane at the floating body attachment point (º)

In variable situations in respect of action by ships, the anchoring chain of the floating body usually becomes approximately straight when the tractive force acts on it, so that the following approximation can be used:

$$\theta_2 \approx \theta_1 = \cos^{-1}\frac{\Delta K}{l} \qquad \textbf{(3.3.3)}$$

③ The weight of the sinker most commonly used for 5,000 GT ships and 10,000 GT ships is about 50 kN and 80 kN respectively, so that it can be determined using these values as references. The values mentioned above indicate the weight in water. Sinkers may be of any shape and material as long as they satisfy the weight requirement, but in Japan disk-shaped cast iron sinkers are used commonly while concrete is seldom used.

When the bottom surface of the sinker is made slightly concave, if the seabed is soft, a considerable adhesion effect with the ground is expected.

④ The role of the sinker is to absorb the impact force acting on the chain and to make the anchoring chain of the floating body shorter. When the anchoring chain of the floating body is to be shortened to reduce the distance of movement of the ship, the weight of the sinker must be increased accordingly.

⑤ In certain cases, buried anchors may be used instead of sinkers.

(4) Ground Chain

① The angle that the ground chain makes with the seabed at the mooring anchor attachment point is desirably less than 3º because the holding power of the mooring anchor decreases sharply as the angle increases to 3º or more.[5)] In many cases, the weight of the ground chain is determined in such a way that the ground chain satisfies the above mentioned condition when the tractive force acts on the mooring buoy. When the tractive force is large, the attachment angle between the mooring anchor and the ground chain may be made smaller by lengthening the ground chain. The inclination angle θ_1 of the ground chain at the mooring anchor attachment point can be generally calculated by **equation (6.4.8)** described in **Part III, Chapter 5, 6.4. Performance Verification.** The equation is redefined and expressed as **equation (3.3.4)**. In addition, **Fig.3.3.1** shows a situation in which a ship is moored at a mooring buoy of the anchored sinker type, and indicates the notation of the lengths, angles, and so forth to be used in equations for the calculation of tensions on anchor chains and the like.

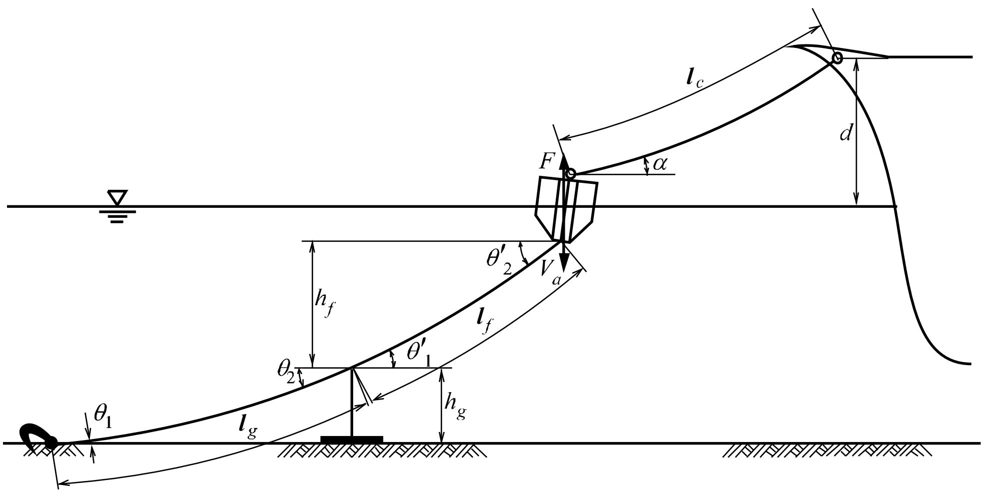


Fig. 3.3.1 Performance Verification of Anchored Sinker Type Mooring Buoy

$$\left.\begin{aligned} l_g &= \frac{P_H}{w}(\tan\theta_2 - \tan\theta_1) \\ h_g &= \frac{P_H}{w}(\sec\theta_2 - \sec\theta_1) \end{aligned}\right\} \qquad \textbf{(3.3.4)}$$

where

l_g : length of the ground chain (m)

h_g : vertical distance between the upper end of the ground chain and the seabed (i.e. the sum of the length of the sinker chain, the height of the sinker, and the allowance) (m)

P_H : horizontal component of the tractive force acting on the floating body (kN)

w : weight of the ground chain per unit length in water (kN/m)

θ_1 : inclination angle of the ground chain at the attachment point to the mooring anchor (º)

θ_2 : inclination angle of the ground chain at the upper end of the chain (º)

It should be noted that the calculation should be made by assuming l_g, w, and h_g such as to obtain θ_1 that becomes less than 3°.

② The maximum tension T_g of the ground chain can be calculated using **equation (6.4.5)** described in **Part III, Chapter 5, 6.4 Performance Verification**. The equation is redefined and expressed as **equation (3.3.5)**.

$$T_g = P_H \sec\theta_2 \qquad \textbf{(3.3.5)}$$

where

P_H : horizontal component of the tractive force acting on the floating body (kN)

θ_2 : inclination angle of the ground chain at the upper end of the chain (º)

③ The tensile yield strength of the chain shall be set based on **Part III, Chapter 5, 6 Floating Piers**. In the case of mooring buoys, however, the diameter of the chain is usually determined not only on the basis of strength, but on the basis of the theory that the use of a heavier chain helps absorb the energy of impact forces, or, as is known from **equation (3.3.4)**, the use of a shorter chain reduces the radius of the ship's swinging motion; in general, the chain diameter is designed so that it is equal to that of a chain on which a maximum tension equivalent to around 1/5 to 1/8 of the breaking test load can act.

(5) Anchoring Chain of Floating Body

① The length l_f of the anchoring chain of the floating body shall be determined in such a way as to reduce the tension acting on both the anchoring chain of the floating body and the mooring rope as well as to lessen the radius of the ship's swinging motion. It should be noted that the relation between the ratio of the anchoring chain length to the water depth and the degree of abrasion of the anchoring chain of the floating body has not been clarified yet.

② It is desirable that the tension acting on the anchoring chain of a floating body and the displacement of the floating body be obtained by means of a numerical simulation of motions, but the results obtained under similar conditions in the past as well as the method shown in the following ③ to ⑤ can be applied.

③ The weight of the anchoring chain of a floating body per unit length in water w_f (kN/m) can be calculated generally by replacing w with w_f in **equation (3.3.4)**. Here, l_g and h_g in the equation should be replaced by the length of the anchoring chain of floating body l_f (m) and the vertical distance between the upper and the lower ends of the anchoring chain of floating body h_f (m), respectively. The vertical distance between the upper and the lower ends of the anchoring chain of floating body h_f denotes the vertical distance between the attachment point to the floating body and the upper end of the sinker chain when the sinker is lifted up to the point where its bottom is completely separated from the seabed surface. The force P_H (kN) represents the horizontal component of the tractive force acting on the mooring buoy, and θ_2 and θ_1 should be replaced by the inclination angles of the anchoring chain of the floating body at the upper and lower ends, θ_2' (º) and θ_1' (º), respectively. Further, the inclination angle θ_1' can be calculated as shown in **Fig. 3.3.2** from the conditions of balance among the lower end tension of the anchoring chain of floating body T_{fv}, the upper end tension of the ground chain T_g, and the upper end tension of the sinker chain T_{sv}, where T_{sv} is exactly the summation of the weights of the sinker and sinker chain in water, and T_g and its direction can be calculated using **equation (3.3.5)**.

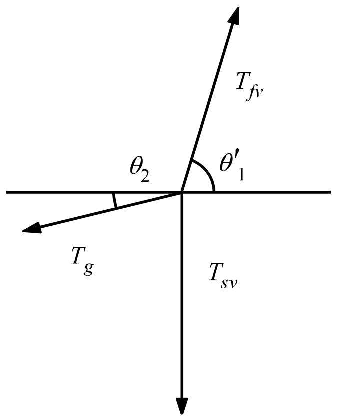


Fig. 3.3.2 Schematic Drawing for Tension of Ground Chain

④ The tension of the anchoring chain of a floating body at the upper end can be calculated using **equation (3.3.5)**. Here, the horizontal component of the tractive force can be adopted as the horizontal force. The angle θ_2' that the anchoring chain of the floating body makes with the horizontal plane at the floating body attachment point can be calculated by **equation (3.3.4)** using the previously obtained weight of the anchoring chain of the floating body per unit length in water. In general, this tension is used for the performance verification of the stress on the anchoring chain of the floating body.

⑤ The horizontal displacement ΔK of the floating body can be generally calculated by means of **equation (6.4.9)** described in **Part III, Chapter 5, 6.4 Performance Verification.** The equation is redefined and expressed as **equation (3.3.6)**.

$$\Delta K = \frac{P_H}{w}\left\{\sinh^{-1}(\tan\theta_2') - \sinh^{-1}(\tan\theta_1')\right\} \tag{3.3.6}$$

The appropriateness of the resultant value of the displacement of the floating body obtained from this equation should be examined in comparison with the area of the mooring basin; if the value is found to be excessively large, it is necessary to either shorten the anchoring chain of the floating body and increase the weight of the sinker or increase the unit length weight of the anchoring chain of the floating body.

(6) Floating Body

In variable situations in respect to the action of moored ships, the floating body of a mooring buoy shall be designed in such a way that it does not become submerged. Even when no ship is moored, the floating body must be kept afloat with a freeboard equal to around 1/2 to 1/3 of its height maintained with the anchoring chain of the floating body, and also, if applicable, part of the sinker chain and the ground chain suspended from it. The buoyancy necessary for the floating body shall be determined to meet these two requirements. A floating body buoyancy satisfying the former requirement can be generally calculated by **equation (3.3.7)**

$$F = V_a - \frac{P}{\sqrt{\left(\frac{l_c}{d}\right)^2 - 1}} \tag{3.3.7}$$

where

F : required buoyancy of the floating body (kN)

V_a : vertical force acting on the floating body (kN)

P : tractive force (kN)

l_c : length of the mooring rope (m)

d : vertical distance between the ship's hose pipe and the water surface (m)

Here, the vertical force V_a acting on the floating body can be obtained by **equation (6.4.6)** shown in **Part III, Chapter 5, 6.4 Performance Verification**. However, it should be noted that the total buoyancy actually required is the sum of the buoyancy needed to resist the tractive force and the self-weight of the floating body.

3.4 Performance Verification of Structural Members

(1) For floating bodies, there is a spinning top type, discus type, pear type, barrel type, sphere type, cone type, and so forth. The most commonly used types are the spinning top type and the discus type. Each part of a floating body shall have the strength to resist design water pressures under conditions in which the floating body is totally submerged and capsized in any direction. Partition walls may be built inside the floating body. There are also cases in which a movable lever is installed inside the floating body to keep its top surface constantly horizontal, as shown in **Fig. 3.4.1**. Wooden or rubber fenders need to be attached to the floating body to protect it against damage due to ship impact.

(2) The metal fittings to be used, including shackles, swivels, links, and mooring pieces, must have a strength corresponding to that of the chain.

(3) Various types of shackles, swivels, etc. used at the connection points of the floating body to the anchoring chain of a floating body and the sinker chain to the anchoring chain of the floating body are subject to extensive abrasion due to the swinging motion of the floating body, and this therefore should be noted for the implementation of performance verification and maintenance.

(4) It is desirable that anchoring chains of a floating body, ground chains, and sinker chains be inspected once or twice every year, and be renewed if abrasion and/or corrosion corresponding to 10% or more of the original diameter is found. Under average usage, they are generally renewed every 4 years.

(5) The standard sizes of harp shackles are as shown in **Table 3.4.1**. Simple metal fittings for line handling may sometimes be used with harp shackles.

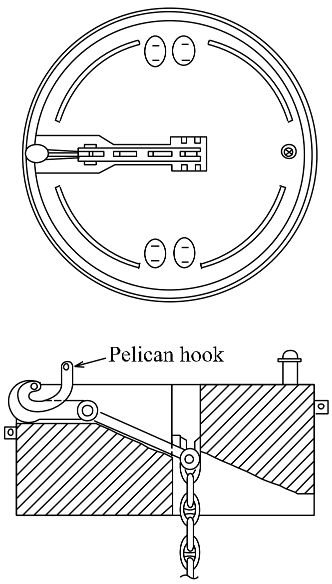


Fig. 3.4.1 Floating Body for Keeping Top Surface Horizontal

Table 3.4.1 Standard Sizes of Harp Shackles

Moored Ship GT (ton)	Inside Diameter (mm)	Thickness of Ring
500	200	80
1,000	240	80
2,000	280	100
3,000	320	100
4,000	360	110
5,000	400	110
6,000	440	110
8,000	480	120
10,000	520	130
15,000	520	130
20,000	520	130
25,000	560	140
30,000	600	150

[References]

1) Yoneda, K.: Wind tunnel experiment on drifting motion of buoy moored ship, Proceedings of 28th Conference of Japan Institute of Navigation, (Mooring buoy -process for standardization- reference), 1962. (in Japanese)

2) Suzuki, Y.: Study on the Design of Single Point Buoy Mooring, Technical Note of PHRI, No.829, 1996. (in Japanese)

3) Hiraishi, T. and Y. Tomita: Model Test on Countermeasure to Impulsive Tension of Mooring Buoy, Technical Note of PHRI, No.816, p.18, 1995. (in Japanese)

4) JSCE: Guideline and Commentary for Design of Offshore Structures (Draft), 1973. (in Japanese)

5) U.S. Navy Bureau of Yards and Docks: Mooring Guide, Vol.1, p.61, 1954.

4 Mooring Piles

[Ministerial Ordinance] (Performance Requirements for Mooring Piles)

Article 28

The performance requirements for mooring piles shall be as prescribed respectively in the following items:

(1) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable the safe mooring of ships.

(2) Damage, etc. due to the actions of berthing, traction by ships, etc. shall not impair the function of the mooring piles, and shall not adversely affect the continuous use of the mooring piles

[Public Notice] (Performance criteria of Mooring Piles)

Article 54

The performance criteria of mooring piles shall be as prescribed respectively in the following items:

(1) The mooring piles shall have the dimensions required for the usage conditions.

(2) The following criteria shall be satisfied under the variable situation, in which the dominating actions are ship berthing and traction by ships:

(a) For mooring piles with superstructures, the risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(b) The risk that the axial forces acting on the piles may exceed the resistance capacity due to failure of the ground shall be equal to or less than the threshold level.

(c) The risk that the stress on the piles may exceed the yield stress shall be equal to or less than the threshold level.

[Interpretation]

11. Mooring Facilities

(8) Performance criteria of mooring piles (Article 28 of the Ministerial Ordinance and the interpretation related to Article 54 of the Public Notice)

The performance requirement for mooring piles under the variable situation in which the dominating actions are ship berthing and/or traction by ships shall be serviceability. In addition, the performance verification items and the standard indexes for the determination of the limit values to such actions shall be as shown in **Attached Table 11-20**.

Attached Table 11-20 Performance Verification Items and Standard Indexes for Determination of Limit Values for Mooring Piles in Each Design State (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
28	–	2	54	–	2a	Serviceability	Variable	Ship berthing and/or traction by ships	Self-weight	Sectional failure of superstructure *1)	Design resistance force of section
					2b					Axial force on pile	Ratio of bearing-capacity-related action on pile to resistance force (pushing, pulling)
					2c					Yielding of pile	Design yield stress

*1): Only for structures with a superstructure.

(1) In setting the sectional dimensions and ancillary facilities in regard to the performance verification of mooring piles, consideration shall be properly given to their expected usage conditions.

(2) In the implementation of the performance verification of mooring piles, **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles** and **5.3 Open-type Wharves on Coupled Raking Piles of this Chapter** shall be referred to in accordance with the characteristics of the facilities.

5 Piled Piers

[Ministerial Ordinance] (Performance Requirements for Piled Piers)

Article 29

1 The performance requirements for piled piers shall be as prescribed respectively in the following items in consideration of the structural type:

(1) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable the safe and smooth berthing of ships, embarkation and disembarkation of people, and handling of cargo.

(2) Damage to the piled pier due to self-weight, earth pressure, Level 1 earthquake ground motions, berthing and traction by ships, surcharge load, etc. shall not impair the functions of the piers and shall not adversely affect its continuous use.

2 In addition to the provisions of the previous paragraph, the performance requirements for piled piers listed in the following items shall be as prescribed respectively in those items:

(1) "Performance requirements for piled piers for the purpose of environmental conservation" means that piled piers shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to contribute to conservation of the environment of ports and harbors without impairing the original functions of the piled piers.

(2) "Performance requirements for piled piers classified as high earthquake-resistance facilities" means that damage to piled piers, etc. due to Level 2 earthquake ground motions, etc. shall not affect the restoration through minor repair works of functions required for the quatwalls in the aftermath of the occurrence of Level 2 earthquake ground motion. Provided, however, that for the performance requirements for the piled piers which requires further improvements in earthquake-resistant performance due to environmental conditions, social conditions, etc. to which the piled piers are subjected, damage due to Level 2 earthquake ground motions, etc. shall not impair the functions necessary for the quatwalls in the aftermath of the occurrence of Level 2 earthquake ground motion, and shall not adversely affect the continuous uses of the piled piers.

[Public Notice] (Performance Criteria of Piled Piers)

Article 55

1 The provisions of Article 48 apply mutatis mutandis to the performance criteria of piled piers.

2 In addition to the provisions of the preceding paragraph, the performance criteria of the access bridge of piled piers shall be as prescribed respectively in the following items:

(1) The access bridge of piled piers shall satisfy the following criteria:

(a) The access bridge of piled piers shall have the dimensions necessary for enabling the safe and smooth loading, unloading, embarkation and disembarkation, etc. in consideration of the usage conditions.

(b) The access bridge of piled piers shall not transmit horizontal loads to the superstructure of the piled pier, and shall not fall down even when the piled pier and the earth-retaining part are displaced owing to the actions of earthquakes, etc.

(2) The following criteria shall be satisfied in variable situations in which the dominant actions are Level 1 earthquake ground motions, ship berthing and traction by ships, and imposed load:

(a) The risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(b) The risk that the axial force acting on the piles may exceed the resistance capacity owing to failure of the ground shall be equal to or less than the threshold level.

(c) The risk that the stress in the piles may exceed the yield stress shall be equal to or less than the threshold level.

(3) The following criteria shall be satisfied under the variable situation in which the dominating action is variable waves:

(a) The risk of impairing the stability of the access bridge due to uplift acting on the access bridge shall be equal to or less than the threshold level.

(b) The risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(c) The risk that the axial force acting on piles may exceed the resistance capacity owing to failure of the ground shall be equal to or less than the threshold level.

(4) For the structures with stiffening members, the risk of impairing the integrity of the stiffening members and connection points of the structures under the variable situation, in which the dominating actions are variable waves, Level 1 earthquake ground motions, ship berthing and traction by ships, and surcharge load, shall be equal to or less than the threshold level.

3) The provisions of Article 49 through Article 52 apply mutatis mutandis to the performance criteria of the earth-retaining parts of piled piers in consideration of the structural type.

[Interpretation]

11. Mooring Facility

(9) Performance Criteria of Piled Piers

① **Piled piers which are classified as high earthquake-resistance facilities** (Article 29 paragraph 2 item 2 of the Ministerial Ordinance and the interpretation related to Article 55 paragraph 1 of the Public Notice)

a) In regard to the interpretation concerning performance requirements and performance criteria of piled piers that are high earthquake-resistance facilities, the interpretation concerning performance requirements and performance criteria of quay walls that are high earthquake-resistance facilities is applied, excluding performance verification items and standard indexes to provide limit values.

b) Verification items and standard indexes to provide limit values of piled piers that are high earthquake-resistance facilities to accidental situations with a dominating action of Level 2 earthquake ground motion shall be in accordance with **Attached Table 11-21**.

Attached Table 11-21 Performance Verification Items and Standard Indexes to Provide Limit Values of Piled Piers That Are High Earthquake-resistance Facilities

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to provide limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non- dominating action		
29	2	2	55	1	–	Restorability and Serviceability	Accidental	Level 2 earthquake ground motion	Self weight, surcharges	Deformation of face line	Residual deformation
										Cross-sectional failure of the superstructure	Design cross-sectional resistance
										Damage to piles	Limit curvature
										Axial forces in the piles	Bearing power of piles

c) In **Attached Table 11-21**, the standard index to provide the limit value for the deformation of the face line shall apply to gravity-type mooring quay walls that are high earthquake-resistance facilities.

d) In **Attached Table11-21**, the following performance verification shall be carried out concerning damage to piles of piled piers that are high earthquake-resistance facilities in consideration of the types of high earthquake-resistance facilities.

i) Specifically designated (emergency supply transport) and specifically designated (trunk line cargo transport)

It shall be verified that no pile which reaches the limit curvature at two locations exists in the cross section of the piled pier concerned.

ii) Standard (emergency supply transport)

It shall be verified that at least one pile which reaches the limit curvature at less than two locations on a pile exists among the piles comprising the piled pier concerned. (It shall be verified that all the piles existing in the cross section of the piled pier concerned are not in a state such that the limit curvature at two or more locations is reached on a pile.)

e) The verification items and standard indexes to provide limit values of the high earthquake-resistance facilities of open-type wharves on vertical piles shall be applied for piled piers that are high earthquake-resistance facilities of structures with stiffened members.

② **Main structure of piled piers** (Interpretation related to Paragraph 1, Article 29 of the Ministerial Ordinance and Paragraph 2, Article 55 of the Public Notice)

a) The performance requirement for piled piers under a variable situation where the dominating actions are Level 1 earthquake ground motions, berthing and traction by ships, surcharges, and variable waves shall be serviceability. Performance verification items and the standard indexes to provide limit values to these actions concerning the superstructure and the piles of piled piers are shown in **Attached Tables 11-22** and **11-23**.

Attached Table 11-22 Performance Verification Items and Standard Indexes to Provide Limit Values in Each Design Situation (Excluding Accidental Situations) Concerning Superstructure of Piled Piers

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design situation</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index to provide limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>Situation</th><th>Dominating action</th><th>Non- dominating action</th></tr>
<tr><td rowspan="6">29</td><td rowspan="6">1</td><td rowspan="6">2</td><td rowspan="6">55</td><td rowspan="6">2</td><td rowspan="5">2a</td><td rowspan="6">Serviceability</td><td rowspan="6">Variable</td><td>Berthing and traction by ships</td><td>Self weight, surcharges</td><td rowspan="3">Cross-sectional failure of superstructure</td><td rowspan="3">Design cross-sectional resistance</td></tr>
<tr><td>Level 1 earthquake ground motion</td><td>Self weight, surcharges</td></tr>
<tr><td>Surcharges (including surcharges during cargo handling)</td><td>Self weight, wind acting on cargo handling equipment and ships</td></tr>
<tr><td>Surcharges (including surcharges during cargo handling)</td><td>Self weight, wind acting on cargo handling equipment and ships</td><td>Crack width of superstructure cross-section</td><td>Limit value of bending crack width</td></tr>
<tr><td>Repeatedly applied surcharges</td><td>Self weight</td><td>Fatigue failure of superstructure</td><td>Design fatigue strength</td></tr>
<tr><td>3b</td><td>Variable waves</td><td>Self weight</td><td>Cross-sectional failure of superstructure</td><td>Design cross-sectional resistance</td></tr>
</table>

Attached Table 11-23 Performance Verification Items and Standard Indexes to Provide Limit Values in Each Design Situation (Excluding Accidental Situations) Concerning Piles of Piled Piers

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to provide limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non- dominating action		
29	1	2	55	2	2b	Serviceability	Variable	Berthing, traction by ships	Self weight, surcharges	Axial forces in piles	Action-resistance ratio concerning bearing capacity of piles
								Level 1 earthquake ground motion	Self weight, surcharges		
								Surcharges (including surcharges during cargo handling)	Self weight, wind acting on cargo handling equipment and ships		
					2c			Berthing and traction by ships	Self weight, surcharges	Yielding of piles	Design yield stress of piles
								Level 1 earthquake ground motion	Self weight, surcharges		
								Surcharges (including surcharges during cargo handling)	Self weight, wind acting on cargo handling equipment and ships		
					3c			Variable waves	Self weight	Axial forces acting in piles	Action-resistance ratio concerning bearing capacity of piles

b) The performance verification item and standard index to provide the limit value concerning access bridges of piled piers under the variable situation in which the dominant action is variable waves is shown in **Attached Table 11-24**. In addition to that shown in **Attached Table 11-24**, performance verification items and standard indexes to provide limit values concerning access bridges of piled piers shall be adequately established as necessary under the variable situation in which the dominant action is surcharges.

Attached Table 11-24 Performance Verification Item and Standard Index to Provide Limit Value in Each Design Situation (Excluding Accidental Situations) Concerning Access Bridges of Piled Piers

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to provide limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non- dominating action		
29	1	2	55	2	3a	Serviceability	Variable	Variable waves	Self weight	Uplift force on access bridge	Design cross-sectional resistance

c) Performance verification items and the standard index to provide a limit value concerning piled piers of structures with stiffening members under the variable situation in which the dominating actions are Level 1 earthquake ground motion, berthing and traction by ships, surcharges, and variable waves shall comply with those of piled piers, and are shown in **Attached Table 11-25**.

Attached Table 11-25 Performance Verification Items and Standard Indexes to Provide Limit Values in Each Design Situation (Excluding Accidental Situations) Concerning Piled Piers of Structures with Stiffening Members

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to provide limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
29	1	2	55	2	4	Serviceability	Variable	Berthing and traction by ships	Self weight, surcharges	Yielding of stiffening members	Design yield stress Design shear force resistance
								[Level 1 earthquake ground motion]	(Self weight, surcharges)	Failure of connections at joints	Design shear force resistance
								[Surcharges (including surcharges during cargo handling)]	(Self weight, surcharges, and wind acting on ships)	Punching shear failure at joints	Design shear force resistance
								Repeatedly acting surcharges	Self weight	Fatigue failure of joints	Design fatigue strength
								Variable waves	Self weight	Failure of connections at joints	Design shear force resistance

* Items within square brackets [ ] in the column "Dominating action" indicate that the design situation replaces the dominating actions.
* Items within parentheses ( ) in the column "Non-dominating action" indicate that this shall be read according to dominating actions.

③ **Earth-retaining sections of piled piers** (Interpretation related to Paragraph 1, Article 29 of the Ministerial Ordinance and Paragraph 3, Article 55 of the Public Notice)

The performance criteria and interpretation concerning earth-retaining sections of piled piers shall, in consideration of the structural types, comply with the criteria and their interpretation in Article 49 "Performance Criteria of Gravity-type Quay Walls" through Article 52 "Performance Criteria of Cell Type Quay Walls" of the Public Notice.

④ **Symbiosis piled pier** (Interpretation related to Item 1, Paragraph 2, Article 29 of the Ministerial Ordinance and Paragraph 1, Article 55 of the Public Notice)

a) A piled pier for environmental conservation is called a "symbiosis piled pier". The following are applied together with the criteria for piled piers:

b) The performance requirement for symbiosis piled piers shall be serviceability. Here, serviceability indicates the performance required to contribute to the preservation of the port environment, such as wildlife and the ecosystem, without impairing the original functions of the piled pier concerned.

c) The dimensions of piled piers for environmental conservation include the structure, cross-sectional dimensions, and ancillary facilities. In establishing the structure and cross-sectional dimensions and installing ancillary facilities in the performance verification of piled piers for environmental conservation, contributing to the preservation of the port environment, including wildlife and the ecosystem, without impairing the original functions of the piled piers concerned shall be considered adequately.

5.1.1 Dimensions of piled piers

(1) Performance verification items held in common to multiple piled piers may be in accordance with **[Facilities] 2.1 Common Items for Quay walls** in this Chapter.

(2) The structural types of piled piers include open-type wharves on vertical piles, open-type wharves on coupled raking piles, jacket type piers, and strutted frame type piers.

(3) Ancillary facilities

In the performance verification of piled piers, it is necessary to appropriately consider ancillary facilities for the piled pier to be safely and efficiently used.

(4) Access bridges

In setting the structure and cross-sectional dimensions of access bridges in the performance verification of piled piers, it is necessary to appropriately consider the conditions of use of the concerned piers in order for the piled pier to be safely and efficiently used.

Also, in setting the structure and cross-sectional dimensions of access bridges in the performance verification of piled piers, it is necessary to appropriately consider the amount of relative deformation between the main structures of the piled pier.

5.1.2 Symbiosis piled piers

(1) Symbiosis piled piers[1)] are piled piers which contribute to creating a hospitable environment in ports and which aim for wildlife inhabitation at beaches, etc. according to the natural circumstances where the facilities concerned are located (**[Reference (Common)] Chapter 3. 2. Symbiosis Port Facilitie**s). It is also possible to add a wildlife inhabitation function when an existing piled pier is improved in order to convert it to a symbiosis piled pier.

(2) The effect on the target of wildlife inhabitation (**[Reference (Common)] Chapter 3. 2. Symbiosis Port Facilities**) shall be grasped with environmental surveys and numerical models, etc. In performance verification, it shall be confirmed that the structure, the cross-section, and the ancillary facilities are suitable to achieve the goal.

(3) The performance requirement for symbiosis piled piers shall be possessing inhabitation functions for wildlife. The impact (dominating actions) shall be ensuring the environment necessary for the inhabitation of wildlife, the existence of the foundation for wildlife, and the external forces such as waves and flows. The environment necessary for wildlife inhabitation includes, for example, water depth and clarity, which affect the amount of light necessary for photosynthesis, and water temperature, which affects the activities of life. More precisely, if the goal is the inhabitation of sessile organisms, it is necessary that the structure and the cross-section of the piled pier and the foundation and the slope of the ancillary facilities be suitable for the target sessile organisms to attach to.

(4) The performance verification of symbiosis piled piers shall be carried out by confirming on the basis of existing knowledge that the environment of the place where wildlife shall live symbiotically is within the range in which the target wildlife can live. For example, in the performance verification for a piled pier which shall serve for the inhabitation of seaweed beds, the light amount, which affects photosynthesis and respiration, and water temperature shall be considered and the performance verification shall confirm that the environment is within the range in which the target seaweed beds can thrive. In cases in which it is possible to estimate the change in the environmental conditions after the symbiosis piled pier is installed or to estimate future environmental changes, etc., verification to confirm that the environment is suitable for wildlife using numerical models of growth is also possible.

(5) In the performance verification of symbiosis piled piers, **[Facilities] Chapter 4. 4. Symbiosis breakwater, [Reference (Common)] Chapter 3. 2. Symbiosis port facilities and Guidelines[1)] for maintenance of symbiosis port facilities** can be referred to.

5.2 Open-type Wharves on Vertical Piles

5.2.1 General

(1) An example of a cross-section of an open-type wharf on vertical piles is shown in **Fig. 5.2.1.**

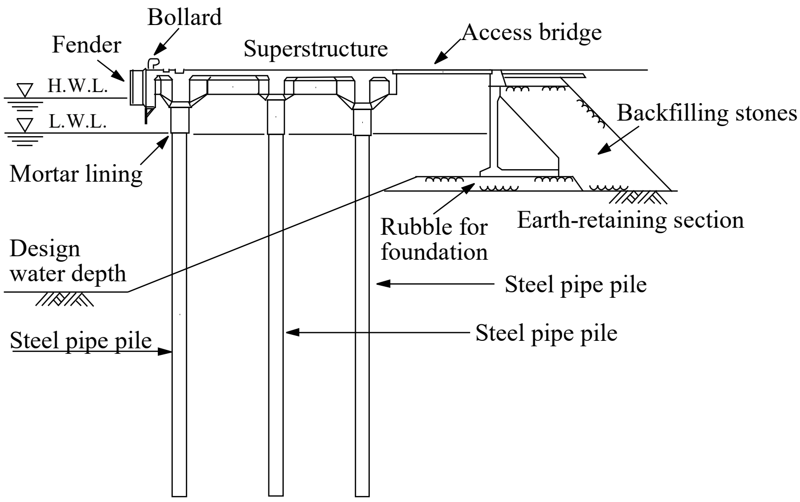
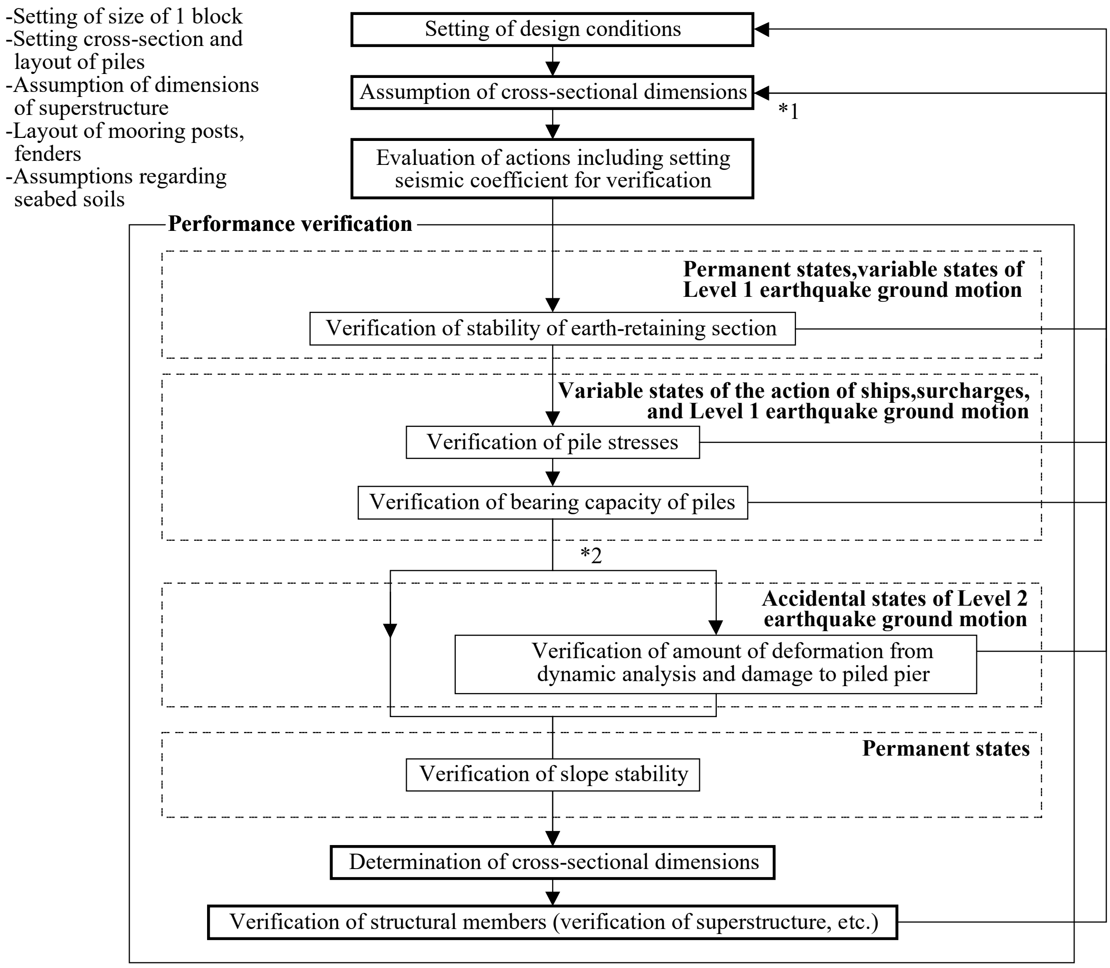


Fig. 5.2.1 Example of a Cross-section of an Open-Type Wharf on Vertical Piles

(2) The following refers to open-type wharves on vertical piles using either steel pipe piles or steel sections; however, it may also be applied to similar facilities when their dynamic characteristics are taken into account.

(3) For the performance verification procedure of open-type wharves on vertical piles, it is possible to refer to **Fig. 5.2.2**. However, the evaluation of the effect of liquefaction owing to earthquake ground motion is not shown in **Fig. 5.2.2;** therefore, it is necessary to appropriately investigate the potential for liquefaction and the measures against it (refer to **Part II, Chapter 7 Ground Liquefaction**).



*1: Evaluation of the effect of liquefaction and settlement is not shown in the diagram; therefore, it is necessary these effects separately.

*2: Verification shall be carried out for high earthquake-resistance facilities against Level 2 earthquake ground motion.

Fig. 5.2.2 Example of the Sequence of Performance Verification of a Piled Pier

(4) In principle, the performance verification method presented in this section will be examined based on the condition that the effect of deformation of the earth-retaining section and other such sections will not be transmitted to the frame. Therefore, it is necessary to adapt the structure dimensions and construction by considering this fact. For example, as the earth-retaining section or the reclaimed land sinks, a part or the whole of the piled pier may cave in or lateral flow may occur; therefore, it is necessary to take measures to ensure that the actions caused by this incident will not transmit to the main body of the piled pier. It is also necessary to take various measures to ensure that the actions caused by deformation of the earth-retaining section and other such sections at the time of an earthquake ground motion will not be transmitted to the superstructure of the piled piers through the access bridges and that the significant deformation of the ground around a pile to the sea side will not negatively affect the piles.

(5) For the variable situation in case of Level 1 earthquake ground motion, it is possible to perform verification by obtaining the natural periods of the piled pier based on frame analysis and by further calculating the seismic coefficient for verification based on the obtained natural periods and the acceleration response spectrum. For open-type wharves on vertical piles other than high earthquake-resistant facilities, the verification of the accidental situation in case of Level 2 earthquake ground motion can be omitted.

(6) While verifying the performance of open-type wharves on the vertical piles, the cross-section is normally set with respect to actions other than that of Level 2 earthquake ground motion; further, the seismic performance is verified with respect to Level 2 earthquake ground motion. This is because the performance verification is performed based on the yield stress of the steel pipe piles for the verification of a variable situation with respect to the action of ships and Level 1 earthquake ground motion; however, a verification method that considers the extent of damage to the

piled pier is used for performing the seismic performance verification of seismic-resistant structures with respect to Level 2 earthquake ground motion.

(7) When cargo-handling equipment, such as container cranes, is to be installed on an open-type wharf on vertical piles, it is preferable to install it in such a manner that all its feet are positioned on either the pile-supported section or the earth-retaining section. For example, if one foot of the cargo-handling equipment is positioned on the pile-supported section and another foot is positioned on the earth-retaining section, the equipment will become susceptible to the adverse effects caused by uneven settlement and ground motions owing to the difference between the response characteristics of the two sections. When it is unavoidable to position one foot on the pile-supported section and another foot on the earth-retaining section, sufficient foundation, such as foundation piles, should be provided to prevent uneven settlement owing to the settlement on the earth-retaining section. In this case, in general, the fixed foot of the cargo-handling equipment, such as the portal crane, should not be installed on the pile-supported section. While installing the cargo-handling equipment, such as the container cranes, on the pile-supported section, seismic response analysis should be performed by considering the coupled oscillation of the cargo-handling equipment and the open-type wharf.

(8) In general, the open-type wharves on vertical piles possess structural types that can be adversely affected by the performance of the members because of material deterioration, including the chloride-induced corrosion of the concrete material and the corrosion of the steel material. Therefore, it is necessary to take maintenance through the service life into account sufficiently at the design stage.

① The superstructures of the piled piers are the concrete structures located in a severe chloride-induced corrosion environment, and it is necessary to pay close attention to maintenance through their service life. If the distance between the superstructure of a piled pier and the sea level is less and if it is difficult to secure work space, inspection and diagnosis, and measures are difficult to be conducted during the service life. In general, with respect to the superstructures of the piled piers, the members of ocean-side tend to deteriorate relatively early[2)]. However, depending on the distance between the superstructures of the piled piers and the sea level or on the form or layout of the earth-retaining sections, the superstructure's members which are located around the earth-retaining sections may easily deteriorate owing to the wave-breaker actions. It is necessary to consider such factors while setting the maintenance level during the designing stage of the superstructures of piled piers. The superstructures of the piled piers shall be designed based on **Part III, Chapter 2, 1.2.4 Examination Concerning the Time Worn of Performance.**

② The steel pipe piles and steel sections, which compose the piled piers, are located in a severe corrosive environment. Therefore, when the steel members are designed for making the piled piers, appropriate corrosion protection must be performed based on **Part III, Chapter 2, 1.3.4 Examination Concerning the Change of Performance Over Time.**

③ With respect to the piled piers located in a severe environment, it is desirable that labor saving and rationalization should be considered for maintenance. Therefore, inspection holes and inspection scaffolding may be arranged, and sensors for monitoring may be installed to ensure that inspection and diagnosis, and measures may be easily conducted during the service life. Please refer to **reference 3)**, which includes case studies that consider maintenance at the design stage.

④ If the backfilling material could be eliminated from the earth-retaining sections, preventive measures will be adopted according to the structural types of earth-retaining sections by referring to **Part III, Chapter 5, 2.2. Gravity-type Quaywalls** and **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls.**

5.2.2 Setting of the Basic Cross-section

(1) The size of a deck block, the distances between the piles, and the number of pile rows shall be appropriately determined by considering the following:

① apron width;
② location of the sheds;
③ seabed (especially, the slope stability);
④ existing revetments;
⑤ matters related to construction work, including the concrete casting capacity; and
⑥ surcharges (especially, the crane specifications).

(2) The larger the size of one deck block, the bigger the rigidity of the structure will be to the fender reaction force, tractive force, and so on. Even though this is preferable, it becomes weaker against uneven settlement on the other hand. In addition, the size is limited because of the concrete casting capacity. The normal length of one deck block of large-scale wharves is approximately 20 to 30 m in Japan.

(3) In general, the distances between the piles and the number of pile rows will be decided based on the shape of the cross-section of piles, economic comparison between various cases, and examination of construction restrictions. In case of a wharf that is planned to be equipped with a rail mounted crane, an unloader, and so on, construction is often restricted because of the gauge and action status of such equipment. In a case in which large quay cranes are to be installed for ships of the 10,000 DWT class, the piles are usually designed to be placed in intervals of 5 m with 3–4 pile rows in the cross-section. If the superstructures are manufactured from cast-in-place reinforced concrete, the distance between the piles shall be ca. 4–6 m because it is restricted by the concrete casting construction. The distance between the piles could be larger in case of pre-stressed concrete.

(4) The dimensions of the superstructure of the open-type wharf will be appropriately determined by considering the following:

① the distances between the piles, number of pile rows, and shapes and dimensions of piles;
② construction problem of shattering forms and scaffold;
③ ground conditions;
④ arrangement of the mooring posts; and
⑤ arrangement, shape, and dimensions of the fenders.

(5) If the concrete pavement is placed on piled piers after the construction of the piled pier floorboards, cracks may occur in the concrete pavement (refer to **Part III, Chapter 5, 9.18 Apron**).

(6) Assumptions Regarding the Seabed Condition

① Determination of the slope gradient

(a) When an earth-retaining structure is provided behind the slope, the position of the earth-retaining structure should be appropriately determined by considering the stability of the slope.

(b) It is necessary to examine the stability of the slope with respect to the circular slip failure. When an earth-retaining structure is installed behind the slope, it is preferable that the structure is not constructed in front of the slope surface from the toe of the slope at the slant angle indicated by **equation** (**5.2.1**) (see **Fig. 5.2.3**).

$$\alpha = \phi - \varepsilon \tag{5.2.1}$$

where

α : angle between the slope and the horizontal surface (°),

ϕ: : angle of shear resistance of the main material forming the slope (°),

$\varepsilon = \tan^{-1} k_h'$, and

k_h' : apparent horizontal seismic coefficient

When the slope contains a hard mudstone or rock, **equation (5.2.1)** may not be applied. For the seismic coefficient for verification to calculate the apparent horizontal seismic coefficient, the value calculated in the analysis of the earth-retaining section may be used. Refer to **Reference (Part III), Chapter 1, 1 Particularities Concerning the Seismic Coefficient for Verification** for the calculation of the seismic coefficient to verify the presence of the earth-retaining section.

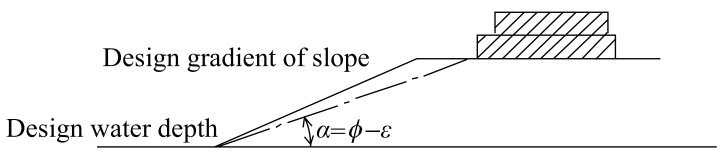


Fig. 5.2.3 Position of the Earth-Retaining Structure on the Slope

(c) The angle set in **equation (5.2.1)** in (b) is a restriction when a structure will be constructed behind the top of this slope. The slant angle as an actual design cross-section is usually steeper than α. For example, if the foundation of the earth-retaining section and the slope face is comprised of rubble, the ratio is usually set to be approximately 1:1.5 to 1:2. This is to take the top of the slope at the front of the earth-retaining structure as much as possible to ensure that the effect of scouring or the effect of local collapse does not reach the front toe of the structure.

② **Virtual ground surface**

(a) In the calculation of the lateral resistance and the bearing capacity of the piles, a virtual ground surface will be assumed at an appropriate elevation shown in the following step for each pile.

(b) When the inclination of the slope is considerably steep, the virtual ground surface for each pile to be used in the calculation of the lateral resistance or the bearing capacity may be set at an elevation that corresponds to half of the vertical distance between the surface of the slope at the pile axis and the seabed as shown in **Fig. 5.2.4**.

(c) The calculation method of lateral resistance of piles used for analysis of open-type wharves on vertical piles is primarily for the horizontal ground surface. Therefore, if the lateral resistance of the piles, which are driven on the slopes as piles of open-type wharves, will be calculated, some kind of correction will be required. To simplify the calculation method, this will generally be set as shown in (b).

(d) The application of this method is not appropriate for wide piers having a width of more than 20 m and having an extremely long slope. In such cases, it is preferable that other methods, such as that denoted in **reference 4)**, will be adopted.

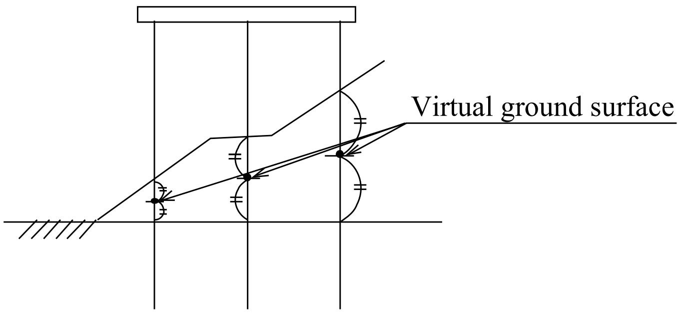


Fig. 5.2.4 Virtual Ground Surface

(7) Coefficient of the Lateral Subgrade Reaction

① In the calculation of the lateral resistance of piles, it is preferable to obtain the coefficient of the lateral subgrade reaction of the subsoil through lateral loading tests of the piles in-situ. If no tests are conducted, the coefficient of the lateral subgrade reaction of the subsoil may be estimated by appropriate analytical methods derived from the lateral resistance tests as shown in ③.

② Some measured data are available about the coefficient of the lateral subgrade reaction that is obtained by the loading tests in which the lateral loads are applied to the piles of open-type wharves up to the yield points. Although some of these data are related to the *N*-value, the coefficient of lateral subgrade reaction cannot be precisely estimated from the *N*-value. Thus, it is preferable to estimate the coefficient by performing the lateral loading tests in-situ.

③ When lateral loading tests of piles are not conducted because of small-scale construction works or time constraints, the coefficient of the lateral subgrade reaction of the subsoil may use the mean value of the minimum value and central value obtained from the lateral resistance tests. While using Chang's method, **equation (5.2.2**) may be utilized, and **Part III, Chapter 2, 3.4.6 Deflection of Piles under the Action of Lateral Load** can be referenced. However, some in-situ measurement data indicate that the coefficient of the lateral subgrade reaction of the rubble stones is smaller than the estimate by **equation (5.2.2)** using Chang's method. In this case, it is recommended to set the coefficient of lateral subgrade reaction as 3,000–4,000 kN/m^3 in Chang's method.

$$k_{CH} = 1{,}500N \tag{5.2.2}$$

where

k_{CH} : coefficient of horizontal subgrade reaction (kN/cm^3) and

N : average N-value of the ground to a depth of approximately $1/\beta$.

④ The coefficient of the lateral subgrade reaction denoted in **equation (5.2.2)** is a static coefficient of the subgrade reaction and may be used while verifying the stress of piles by static frame analysis and other such methods. However, it cannot be used for liquefied ground. In addition, if the natural periods of the piled piers are to be calculated, doubling **equation (5.2.2)** and using the actual ground surface instead of the virtual ground surface will yield values nearer to the actual values [6) 7)]. For the calculation of the natural periods of piled piers, refer to **Part III, Chapter 5, 5.2.3 Actions.**

⑤ Because soil is not an elastic body, the relation between the lateral loads to piles and displacement is generally nonlinear. As the lateral loads become larger, k_{CH} decreases. To derive **equation (5.2.2)** from the result obtained using the lateral resistance tests, it is preferable to provide relatively low k_{CH}.

(8) Virtual Fixed Point

① With respect to an open-type wharf on vertical piles, the virtual fixed points of the piles may be considered to be located at a depth of $1/\beta$ below the virtual ground surface. The value of β, which denotes the characteristic value of the piles, is calculated by **equation (5.2.3)**.

$$\beta = \sqrt[4]{\frac{k_{CH}D}{4EI}} \quad (\mathrm{m}^{-1}) \tag{5.2.3}$$

where

k_{CH} : lateral subgrade reaction coefficient (kN/m^3) (calculated by **equation (5.2.2)**),

D : diameter or width of the pile (m), and

EI : flexural rigidity of the pile (kN·m^2).

② The method of usage of virtual fixed points is used for simple calculation of the pile head moments. If the projected length of the piles from the sea bottom is long, **equation (5.2.3)** can be approximately applied[8)]. The pile head moments and other such factors may be derived using methods other than those shown here.

③ If the virtual fixed-point method based on Chang's method is used, the locations of the virtual fixed points can be decided using the following methods corresponding to Chang's method:

(a) the method in which the first immobile point in Chang's method is set as the virtual fixed point;

(b) the method in which the virtual fixed point is set so that the pile head reaction and the pile head flexural moment using Chang's method are the same as those in beams with fixed ends;

(c) the method in which the virtual fixed point is set so that the pile head displacement and the pile head flexural moment using Chang's method are the same as those in beams with fixed ends; and

(d) the method in which the virtual fixed point is set so that the pile head reaction and the pile head displacement using Chang's method are the same as those in beams with fixed ends.

① is based on the step (b) of the aforementioned method. As the depth of the virtual fixed points calculated by the methods (a) to (d) exhibits a little variation, the pile head displacement also exhibits some difference. However, the depth is within a sufficiently permissible range from an engineering perspective.

④ The virtual fixed-point method based on the PHRI method uses the PHRI method instead of Chang's method to calculate the lateral resistance of each pile. It sets the virtual fixed points so that the pile head displacement calculated by the PHRI method becomes equal to the head displacement of the beam with fixed ends and a virtual fixed point. As the PHRI method considers the nonlinearity of the lateral subgrade reaction of piles, the virtual fixed points stated here are described to depend on the horizontal forces acting on the vertical piles [9) 10) 11) 12)].

(9) It is desirable that the fenders and mooring posts are arranged so that the actions inclining to one deck block do not occur when possible.

(10) If the fender reaction force through fenders or the tractive force transmitted from the mooring posts is added with eccentricity, the reactions by the rotation of deck blocks become larger. Therefore, it is reasonable to install a fender or a mooring post at the center of a deck block so that the inclined action does not occur when possible (refer to **Fig. 5.2.5**).

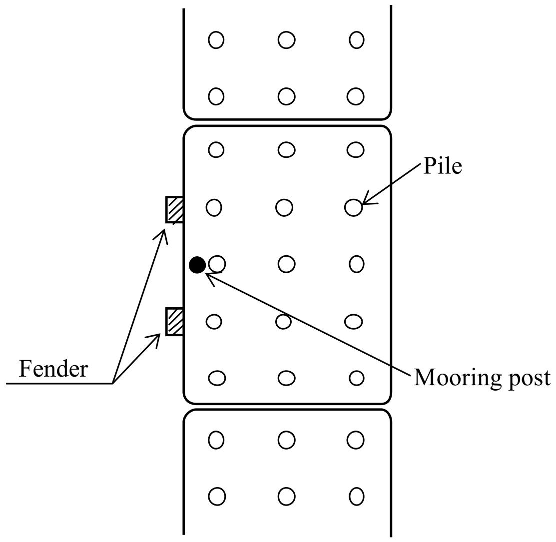


Fig 5.2.5 Arrangement of the Ancillary Facilities

5.2.3 Actions

(1) Types of actions to be taken account of for each design state

The following actions will be considered in the stability verification of open-type wharves on vertical piles with respect to each design state.

① **Permanent situation**

If the earth-retaining sections are to be constructed behind piled piers, the actions, which will be considered in a permanent situation, will be adequately set according to the structure types of the earth-retaining sections.

② **Variable situation**

The following actions shall be considered to be the dominating actions:

(a) Level 1 earthquake ground motion;

(b) ship tractive force and ship berthing force;

(c) surcharge (including surcharge during cargo handling);

(d) surcharge that repeatedly generates actions; and

(e) variable waves.

For understanding Level 1 earthquake ground motion, readers can refer to **Part II, Chapter 6, 1.2 Level 1 Earthquake Ground Motions Used in Performance Verification of Facilities**. With respect to the ship tractive force and ship berthing force, **Part II, Chapter 8, 2 Actions Caused by Ships** can be referred. With respect to the surcharges during cargo handling, **Part II, Chapter 10, 3 Surcharge** can be referred. With respect to the variable waves, **Part II, Chapter 2 Setting of Wave Conditions** can be referred.

③ **Accidental situations**

Level 2 earthquake ground motion will be considered as the dominating action. With regard to the setting of Level 2 earthquake ground motion, **Part II, Chapter 6, 1.3 Level 2 Earthquake Ground Motions used in Performance Verification of Facilities** can be referred.

(2) Points to be noticed regarding setting the actions

① With respect to the actions on the earth-retaining sections, each chapter of this part can be referred to according to their types, and the self-weight of the access bridges is added.

② For calculating the self-weight of the reinforced concrete superstructures, each part of the dimensions is assumed based on the dimensions of the superstructure, and the volume is calculated on them. The self-weight can be obtained by multiplying the unit weight obtained from **Part II, Chapter 10, 2 Self-Weight** by the volume. However, by considering that the superstructure is established not at the design stage of the basic cross-sections but during the performance verification of the structure members, if the performance verification of the piles of open-type wharves on vertical piles is conducted, 21 kN per 1.0 m^2 of the deck area of the superstructure of the piled pier may be assumed for the calculation of the self-weight of the reinforced concrete superstructures. Further, this weight per unit area of the superstructure is calculated from the case examples of large-scale mooring facilities. However, this value can be used for the small-scale mooring facilities as well if the distances between the piles and the superstructure dimensions are not drastically changed.

③ In peculiar piled piers, it is inappropriate to use the self-weight of the superstructures shown above. If the distances between the piles are larger than the normal values, if large cargo-handling equipment, such as container wharves, will be installed on the piled piers, or if piles on the sea side or the land side of the piled piers are steel sheet piles that are also used as earth retainer, it is preferable to decide the self-weight of the superstructure by separately calculating the volume.

④ At the site expected to be subjected to waves, the following items should be examined with respect to the wave uplift on the superstructure of the piled pier and the access bridge:

(a) stability of the access bridges and pulling resistance of the piles against the uplift

(b) member strength of the superstructures and access bridges against uplift.

For uplift, refer to Part II, Chapter 2, 6.4 Wave Action Acting on Structure near the Water Surface.

⑤ The static loads may be determined in accordance with **Part II, Chapter 10, 3.1 Static Load**. The earthquake inertia forces owing to static loads may normally be considered to act on the upper surface of the deck slab. However, when the center of gravity of the static loads is located at an especially high elevation, it is important to consider the height of the center of gravity as the point of application of the horizontal force.

⑥ Live loads should be determined in accordance with **Part II, Chapter 10, 3.2 Live Load**. The seismic force that can be attributed to a rail mounted crane should be calculated by multiplying its self-weight with the seismic coefficient for verification, and the force can be considered to be transmitted from the wheels of the crane to the pile-supported section. It is also necessary to perform seismic response analysis by considering the coupled oscillations of the cargo-handling equipment and the open-type wharf (refer to **Part III, Chapter 7, 2.2.3 Verification of Earthquake-Resistant Performance**). In this case, the ground motion will be applied in the form of a time-series seismic wave profile. Further, the wind load acting on the crane may be determined in accordance with **Part II, Chapter 2, 2.3 Wind Pressure**.

⑦ It is generally preferable to decide the coexistence of static and live loads by considering the utility form of the wharf.

⑧ The fender reaction force used for the performance verification of piled piers can be calculated in accordance with **Part II, Chapter 8, 2.2 Actions Caused by Ship Berthing** and **Part II, Chapter 8, 2.3 Actions Caused by Ship Motions**, and **Part III, Chapter 5, 9.2 Fender System**.

⑨ The tractive force of the vessels can be determined in accordance with **Part II, Chapter 8, 2.4 Actions due to Traction by Ships**. In several cases, one bollard is installed on one deck block.

⑩ When rubber fenders are installed as a damper on an ordinary large wharf with a unit deck block of 20 to 30 m in length, a common practice is to provide two rubber fenders on one block. In several cases, fender intervals of 8 to 13 m are used. The berthing behavior of various sizes of ships has been examined by installing 1.5-m long rubber fenders on an ordinary large wharf. The results of the examination have revealed that it is appropriate to calculate the berthing force based on the assumption that the ship's berthing energy is absorbed by one fender. Therefore, the reaction force may be basically calculated on the assumption that the berthing energy is absorbed by one fender while using rubber fenders as the damper. However, this is not applicable when fenders are continuously installed along the face line of a wharf.

⑪ The berthing energy is absorbed by the displacement of the main structure of the pier. However, it is a common practice to not consider this because the energy absorbed by the main structure of the pier accounts for less than 10% of the total berthing energy in several cases.

⑫ **Fig. 5.2.6** denotes an example of the displacement–energy curve and the displacement–reaction force curve of the rubber fender. If a single fender absorbs the berthing energy E_1, the corresponding fender deformation δ_1 can be obtained. Further, using the other curve, the corresponding reaction force acting on the pier can be obtained as $H_1(\delta_1 \rightarrow C \rightarrow H_1)$. However, if fenders are installed too close to each other and if the berthing energy is absorbed by two fenders, the berthing energy acting on one fender becomes $E_2 = E_1/2$, and the corresponding fender deformation becomes δ_2. As can be obtained from **Fig. 5.2.6** ($\delta_2 \rightarrow D \rightarrow H_2$), the reaction force acting on the pier in the two-fender case is almost the same as that generated in the single fender case because of the characteristics of the rubber fender. Thus, if two fenders are installed close to each other, the horizontal reaction force acting on the pier becomes $2H_2 \approx 2H_1$, which indicates that the horizontal force to be used in the performance verification doubles. Therefore, it is preferable to carefully consider this behavior of the reaction force in the performance verification and determination of the location of fenders while using the fenders that have such characteristics.

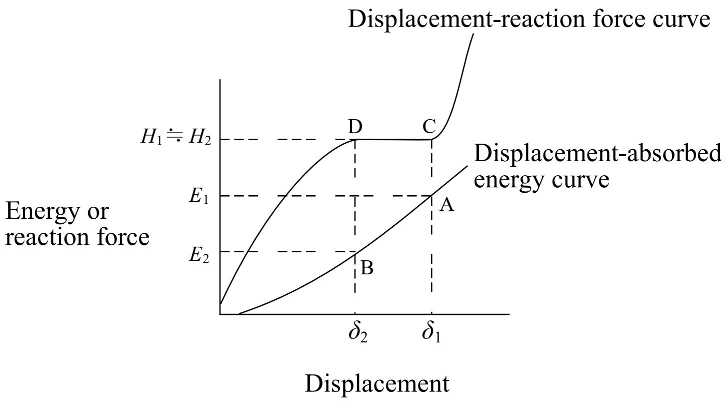


Fig. 5.2.6 Curve of Rubber Fender Characteristics

⑬ When it is considered necessary to examine the rotation of the piled pier unit while evaluating the actions, the verification should take this into consideration. In this case, the distribution of forces on each pile may be evaluated as described below.

(a) When the symmetry axis of the piled pier unit is perpendicular to the face line of the wharf and when the direction of action of the horizontal force is parallel to the symmetry axis as depicted in **Fig. 5.2.7**, the horizontal force may be calculated using **equation** (**5.2.4**). Further, the horizontal force without considering the rotation of the piled pier unit may be calculated by **equation** (**5.2.4**) with e = 0.

$$H_i = \frac{K_{H_i}}{\sum_i K_{H_i}} H + \frac{K_{H_i} x_i}{\sum_i K_{H_i} x_i^2} eH \qquad \textbf{(5.2.4),}$$

where

H_i : horizontal force on pile (kN),

K_{Hi} : horizontal spring constant of pile (kN/m),

$$K_{H_i} = \frac{12EI_i}{\left(h_i + \frac{1}{\beta_i}\right)^3},$$

h_i : vertical distance between the pile head and the virtual ground surface (m),

β_i : inverse of the distance between the virtual ground surface and the virtual fixed point of the pile, (m^{-1})

EI_i : flexural rigidity of the pile (kN·m^2),

H : horizontal force acting on the unit (kN),

e : distance between the block's symmetry axis and the horizontal force (m), and

x_i : distance between the unit's symmetry axis and each pile (m).

The subscript i refers to the i-th pile.

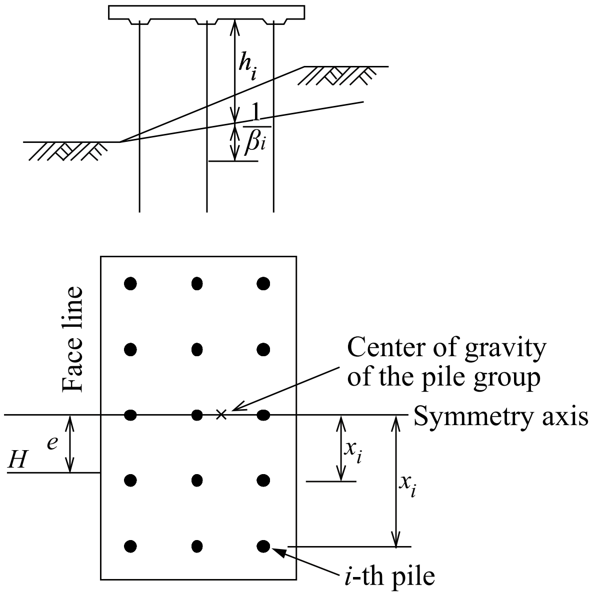


Fig. 5.2.7 Distance between the Center of Gravity of the Pile Group and Individual Piles

(b) The row of piles bearing the maximum total horizontally distributed forces is subject to the verification.

(c) While obtaining K_{Hi}, it is necessary to appropriately set the coefficient of the subgrade reaction in the lateral direction of the ground and calculate β.

(d) For the methods to obtain the axial force of a piled pier, the pile head moment, and other such factors, the method in Reference 13) can be referred.

⑭ **Ground Motion used in the Performance Verification of Seismic-resistant and Seismic Coefficient for Verification**

(a) The ground motion used in the performance verification of seismic resistance is set by considering the effect of the surface strata using ground seismic response analysis. It is necessary to use a seismic response analysis code capable of appropriately evaluating the amplification of ground motions in soft ground (refer to **Part II, Chapter 6, 1.2.3 Seismic Response Analysis of Surface Ground**).

(b) Using a one-dimensional seismic response analysis as described in **Part II, Chapter 6, 1.2.3 Seismic Response Analysis of Surface Ground**, the acceleration time history at a position $1/\beta$ below the virtual ground surface can be calculated with the acceleration time history of the ground motion set at the seismic bedrock as the input ground motion. While calculating the acceleration time history, the average depth of the $1/\beta$ ground point for each pile may be obtained, as depicted in **Fig. 5.2.8**. From the acceleration response spectrum obtained in this manner, the response accelerations corresponding to the natural periods of the piled pier are calculated, and the value obtained by dividing this with the gravitational acceleration can be regarded as the characteristic value of the seismic coefficient for verification. A damping factor of 0.2 may be used while calculating the acceleration response spectrum.

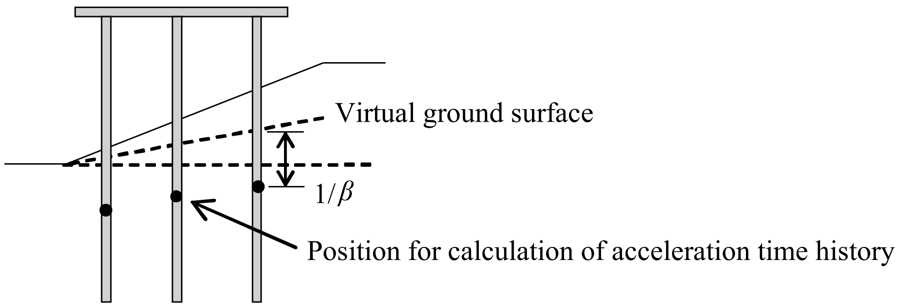


Fig. 5.2.8 Positions for the Calculation of the Earthquake Ground Motions

(c) An example of a typical procedure for setting the seismic coefficient for performing verification is denoted in **Fig. 5.2.9**. While verifying the seismic performance of the earth-retaining parts using the seismic coefficient method, the structural characteristics are observed to be different from those of the piled pier; therefore, the seismic coefficient indicated here may not be used. To calculate the seismic coefficient for verification of the earth-retaining parts, refer to (g).

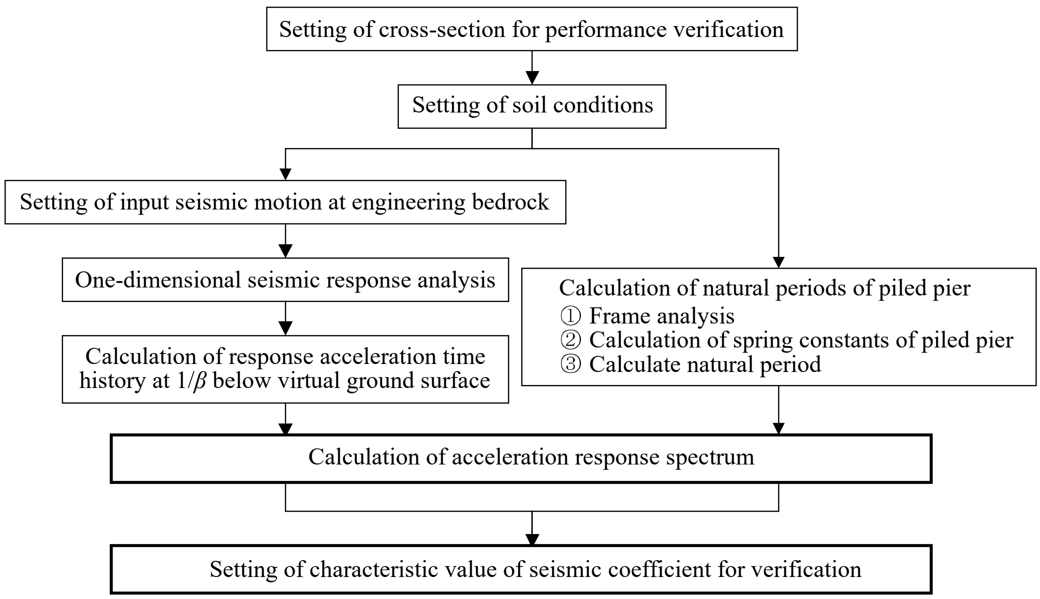


Fig. 5.2.9 Typical Procedure for Setting of the Seismic Coefficient for Verification

(d) The natural periods of the piled pier may be calculated using frame analysis. If the relation between the displacement and the load is obtained from the frame analysis, as depicted in **Fig. 5.2.10**, when minute loads act on the piled pier, the spring constants of the piled pier can be set, and the natural periods can be obtained from **equation (5.2.5)**. While calculating the natural periods, the values that are twice the ground spring constants obtained using **equation (5.2.2)** are often used.

$$T_s = 2\pi\sqrt{\frac{W}{gK}} \qquad \textbf{(5.2.5)}$$

where

T_s : natural period of the piled pier (s),

W : self-weight and static load during an earthquake per one row of pile group (kN),

g : gravitational acceleration (m/s^2), and

K : spring constant of the piled pier (kN/m)

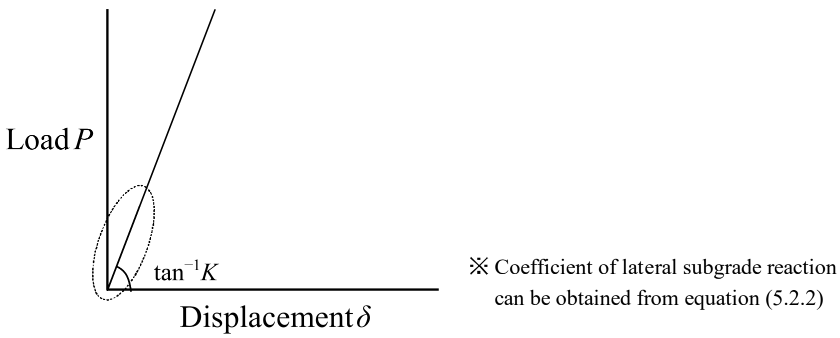


Fig. 5.2.10 Relation between Load and Displacement from Frame Analysis

(e) If it is judged that the ratio of the sum of the self-weight of the piles, the weight of water in the piles, and the weight corresponds to the mass around the piles added by the earthquake ground motion to the sum of the self-weight of the superstructures of piled piers and surcharges at the time of earthquake is relatively large, it is preferable that all these masses are considered in a flame analysis to estimate the natural period and the cross-section force.

(f) The natural period of the piled pier obtained from the spring constants of the piled pier by frame analysis usually involves some amount of errors. Therefore, if the value in the acceleration response spectrum corresponding to the natural period is a local minimum, the seismic coefficient for verification may be underestimated, and this value should not be applied as it is. In addition, **Reference 14)** indicates the possibility that the natural periods based on **equations** (**5.2.2**) and (**5.2.5**) may be approximately twice as long as the natural periods calculated by the two-dimensional earthquake response analysis etc.. Therefore, it is preferable that the spectral value should be determined to calculate the seismic coefficient for verification with a certain range of natural periods. However, this does not deny the importance of avoiding a local maximum in the acceleration response spectrum caused by the site effects. In case the natural period of the piled pier corresponds to a local maximum in the acceleration response spectrum, it is very likely that the cross-section will not be optimum from the viewpoint of the seismic resistance performance and cost. It is necessary to devote attention to this point for setting the cross-section for verification.

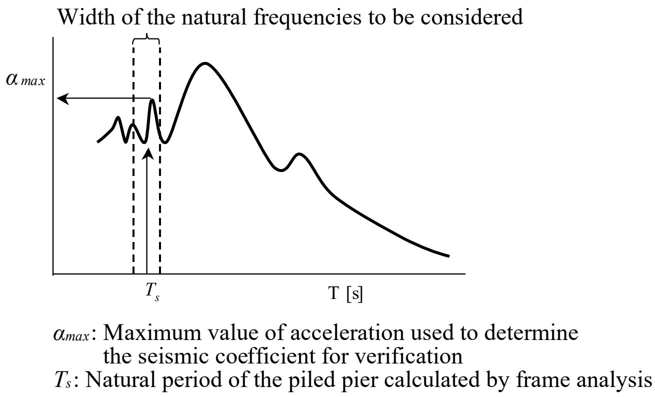


Fig. 5.2.11 Consideration of the Natural Period in the Acceleration Response Spectrum

(g) Seismic coefficient for verification used in the performance verification of the seismic resistance of the earth-retaining sections

The performance verification of the seismic resistance of the earth-retaining sections can be performed by directly evaluating the deformation of the earth-retaining section using nonlinear effective stress analysis etc.. However, simple methods, such as the seismic coefficient method, can also be used. In this case, it is necessary to appropriately set the seismic coefficient for verification used in the performance verification corresponding to the amount of deformation of the facility by considering the effect of the frequency characteristics of the ground motion and the duration. The normal procedure for calculating the seismic coefficient for verification is as shown in **Reference (Part III), Chapter 1, 1 Particularity Concerning Seismic Coefficient for Verification**.

5.2.4 Performance Verification Regarding Open-Type Wharves on Vertical Piles

(1) Performance verification items to be considered in the performance verification of open-type wharves on vertical piles

① In the performance verification of the open-type wharves on vertical piles, the necessary items will be appropriately investigated and set as necessary. Performance verification under Level 2 earthquake ground motion shall be in accordance with **Part III, Chapter 5, 2.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situation**. For the cross-sectional forces in the superstructure and fatigue failure, refer to **Part III, Chapter 5, 2.6 Performance Verification of Structural Members**.

② In the performance verification of the piled pier section of the open-type wharves on vertical piles as described below, no load transmission is considered from the earth-retaining section to the wharves. A piled pier is a very flexible structure if it is affected by deformation of the ground; hence, the piled pier section will be structurally independent of the earth-retaining section. However, in case the cross-sectional dimensions are such that it is not possible to eliminate the effect from the earth-retaining section, it is necessary to perform the verification using a method by considering the interaction between the earth-retaining section and the piled pier section because of the physical restrictions due to ground condition.

③ In the performance verification for Level 1 earthquake ground motion, the seismic coefficient for verification is calculated from the acceleration response spectrum values corresponding to the natural periods of the piled pier; thus, when the dimensions of the piles are not determined, it is not possible to determine the natural periods of the piled pier. Therefore, the dimensions of the piles are assumed, and the seismic coefficient for verification is calculated from the acceleration response spectrum corresponding to the natural periods; further, the verification is conducted. If the performance requirements are not satisfied, the pile dimensions are changed, and the same calculation has to be repeated.

④ The performance verification of the deformation may be conducted by setting an appropriate limiting value considering the dynamic deformation of the piled pier. For example, the amount of deformation required to ensure that the access bridge does not fall down may be considered to be the limiting value. In this case, it is appropriate to use the response displacement by considering the dynamic action, such as the displacement response spectrum, and not the static action.

(2) Performance Verification for the Stability of the Earth-retaining Section

① The examination of the structural stability of the earth-retaining section of the open-type wharf on vertical piles can be performed in accordance with the performance criteria prescribed in **Part III, Chapter 5, 2.2 Gravity-type Quaywalls,** and **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls** depending on its structural type.

② The superstructure and the earth-retaining section of the open-type wharf should be connected by a simply supported slab having clearances on its both ends or the buffer material provided on the both ends of slab to prevent the transmission of the forces acting on the earth-retaining section to the superstructure. It is also preferable to prepare various measures against the relatively uneven settlement between the wharf and the earth-retaining section. Furthermore, the clearance between the superstructure and the earth-retaining section should be appropriately determined by considering the dynamic deformation of the superstructure and the earth-retaining section.

③ The stability of the earth-retaining section of the open-type wharf on vertical piles should be examined against the circular slip failure by applying **Part III, Chapter 2, 4.2.1 Stability Analysis by Circular Slip Failure Surface.**

(3) Performance verification for the stresses in the piles under variable situation (surcharge, ship berthing force, tractive force by ship, and Level 1 earthquake ground motion)

① The stresses occurring in the piles of a piled pier may be verified using **equation** (**5.2.6**). In the following equations, γ denotes the partial factor corresponding to the suffix, where the suffixes k and d indicate the characteristic value and the design value, respectively. As for the partial factors in the relevant equations, the values shown in **Table 5.2.1** can be used. The values shown as "-" in **Table 5.2.1** indicates that the values may be verified using the values enclosed in parentheses ( ) to ensure convenience. If the axial forces are tensile, S_k and R_k can be calculated using **equations** (**5.2.6 (b-1)**) and (**5.2.6 (b-2)**), respectively, and each value should satisfy **equation** (**5.2.6**).

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \tag{5.2.6}$$

(a) When the axial forces are compressive,

$$S_k = \left(\frac{\sigma_{c_k}}{red} + \sigma_{bc_k} \right) \qquad R_k = \sigma_{by_k} \tag{5.2.6 (a)}$$

(b) When the axial forces are tensile,

$$S_k = \sigma_{t_k} + \sigma_{bt_k} \qquad R_k = \sigma_{ty_k} \tag{5.2.6 (b-1)}$$

$$S_k = -\sigma_{t_k} + \sigma_{bc_k} \qquad R_k = \sigma_{by_k} \tag{5.2.6 (b-2)}$$

where

red : coefficient defined as the value of the axial compressive yield stress (refer to **Table 5.2.2**) divided by the characteristic value of the yield stress,

σ_t and σ_c : tensile stress due to the axial tensile forces acting on the cross-section and compressive stress due to the axial compressive forces, respectively (N/mm^2),

σ_{bt} and σ_{bc} : maximum tensile stress and maximum compressive stress because of the flexural moment acting on the cross-section, respectively (N/mm^2),

σ_{ty} and σ_{cy} : axial tensile yield stress and axial compressive yield stress, respectively (N/mm^2),

σ_{by} : bending compressive yield stress (N/mm^2),

R : resistance term (N/mm^2),

S : load term (N/mm^2),

γ_R : partial factor that is to be multiplied with the resistance term,

γ_S : partial factor that is to be multiplied with the load term, and

m : adjustment factor.

Table 5.2.1 Partial Factor Used for Verification of the Stresses Occurring in the Piles of a Piled Pier

Verification target	Installation water depth	Partial factor to be multiplied with resistance term γ_R	Partial factor to be multiplied with load term γ_S	Adjustment factor m
Stress occurring in the piles of a piled pier (variable action due to surcharge (during work))	All water depth	– (1.00)	– (1.00)	1.67
Stress occurring in the piles of a piled pier (variable action due to surcharge (during storm))	All water depth	– (1.00)	– (1.00)	1.12
Stress occurring in the piles of a piled pier (variable action due to tractive force by ship)	All water depth	– (1.00)	– (1.00)	1.67
Compressive stress occurring in the piles of a piled pier (variable action due to ship berthing force)	Less than 12.0m	0. 97	1.34	- (1.00)
	12.0m and above	1. 01	1.29	
Tensile stress occurring in the piles of a piled pier (variable action due to ship berthing)	All water depth	– (1.00)	– (1.00)	1.67

Verification target	Installation water depth	Partial factor to be multiplied with resistance term γ_R	Partial factor to be multiplied with load term γ_S	Adjustment factor m
Stress occurring in the piles of a piled pier (variable action due to Level 1 earthquake ground motion)	All water depth	– (1.00)	– (1.00)	1.12

② The partial factors are used for the verification of the compressive stresses occurring in the piers of a piled pier at the time when a ship berth in **Table 5.2.1** is the coefficient obtained by the conducted code calibrations so that the obtained dimensions are averagely equivalent as the cross-sections of open-type wharves on vertical piles designed using the previous design methods[12) 16)]. In addition, partial factors related to other design states set by referring to the allowable stresses of the steel members in the previous design methods.

③ Each stress in the equation shown in ① can be calculated using **equation** (**5.2.7**). The suffix k indicates the characteristic value.

$$\sigma_{t_k} = \frac{P_k}{A}, \quad \sigma_{c_k} = \frac{P_k}{A}, \quad \sigma_{bt_k} = \frac{M_k}{Z}, \quad \sigma_{bc_k} = \frac{M_k}{Z} \tag{5.2.7}$$

where

A : cross-sectional area of the piles (mm^2),

P : axial force on the pile (N),

Z : section modulus of the piles (mm^3), and

M : flexural moment of the piles (N·mm).

④ For the yield stress of piles, refer to **Part II, Chapter 11, 2 Steel**. The axial compressive yield stress may be calculated from the equation in **Table 5.2.2**. As for the effective buckling length of the members, the distance from the lower end of the superstructure to $1/\beta$ under the virtual ground surface may be used as denoted in **Fig. 5.2.12**. If verification is necessary for local buckling, the S**pecification and Commentary for Highway Bridges**[23)] may be referred.

Table 5.2.2 The Axial Compressive Yield Stresses

SKK400	SKK490
a) When $\frac{\ell}{r} \leq 19$ $\quad 235$	a) When $\frac{\ell}{r} \leq 16$ $\quad 315$
b) When $19 < \frac{\ell}{r} \leq 93$ $\quad 235 - 1.4\left(\frac{\ell}{r} - 19\right)$	b) When $16 < \frac{\ell}{r} \leq 80$ $\quad 315 - 2.1\left(\frac{\ell}{r} - 16\right)$
c) When $\frac{\ell}{r} > 93$ $\quad \dfrac{2.0 \cdot 10^6}{6.7 \cdot 10^3 + \left(\frac{\ell}{r}\right)^2}$	c) When $\frac{\ell}{r} > 80$ $\quad \dfrac{2.0 \cdot 10^6}{5.0 \cdot 10^3 + \left(\frac{\ell}{r}\right)^2}$

SM490Y		SM570	
a) When $\frac{\ell}{r} \leq 15$	355	a) When $\frac{\ell}{r} \leq 13$	450
b) When $15 < \frac{\ell}{r} \leq 76$	$355-2.6\left(\frac{\ell}{r}-15\right)$	b) When $13 < \frac{\ell}{r} \leq 67$	$450-3.7\left(\frac{\ell}{r}-13\right)$
c) When $\frac{\ell}{r} > 76$	$\dfrac{2.0\cdot 10^6}{4.4\cdot 10^3+\left(\frac{\ell}{r}\right)^2}$	c) When $\frac{\ell}{r} > 67$	$\dfrac{2.0\cdot 10^6}{3.5\cdot 10^3+\left(\frac{\ell}{r}\right)^2}$

ℓ: effective buckling length of the member (mm) and r: radius of gyration of the member gross cross-section (mm)

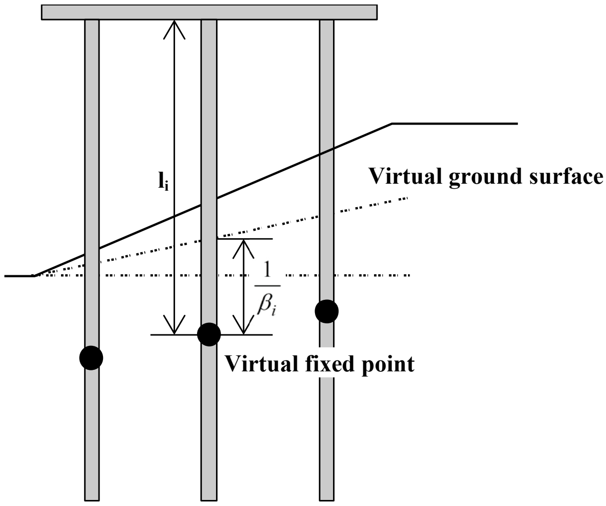


Fig. 5.2.12 Setting of the Effective Buckling Length

⑤ It is preferable to calculate the flexural moments on the piles for the directions both normal and parallel to the face line of the wharf. As in the example shown in **Fig. 5.2.1**, if the ground surface under the floor slab of the piled pier has a sloping surface, the flexural moments in the frontmost row of piles are mostly maximized when the ground motion acts in the direction parallel to the face line.

⑥ The superstructures of the piled piers were equipped with joints for each block interval; however, the horizontal displacements actually transmit each other. Thus, various actions, such as the tractive force and the berthing force, which do not simultaneously occur at individual locations of the whole mooring facility, occur not only in one block but are distributed in a certain section of the piled pier; therefore, it can be assumed that the stress of a pile would not become so dangerous as the stress verified for the normal direction to the face line. However, the earthquake ground movement actions work simultaneously in the whole piled piers and should, therefore, be considered.

(4) Performance verification of the bearing capacity in piles under design situations apart from the accidental situations considering Level 2 earthquake ground motion

① The bearing capacity of the piles in the piled piers can be verified using **equation** (**5.2.8**). The symbol γ is the partial factor corresponding to the suffix, where the suffixes k and d indicate the characteristic value and the design value, respectively. As for the partial factors in the relevant equation, the values shown in **Table 5.2.3** can be used. The values denoted as "-" in **Table 5.2.3** indicates that the values may be verified using the values enclosed in parentheses ( ) to ensure convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \tag{5.2.8}$$

where

R : resistance term (kN/m),

S : load term (kN/m),

γ_R : partial factor that is to be multiplied with the resistance term,

γ_S : partial factor that is to be multiplied with the load term, and

m : adjustment factor.

Table 5.2.3 Partial Factors Used for Performance Verification Regarding the Bearing Capacity in Piles

Verification target	Type of piles	Partial factor to be multiplied with resistance term γ_R	Partial factor to be multiplied with load term γ_S	Adjustment factor m
Bearing capacity of the open-type wharves on vertical piles (variable situation for surcharges during ship actions)	Pulling pile	– (1.00)	– (1.00)	3.00
	Pushing pile	– (1.00)	– (1.00)	2.50
Bearing capacity of the open-type wharves on vertical piles (variable situation during storms, high waves, and Level 1 earthquake ground motion)	Pulling pile	– (1.00)	– (1.00)	2.50
	Pushing pile (bearing pile)	– (1.00)	– (1.00)	1.50
	Pushing pile (friction pile)	– (1.00)	– (1.00)	2.00

② The characteristic value of the bearing capacity of the piles in piled piers can be appropriately estimated in accordance with **Part III, Chapter 2, 3.4.3 Axial Pushing Resistance of Piles** and **Part III, Chapter 2, 3.4.4 Axial Pulling Resistance of Piles**, corresponding to the ground characteristics and an analysis method for pile lateral resistance. In this case, for calculating the characteristic value of the bearing capacity of piles on a sloping surface, the soil strata below the virtual ground surface can be considered to be the effective bearing strata.

③ With respect to the virtual ground surface, refer to **Part III, Chapter 5, 5.2.2 Setting the Basic Cross-section**.

(5) Examination of the Embedment Length for Lateral Resistance

① The embedment length of each vertical pile may be appropriately determined in accordance with the method of analysis of the pile lateral resistance.

② The embedment lengths of the vertical piles are generally set to be $3/\beta$ below the virtual ground surface based on the results of the pile lateral resistance analyses. The value of β can be set in accordance with **Part III, Chapter 5, 5.2.2 Setting of Basic Cross-Section**.

③ Even though the methods for analyzing a single pile receiving lateral force include the PHRI method and Chang's method, it is preferable to use the PHRI method for the estimate described in **Part III, Chapter 2, 3.4.6 Deflection of Piles Receiving Lateral Load**. However, the analytical result of the behavior of piles when the actions occur is almost the same using both Chang's method and the PHRI method for piles with free length-like piled pier structure. Therefore, the virtual fixed-point method based on Chang's method may be used. In the virtual fixed-point method based on Chang's method described here, Chang's method is adapted for calculating the lateral resistance of a single pile. Further, to ensure that the pile head reaction and the pile head flexural moment set by the aforementioned method are the same as those of the beams with fixed ends, the leg length of the lower end embedded rigid frames should be set for each pile.

④ The method mentioned above in ② can be applied as an analysis method for a single pile receiving force from the perpendicular direction by setting $1/\beta$ in Chang's method as the virtual fixed point if stability analysis is conducted under horizontal force. Namely, Chang's method is the obtained solution if the pile length under the ground is assumed to be infinite. In addition, the range of finite underground pile length to which the method can be applied was examined, and it was observed that no large error occurs even if piles with finite lengths are treated as those with infinite lengths if the embedment length of piles is $3/\beta$ or longer.

If the range of approximation between the piles with infinite lengths and the piles with finite lengths is widened in Chang's method, an embedment length of up to $2/\beta$ can be accepted for each vertical pile. However, it is preferable to avoid an embedment length shorter than $2/\beta$ using the virtual ground surface.

⑤ Even when Chang's method is used, if the solution method obtained using the boundary condition of finite embedment length is adopted, it does not have to be in accordance with the method ②.

⑥ If the lateral resistance of the piles is analyzed based on the PHRI method, the minimum embedment length of piles may become $1.5l_{ml}$. Here, l_{ml} is generally the depth from the ground surface to the flexural bending moment second zero point of the fixed head piles.

(6) Other examination

① **Examination of the Pile Joints**

(a) When a pile joint is required in a pile, it is preferable to ensure that the pile can maintain its stability against the impact stress generated in the joint during driving.

(b) The location of the pile joint shall be carefully determined to avoid the portion with excessive stress.

(c) For the method for joining piles, refer to **Part III, Chapter 2, 3.4.12 Particulars.**

② Change of the Plate Thickness or Material of Steel Pipe Pile

(a) Any change in the plate thickness or material along the same steel pipe pile must be made in accordance with **Part III, Chapter 2, 3.4.12 Particulars.**

(b) The strengths of the joints and portion with change in steel thickness should be examined carefully because there are some examples[17)] in which the piles of open-type wharves buckled at these portions because of ground deformation in a deep ground at which no bending stresses were generated under normal load conditions.

5.2.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situations

① The performance verification of the open-type wharves on vertical piles in case of Level 2 earthquake ground motion in accidental situations should be appropriately conducted by considering the situation and importance of facilities in question, and the accuracy of analysis method, and so on.

② The examples of performance verification methods include (a) a method in which the cross-sections for verification shall be set with the dynamic analysis of the lumped mass system; further, one-body analysis of the piled piers and the ground shall be conducted with the nonlinear seismic response analysis by considering the three-dimensional dynamic interaction of the piles and the ground and (b) a method in which the cross-sections for verification shall be set in the same way as above; further, using the ground deformation around the piles that have been separately calculated, the response displacement method will be performed using the frame structure of the piled piers.

③ The deformation of the ground at the time of earthquakes, which leads to the damage of the piled piers, is largely caused by the presence of the soil-retaining sections and soil improvement and the specifications of these sections. For example, for piled piers without soil-retaining sections, if it is judged that the ground is good, that the deformation of the ground would not damage the piled piers, that the ground is not in a good condition, that the deformation is controlled by soil improvement, etc., the verification of the piled piers will be conducted using only the dynamic analysis of the lumped mass system. It is necessary to perform verification using appropriate methods in accordance with the situations.

④ For setting the cross-section for verification, nonlinear dynamic analysis of a spring-mass model with single mass or double masses may be used if a container crane has been installed. The system comprises a spring equivalent to the modeled load–displacement relation of the piled pier structure obtained from the elasto–plastic analysis.

⑤ If container cranes or other cargo-handling equipment are installed on a piled pier, the seismic response characteristics of the piled pier may be considerably altered depending on the ratio of the mass of the cargo-handling equipment to that of the piled pier and the ratio of their natural periods. Therefore, it is necessary to perform a seismic response analysis that considers the coupled oscillations of the cargo-handling equipment and the piled pier. For details, refer to **Part III, Chapter 7, 2.2.3 Performance Verification of Earthquake-Resistance.**

⑥ Apart from the inertia forces acting on the superstructure of the piled pier, the factors that exhibit an adverse effect on the piles include the transmission of the deformation of the ground around the earth-retaining section to the superstructure through the access bridge and the transmission of the forces to the piles when the soil around the piles moves toward the sea owing to the deformation of the soils at this location. Therefore, the structure of the access bridge should be such that deformation of the soils around the earth-retaining section does not adversely affect the superstructure of the piled pier.

⑦ When performance verification using nonlinear seismic response analysis etc. is conducted, it is preferable that the effect of inertial force and added mass for pipe water mass in piles of piled piers should be adequately considered.

⑧ As for the bending strength of the steel pipe piles, as the ratio of diameter D to plate thickness t (D/t) increases, the bending strength becomes lower than the fully plastic moment calculated with cross-section calculation. This tendency is observed to become stronger as the axial force ratio increases. Therefore, when the models of steel pipe piles used for nonlinear seismic response analysis are set, it is necessary to consider the D/t ratio of pipes.

For modeling the steel pipe members in the seismic response analysis, the relation between the bending moment and the curvature, which was obtained by three-dimensional FEM analysis with shell elements, can be replaced with the relation of bilinear type using beam element as depicted in **Fig. 5.2.13**. The ultimate curvature ϕ_u is the curvature at the time when bending moment reached to the maximum bending strength in the three-dimensional FEM analysis. In the bilinear type (beam element analysis), the colored area in the figure under the curve up to the point at which the maximum bending strength M_{max} occurs in the three-dimensional FEM analysis was calculated, and the curvature at the point at which the area under the bilinear lines is the same is considered to be the ultimate curvature ϕ_u.

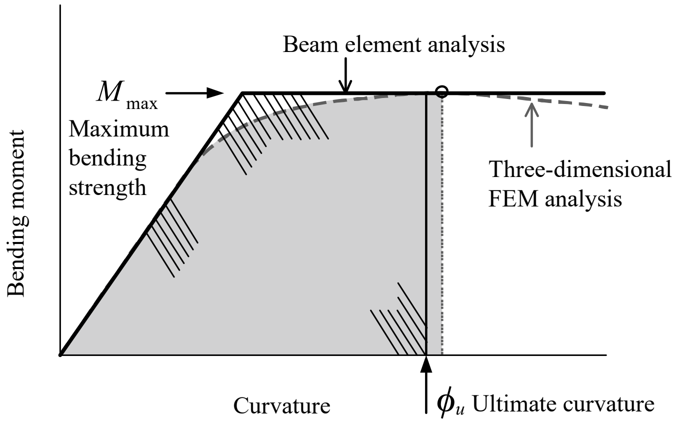


Fig. 5.2.13 Calculation Method of the Ultimate Curvature of the Beam Element (Bilinear Type)

In the beam element based on the infinitesimal deformation theory, if the relation between the bending moment and the curvature of piles are defined in the bilinear type, the ultimate curvature ϕ_u can be calculated from **equations** (**5.2.9**) and (**5.2.10**) by considering the diameter-to-thickness ratio[18].

If the axial force is compressive ($N \geq 0$),

$$\left.\begin{aligned} \phi_u &= \mu \phi_y{}' \\ \phi_y{}' &= \frac{\sigma_y{}' Z}{EI}\left(1 - \frac{N}{N_{yc}{}'}\right) \end{aligned}\right\} \tag{5.2.9}$$

If the axial force is tensile ($N < 0$),

$$\left.\begin{aligned} \phi_u &= \mu \phi_y \\ \phi_y &= \frac{\sigma_y Z}{EI}\left(1 + \frac{N}{N_{yt}}\right) \end{aligned}\right\} \tag{5.2.10}$$

where

ϕ_u : the ultimate curvature (1/mm),

ϕ_y : the curvature corresponding to the yield moment (1/mm),

ϕ_y' : the curvature corresponding to the yield moment in consideration of the reduction in yield stress in the direction of the axial compression (1/mm),

EI : the bending rigidity (N·mm^2),

N_{yc}' : the yield axial force in consideration of the reduction in yield stress in the direction of the axial compression (positive value, N),

N_{yt} : the yield axial force when a steel pipe is subjected to the tensile axial force (negative value, N),

Z : the section modulus (mm^3),

σ_y : the yield stress (N/mm^2),

σ_y' : the yield stress in the direction of the axial compression (N/mm^2), and

μ : ductility factor.

$$\left.\begin{array}{ll}\mu = \gamma\left[(-1.24l/r+209)t/D-0.0119l/r+1.46\right] & \text{(with a retained circular shape)} \\ \mu = \gamma\left[(-4.72l/r+440)t/D+0.0413l/r-2.55\right] & \text{(with an unretained circular shape)}\end{array}\right\}$$

t : the wall thickness (mm)

D : the diameter (mm)

l : the effective member length (mm) (refer to **Reference (Part III), Chapter 1, 2.5.2 Modeling Method of Piles**)

r : the radius of gyration of the cross-section (mm); and

γ : the correction coefficient with respect to yield stress

$$\gamma = \sqrt{235/\sigma_y}$$

As for the correlation coefficient with respect to yield stress, the applicability of up to 450 N/mm^2 has been confirmed[19]. The symbol " ′ " (prime) indicates that the yield stress in the axial compression direction is reduced corresponding to the diameter-to-thickness ratio in accordance with **equation (5.2.11)** [20]. Because the purpose of this equation is reduction corresponding to the diameter-thickness ratio, the upper limit of the reduction coefficient shall be 1, and σ_y' should not become larger than σ_y.

$$\sigma_y' = \sigma_y\left(0.86+5.4t/D\right) \tag{5.2.11}$$

⑨ The standard limit value of deformation in an accidental situation related to Level 2 earthquake ground motion shall be adequately set, and refer to **Part III, Chapter 5, 1.5 Matters concerning High Earthquake-resistance Facilities.**

⑩ With respect to the performance verification of the ultimate curvature of the piles of the open-type wharves on vertical piles for Level 2 earthquake ground motion, if the concerned piled pier is high-earthquake-resistant facilities (designated (emergency supply transport) and designated (trunk line cargo transport)); if no pile, which reaches the ultimate curvature at two points in a pile, exists in the cross-section of the concerned piled pier, it is generally considered that the performance requirements of the piles of the concerned piled pier are satisfied.

With respect to high earthquake-resistance facilities (standard (emergency supply transport)), at least one pile, which reaches the ultimate curvature at less than two points per one pile, exists in the cross-section of the concerned piled pier; therefore, it is generally considered that the performance requirements of the piles of the concerned piled pier are satisfied. **Fig. 5.2.14** schematically depicts the above.

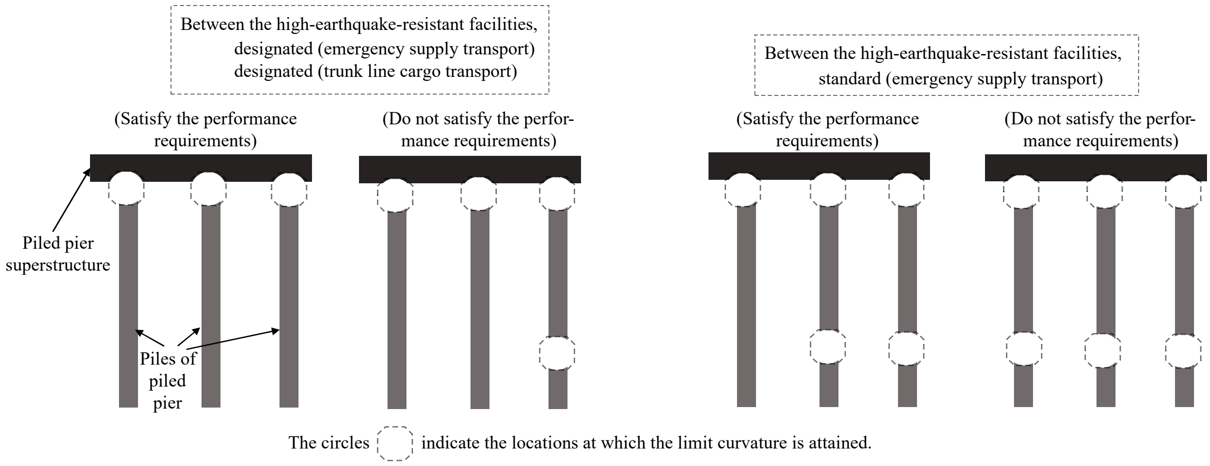


Fig. 5.2.14 Conceptual Diagram of the Counting Piles Reaching the Ultimate Curvature

5.2.6 Performance Verification of the Structural Members

(1) Performance verification can be conducted in accordance with **Part III, Chapter 2, 2 Members of Structure**.

(2) It is necessary to confirm that there will be no loss of the required function because of the deterioration of the concrete superstructure and the steel pipe pile substructure owing to material degradation during the design service life. In particular, there have been several cases where the performance requirements of the concrete superstructures have not been achieved owing to chloride-induced corrosion; therefore, a detailed maintenance management plan should be prepared and followed.

(3) It is necessary to verify that the flexural moment, axial force, and shear force acting on the connections between the steel pipe piles and the superstructure do not reach the ultimate limit state. If the buckling of reinforcing steel or the flaking of covering concrete occurs at the pile heads, the rigid-connecting condition assumed in the calculation of the response values shall not be satisfied, and the actual response value would differ from the response value expected from the calculation. Therefore, attention is required.

(4) In the performance verification of the piled piers, the analysis is performed by assuming the formation of rigid connections between the pile heads and the concrete beams. Further, it is necessary that the pile head flexural moment can be smoothly distributed to the pile head and the concrete beam. The flexural moment that can be distributed to the beam M_{ud} may be calculated using **equation** (**5.2.12**), ignoring the reinforcement connection plates or the vertical ribs that are provided, as necessary. In the following equation, the suffix d indicates the design value.

$$M_{ud} = \frac{DL^2 f'_{cd}}{6\gamma_b} \quad \textbf{(5.2.12),}$$

where

M_u denotes the flexural moment that can be distributed to the part of the pile embedded in the beam (N.mm),

D : diameter of the steel pipe pile (mm),

L : embedded length of the steel pipe pile (mm),

f'_{cd} : design value of the compressive strength of beam concrete (N/mm^2), and

γ_b : member factor (may be considered to be 1.15).

(5) It is assumed that the axial forces are distributed by only the bond among the outer peripheral surface of the piles and the vertical ribs, which are provided, as necessary, as well as the concrete. In this case, the axial force that can be distributed, P_{ud}, can be calculated from **equation** (**5.2.13**). In the following equation, the suffix d indicates the design value.

$$P_{ud} = \frac{1}{\gamma_b}\left(L\varphi + 2A_p\right)f_{bo_d} \quad \textbf{(5.2.13)}$$

where

P_u : axial force that can be distributed to the part of the pile embedded (N),

L : embedded length of the steel pipe pile (mm),

φ : outer perimeter of the steel pipe pile (mm), and

f_{bod} : design value of the bond strength between the pile and the concrete (N/mm^2).

$$f_{bod} = 0.11\, f'_{ck}{}^{2/3} / \gamma_c,$$

where

f'_{ck} : characteristic value of the compressive strength of the concrete (N/mm^2),

γ_c : material coefficient of concrete (= 1.3),

A_p : area of vertical ribs that bonds with concrete (mm^2), and

γ_b : member factor (may be considered to be 1.0).

(6) It is necessary to verify that failure due to punching shear forces in the horizontal direction shall not occur in the beam at the end of which the steel pipe pile is embedded.

(7) With regard to the reinforcing steels of bars, it is necessary that the fixation shall be ensured using measures, such as welding to the steel plates installed at the pile heads of the steel pipe piles, and that the transmission of force between the beams and steel pipe piles should be smooth.

5.3 Open-type Wharves on Coupled Raking Piles

5.3.1 General

(1) The following may be applied to the open-type wharves with a structure in which the horizontal forces acting on the piled pier are distributed to coupled raking piles.

(2) The open-type wharf on coupled raking piles is a structure that resists the horizontal force acting on the wharf, such as the seismic actions, fender reaction force, and tractive force of ships with coupled raking piles. Therefore, this type of wharf must be constructed on ground that yields sufficient bearing capacity for coupled raking piles.

(3) Because the coupled raking piles are so laid out to resist the horizontal forces in the direction normal to the face line of the wharf, the horizontal displacement in that direction is smaller than that of open-type wharves on vertical piles. Coupled raking piles are seldom laid out to resist the horizontal forces in the direction of wharf face line. Therefore, it is preferable to examine the strength of the wharf against the horizontal force parallel to the face line in the same manner as the examination for open-type wharves on vertical piles.

(4) For the procedure for performance verification of open-type wharves on coupled raking piles, refer to **Fig. 5.2.2** of **Part III, Chapter 5, 5.2.1 General** for open-type wharves on vertical piles.

(5) Verification for the variable situations in respect of Level 1 earthquake ground motion may be carried out by obtaining the natural periods of the piled pier with frame analysis and calculating the seismic coefficient for verification with the acceleration response spectrum corresponding to the natural periods. However, as for high earthquake-resistance facilities, appropriate dynamic analysis methods, such as nonlinear seismic response analysis taking account of the 3-dimensional dynamic interaction between the piles and the ground, may be used for verification. For Open-type wharves on coupled raking piles that are not high earthquake-resistance facilities, verification in accidental situations in respect of Level 2 earthquake ground motion can be omitted.

(6) For consideration of the maintenance of open-type wharves on coupled raking piles, refer to **Part III, Chapter 5, 5.2.1 General** for wharves on vertical piles. In addition, as the head parts of coupled piles of open-type wharves on coupled raking piles are narrow, inspection and diagnosis, and countermeasures during the working life are difficult to carry out without special care.

(7) An example of the cross-section of an open-type wharf on coupled raking piles is shown in **Fig. 5.3.1**.

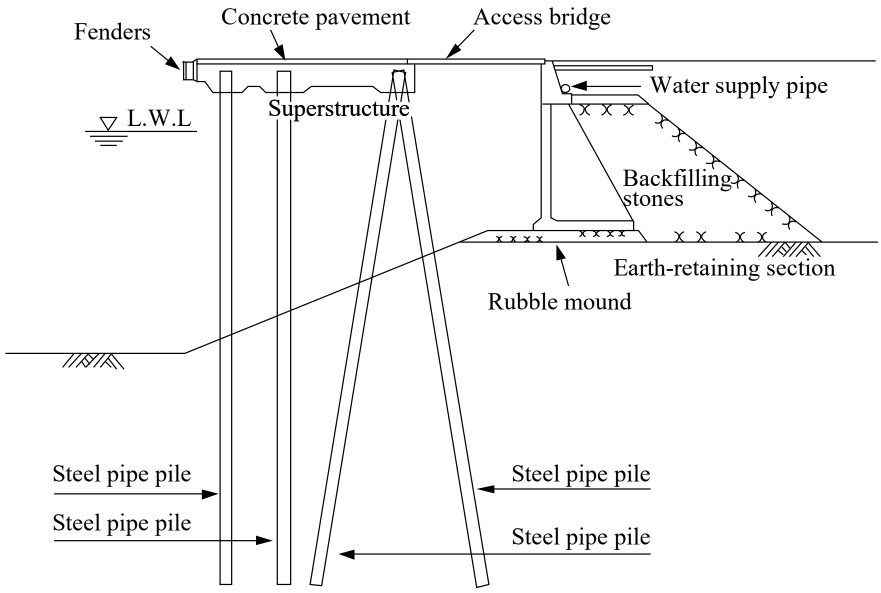


Fig. 5.3.1 Example of Cross-section of an Open-Type Wharf on Coupled Raking Piles

5.3.2 Setting of Basic Cross-section

(1) In the case of open-type wharves on coupled raking piles, the piles come close to adjacent vertical piles and the earth-retaining section; therefore, it is preferable that the layout of the piles be carefully determined considering the construction conditions and the conditions of use.

(2) A large wharf for a design ship size of 10,000 DTW class has one or two sets of coupled raking piles behind one vertical pile in the direction normal to the wharf face line. The distance between piles or between centers of coupled raking piles is usually set to be 4 to 6 m in consideration of loading conditions and construction work.

(3) It is preferable to use a small raking angle of coupled piles from the viewpoint of securing resistance against horizontal force, but in many cases an inclination of 1 : 0.33 to 1 : 0.2 (around 11 to 18 degree from the vertical surface) is used because of constraints related to the required distances from other piles and construction work-related constraints, such as the capacity of the pile driving equipment available.

(4) The beam widths of superstructures of open-type wharves on coupled raking piles depend on how the heads of coupled piles are connected. In general, they tend to be wider than those of open-type wharves on vertical piers.

(5) For setting the basic cross-section of open-type wharves on coupled raking piles, refer to **Part III, Chapter 5, 5.2.2 Setting of Basic Cross-section** in this Chapter.

5.3.3 Actions

(1) Regarding actions to be considered in performance verification of open-type wharves on coupled raking piles, **Part III, Chapter 5, 5.2.3 Actions** can be referred to.

(2) The characteristic value of the seismic coefficient for verification used in the performance verification of open-type wharves on coupled raking piles for the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics of the wharf. For calculation of the seismic coefficient for verification of open-type wharves on coupled raking piles, refer to **Part III, Chapter 5, 5.2.3 (2) ⑭ Ground Motion and Seismic Coefficient for Verification used in Performance Verification of Seismic-resistant**.

(3) Horizontal Forces Distributed to the Pile Head of each Group when Rotation of the Piled Pier Block is Considered

① When it is necessary to consider the rotation of the piled pier block, the horizontal forces distributed to the pile head of each group of piles in an open-type wharf on coupled raking piles may be appropriately calculated in accordance with the cross-section of each pile and the raking angle and length of the raking piles. In this case, it may be assumed that all horizontal forces are distributed to the coupled raking piles. Normally the row of piles having the maximum distributed horizontal force among all the rows of piles is adopted as the row of piles used in the verification.

② In the case where the cross-section of each pile group and raking angle of the raking piles are different, the horizontal force distributed to the pile head of each group may be calculated using **equation (5.3.1)** (see **Fig. 5.3.2**).

(a) When the piles can be regarded fully as end bearing piles

$$H_i = \frac{C_i}{\sum_i C_i} H + \frac{C_i x_i}{\sum_i C_i x_i^2} eH \tag{5.3.1}$$

where,

$$C_i = \frac{\sin^2(\theta_{i1} + \theta_{i2})}{\frac{l_{i1}}{A_{i1}E_{i1}}\cos^2\theta_{i2} + \frac{l_{i2}}{A_{i2}E_{i2}}\cos^2\theta_{i1}} \quad \text{(N/m)}$$

H : horizontal force acting on the block (N/m)

H_i : horizontal force distributed to each pile (N/m)

e : distance between center line of pile group and the acting horizontal force (m)

x_i : distance from each pile group to the center line of a pile group (m)

□$_i$: total pile length (m), being substituted the pile length of the friction pile □ when pulling-out forces are acting.

A_i : cross-sectional area of each pile (m^2)

E_i : Young's modulus of each pile (N/m^2)

θ_{i1}, θ_{i2} : angle of each pile with the vertical direction (°)

The subscript i refers to the ith pile.

The subscripts 1, 2 refer to each pile in one pile group.

The center line of a pile group may be obtained from $\Sigma C_i \xi_i / \Sigma C_i$. ξ_i are the coordinates from an arbitrary coordinate origin of each pile group in face line direction.

(b) When the piles can be regarded fully as friction piles

1) Sandy soil

Equation (5.3.1) is used, substituting, $\frac{2l_i + \lambda_i}{3}$ for □$_i$.

2) Cohesive soil

Equation (5.3.1) is used, substituting, $\frac{l_i + \lambda_i}{2}$ for □$_i$.

where, λ_i: Pile length of the part over which the peripheral surface resistance force is not effectively working (m), l_i: Total pile length (m).

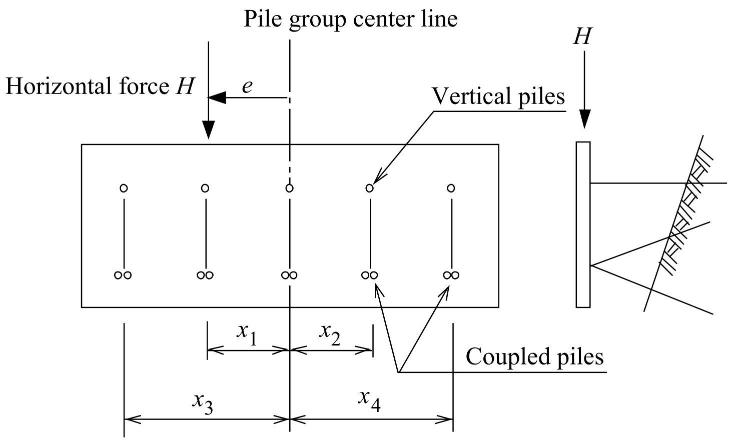


Fig. 5.3.2 Pile Group Center Line and Distance from each Pile Group

③ When the cross-section, raking angle and length of the raking piles of each pile group are all equal, the horizontal force distributed to each pile group may be calculated from **equation (5.3.2)**.

$$H_i = \frac{1}{n} H + \frac{x_i}{\sum_i x_i^{\,2}} eH \tag{5.3.2}$$

where, n: number of coupled piles

5.3.4 Performance Verification of Open-type Wharves on Coupled Raking Piles

(1) Items for the Performance Verification of Open-type Wharves on Coupled Raking Piles

The performance verification of open-type wharves on coupled raking piles shall apply **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers** and be based on the following.

(2) Performance Verification of Earth-retaining Sections

① For the performance verification of earth-retaining sections, refer to Part III, 5Chapter 5, .2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers.

② It is necessary to ensure that the action due to deformation of the earth-retaining section by earthquakes shall not be transmitted to the superstructure of the piled pier via the access bridge, and that the piles are not adversely affected by significant deformation of the soil around the piles toward the sea. Because earth-retaining sections and piles are close to each other in case of raking piles, special attention is required in deciding the location of earth-retaining sections.

(3) Verification of Stresses in Piles

① The cross-sectional stress in each pile may be calculated by applying **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers** in this Chapter for piles subject to axial forces or piles subject to axial forces and flexural moments.

② However, if partial factors are applied in performance verification regarding axial compressive stress of piles in the variable situation (ship berthing) of open-type wharves on coupled raking piles, **Table 5.2.1** in **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers** cannot be applied. In this case, the following **Table 5.3.1** can be used. Values shown as “-” in **Table 5.3.1** means that the values may be verified using values enclosed in parentheses () as a matter of convenience. These values are factors set referring to allowable stress and the like in the previous design methods.

Table 5.3.1 Partial Factors in Variable Situation by Ship Berthing of Open-type Wharves on Coupled Raking Piles

Verification target	Partial factor to be multiplied by resistance term γ_R	Partial factor to be multiplied by load term γ_S	Adjustment factor m
Compressive stress occurring in the piles of a piled pier (variable action due to ship berthing force)	– (1.00)	– (1.00)	1. 67

③ If coupled piles are laid out in the direction normal to the face line of the wharf, it is preferable that the stress of each pile in that direction is 20 to 30% lower than the yield stress based on **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-Type Wharves on Vertical Piles**, in order to deal with the flexural moment or secondary stress not considered in the verification. On the other hand, as for the direction of the wharf face line, stress can be calculated in accordance with **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-Type Wharves on Vertical Piles**.

④ As open-type wharves on coupled raking piles are mostly constructed on ground where sufficient bearing capacity can be expected, it is preferable that special attention is paid to examine impact stress by driving, buckling, etc. For examination, refer to **Part III, Chapter 2, 3.4 Pile Foundations**.

(4) Performance Verification of Bearing Forces on Piles

① The pushing-in and pulling-out forces of each pair of coupled raking piles shall be calculated appropriately based on the vertical and horizontal forces defined in consideration of the wharf operation conditions.

② The pushing-in and pulling-out forces on each raking pile are axial forces of each raking pile obtained with a frame analysis method, taking into consideration the effect of the raking angle of the pile as indicated in **Part III, Chapter 2, 3.4.8 Calculation of Deflection of Piles by PHRI Method**, calculating the ratio of the coefficient of lateral subgrade reaction, and appropriately correcting the coefficient of lateral subgrade reaction.

③ For verification of pushing-in and pulling-out forces in each raking pile, refer to **Part III, Chapter 2 3.4.3 Pushing-in Resistance Force in Axial Direction of Piles** and **Part III, Chapter 5, 3.4.4 Pulling-out Resistance Force in Axial Direction of Piles**.

④ For partial factors used for verification of bearing forces on piles, refer to **Table 5.2.3** in **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers**.

(5) Verification of Pile Embedment

For the embedment length of raking piles, refer to Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers.

(6) Analysis in the Face Line Direction

① If there are coupled raking piles in the face line direction, the analysis should be carried out in the same way as the direction perpendicular to the face line.

② In the sections of open-type wharves on coupled raking piles other than special locations like the pointed end, it is not rare that coupled piles are laid out with respect to actions in the face line direction of wharfs due to restrictions such as processes of driving raking piles and construction equipment. If coupled piles are not laid out in the face line direction, piles resist with lateral resistance to actions in the face line direction. Therefore, stresses, etc. of piles shall be examined in the same way as open-type wharves on vertical piers. In that case, the virtual ground surface, the virtual fixed point, and the like may be considered in the same way as open-type wharves on vertical piers.

③ Horizontal forces in the face line direction include actions of earthquake ground motion, tractive force by ships, and fender reaction force. Although the superstructures of piled piers were equipped with joints for each block interval, but horizontal displacements are actually transmitting each other. Thus, actions like the tractive force and the berthing force, which do not occur simultaneously at individual locations of the whole mooring facility, occur not only in one block but are distributed to a certain section of a piled pier and therefore the stress of a pile would not become so dangerous as the stress of coupled piles in the normal direction to the face line. However, the earthquake ground movement actions work simultaneously to the whole piled piers and therefore require consideration.

5.3.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situation

Performance verification of open-type wharves on coupled raking piles to Level 2 earthquake ground motion in accidental situations needs to be carried out appropriately, taking into account the situation in which the facilities in question, the importance and accuracy of the analysis method, and so on. Performance verification of open-type wharves on coupled raking piles in accidental situations can be conducted in accordance with **Part III, Chapter 5, 5.2.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situation**.

5.3.6 Performance Verification of Structural Members

For performance verification of structural members such as superstructures and access bridges, refer to **Part III, Chapter 5, 5.2.6 Performance Verification of Structural Members** in this Chapter.

5.4 Strutted Frame Type Pier

5.4.1 General

(1) For maintenance of strutted frame type piers, refer to **Part II, Chapter5, 5.2.1 General**, and **Part II, Chapter5, 5.3.1 General** in this Chapter for open-type wharves on vertical and coupled raking pile, respectively. When designing stiffening members and panel points of strutted frame type piers, appropriate corrosion control measures must be adopted in the same manner as that for steel pipe piles based on **Part II, Chapter 2, 1.3.4 Examination concerning the Change of Performance over Time**.

5.4.2 Actions

(1) Regarding actions to be considered in the performance verification of strutted frame type piers, refer to **Part II, Chapter5, 5.2.3 Actions**.

(2) The characteristic value of the seismic coefficient for verification used in the performance verification of strutted frame type piers against variable situations with respect to Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics. For calculating the seismic coefficient for verification of strutted type piers, refer to **Part II, Chapter5, 5.2.3(2) ⑭ Ground Motion used in Performance Verification of Seismic-resistant and Seismic Coefficient for Verification**.

5.4.3 Performance Verification

(1) For performance verification of strutted frame type piers, refer to **Part II, Chapter5, 5.2 Open-type Wharves on Vertical Piles** and **Part II, Chapter5, 5.3 Open-type Wharves on Coupled Raking Piles,** and also refer to **the Strutted Frame Method Technical Manual**[21)].

(2) Verification of Level 2 Earthquake Ground Motion via the Dynamic Analysis Method

① The performance verification of strutted frame type piers in accidental situations with respect to Level 2 earthquake ground motion shall be appropriately conducted considering the concerned circumstances around the facilities, importance of the facility, and the accuracy of the method. Performance verification of strutted frame type piers may basically comply with that of jacket type piled piers.

② The required performance of strutted frame type piers in accidental situations with respect to Level 2 earthquake ground motion is basically the same as that of open-type wharves on vertical piles. In addition, conducting additional examination according to the structure of strutted frame type piers such as stiffening members and panel points is necessary.

5.5 Jacket Type Piled Piers

[Public Notice] (Performance Criteria for Piled Piers)

Article 55

1 The provisions of Article 48 apply mutatis mutandis to the performance criteria of piled piers.

2 In addition to the provisions of the preceding paragraph, the performance criteria of the access bridge of piled piers shall be as prescribed respectively in the following items:

(1) The access bridge of piled piers shall satisfy the following criteria:

(a) The access bridge of piled piers shall have the dimensions necessary for enabling the safe and smooth loading, unloading, embarkation and disembarkation, etc. in consideration of the usage conditions.

(b) The access bridge of piled piers shall not transmit horizontal loads to the superstructure of the piled pier, and shall not fall down even when the piled pier and the earth-retaining part are displaced owing to the actions of earthquakes, etc.

(2) The following criteria shall be satisfied in variable situations in which the dominant actions are Level 1 earthquake ground motions, ship berthing and traction by ships, and imposed load:

(a) The risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(b) The risk that the axial force acting on the piles may exceed the resistance capacity owing to failure of the ground shall be equal to or less than the threshold level.

(c) The risk that the stress in the piles may exceed the yield stress shall be equal to or less than the threshold level.

(3) The following criteria shall be satisfied under the variable situation in which the dominating action is variable waves:

(a) The risk of impairing the stability of the access bridge due to uplift acting on the access bridge shall be equal to or less than the threshold level.

(b) The risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(c) The risk that the axial force acting on piles may exceed the resistance capacity owing to failure of the ground shall be equal to or less than the threshold level.

(4) For the structures with stiffening members, the risk of impairing the integrity of the stiffening members and connection points of the structures under the variable situation, in which the dominating actions are variable waves, Level 1 earthquake ground motions, ship berthing and traction by ships, and surcharge load, shall be equal to or less than the threshold level.

3 The provisions of Article 49 through Article 52 apply mutatis mutandis to the performance criteria of the earth-retaining parts of piled piers in consideration of the structural type.

5.5.1 General

(1) An example of cross-section of a jacket type piled pier is shown in **Fig 5.5.1**.

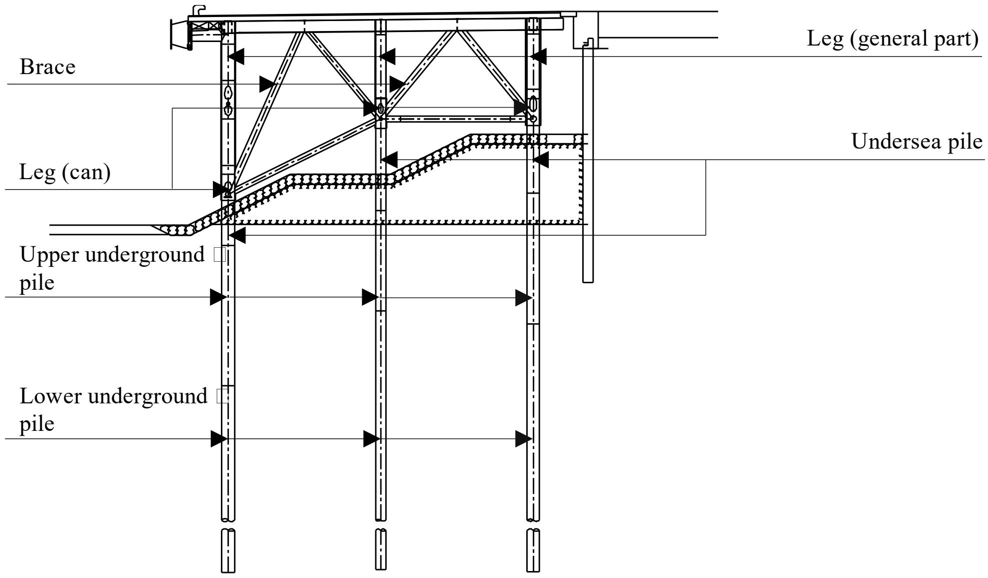


Fig. 5.5.1 Example of Cross-section of a Jacket Type Piled Pier

(2) In some cases, earth-retaining sections are integrated into jacket type piled piers. In one type of such earth-retaining walls, walings are installed and sheet piles are arranged on a straight line. In the other type, no waling is installed and sheet piles are arranged in an arc form.

(3) For consideration for maintenance of jacket type piled piers, refer to **Part III, Chapter5, 5.2.1 General**, **5.3.1 General**, and **Part III, Chapter 5, 5.4 Strutted Frame Type Pier** for open-type wharves on vertical piles, open-type wharves on coupled raking piles, and strutted frame type piers, respectively.

5.5.2 Actions

(1) For actions to be considered in performance verification of jacket type piled piers, refer to **Part III, Chapter5, 5.2.3 Actions**.

(2) The characteristic value of the seismic coefficient for verification used in the performance verification of jacket type piled piers in the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics. For calculation of the seismic coefficient for the verification of jacket type piled piers, refer to **Part III, Chapter5, 5.2.3(2) Ground Motions used in Performance Verification of Seismic-resistant and Seismic Coefficient for Verification**.

5.5.3 Performance Verification

(1) For performance verification of jacket type piled piers or piled piers whose structure has stiffening members, refer to **Part III, Chapter5, 5.2 Open-type Wharves on Vertical Piles**, and **Part III, Chapter5, 5.3 Open-type Wharves on Coupled Raking Piles**; for details, refer to **the Jacket Method Technical Manual**[22].

(2) Verification of Level 2 Earthquake Ground Motion with the Dynamic Analysis Method

① The performance verification of jacket type piled piers in accidental situations in respect of Level 2 earthquake ground motion shall be appropriately carried out considering the concerned circumstances around the facilities, importance of the facility, and preciseness of the method. The performance verification of jacket type piled piers may comply with that of open-type wharves on vertical piles, but the actions occurring in the members shall be appropriately set considering the structure of the trusses. The different points in the dynamic characteristics between jacket type piled piers and open-type wharves on vertical piles are as follows:

(a) The natural periods are short because jacket type piled piers have truss structure.

(b) Because the structure has panel points, the failure mechanisms are complex.

(c) Separate verification of the panel points is necessary.

② Examples of performance verification methods include a method in which cross-sections for verification shall be set with the dynamic analysis of lumped mass system, and then, using the ground deformation around the piles calculated separately, the response displacement method using the frame structure of piled piers can be applied.

③ As jacket type piled piers generally possess resistance even after buckling of stiffening members, elements that can express characteristics after buckling need to be used in modeling non-linear history of brace material of jackets. The buckling curve may be obtained with methods such as the finite element method and the method assuming plastic hinge. Simple models of approximation with several curves and straight lines may be used[23)].

④ The performance required of jacket type piled piers in accidental situations in respect of Level 2 earthquake ground motion is basically the same as that of open-type wharves on vertical piles. In addition, it is necessary to carry out additional examination according to the structure of jacket type piled piers such as joints between jackets and piles and panel points.

5.6 Dolphins

5.6.1 General

(1) The following may be applied to the performance verification of mooring facilities such as pile type, steel cell type, caisson type, and other types of dolphin structures. Depending on their function, the types of dolphin structures include breasting dolphins, which are used for ships' berthing; mooring dolphins, which are used to hook mooring ropes; and loading dolphins.

(2) It should be noted that the guidelines outlined in **Part III, Chapter 5, 5.6.3 Actions** and **Part III, Chapter 5, 5.6.4 Performance Verification** may be used in simplified verification methods. Therefore, it is preferable to adopt highly precise methods (model experiment or numerical analysis that can reproduce mechanisms).

(3) The performance verification of dolphins should be performed by considering the following items. For other items, it is preferable to appropriately perform performance verification in accordance with each structural form.

① The direction of actions on dolphins is not necessarily a constant direction; hence, the verification should be performed for several directions as necessary.

② Conventional torsion in the case of pile-type structures and rotation in the case of caisson-type structures have not been examined comprehensively. However, these factors may affect the stability of structures in certain cases; therefore, it is necessary to carefully consider these aspects.

③ The superstructure of the dolphin should have a height that shall not be affected by waves, and the crown height of the dolphin shall be appropriately set in accordance with its function. In this connection, the position of installation of the fenders for breasting dolphins, the level of the deck of the ship for mooring dolphins, and the working range of the loading arm for loading dolphins should be taken into consideration. For connecting bridges, its height should be sufficient in such a way that it is not affected by the action of waves.

(4) Consideration is required for the appropriate maintenance of dolphins in accordance with their structural forms.

① For pile type dolphins, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Piles**.

② For steel cell type dolphins, refer to **Part III, Chapter 5, 2.9 Cellular-Bulkhead Quaywalls with Embedded Sections**.

③ For caisson type dolphins, refer to **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls**.

(5) **Fig. 5.6.1** shows an example of a cross section of a pile-type dolphin.

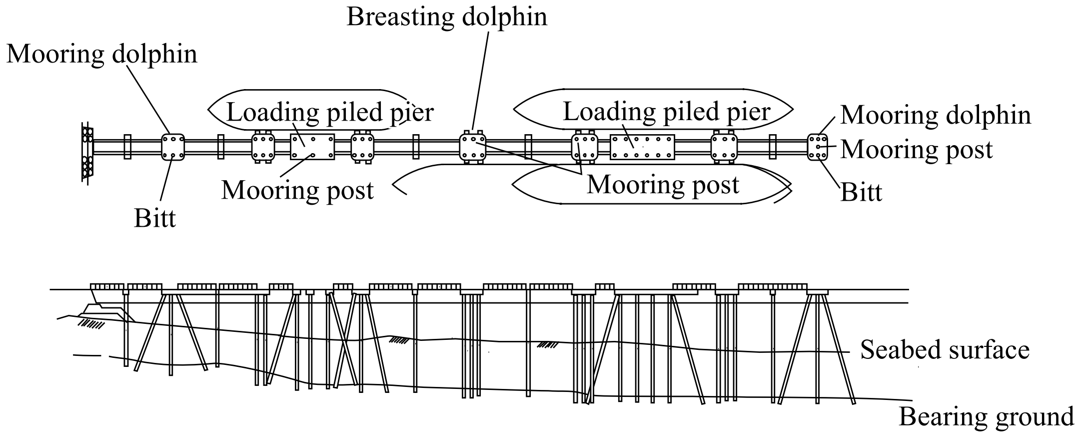


Fig. 5.6.1 Example of a Cross Section of a Pile-Type Dolphin

5.6.2 Layout

(1) The layout of a dolphin berth shall be determined appropriately to avoid adverse effects on the navigation and anchorage of other ships in consideration of the dimensions of the design ships, water depth, wind direction, wave direction, and tidal currents.

(2) In the determination of the layout of breasting dolphins, the following items need to be examined:

① **Dimensions of the design ship**

(a) The side of design ships is usually composed of curve lines that form the outlines of the bow and stern parts, each of which accounts for approximately 1/8 of the overall length (L) of the ship, and a straight line that forms the outline of the central part, which accounts for approximately 3/4 of the overall length (L) of the ship. It is preferable that breasting dolphins are installed in such a way that the ships can be berthed to them with the straight line part. Normally, one breasting dolphin each is installed in the bow and stern. However, for dolphins serving both large and small ships, two breasting dolphins are each provided toward the bow and stern.

(b) When special cargo handling equipment is required for dolphins for oil handling, a cargo handling platform dolphin is installed midway between the breasting dolphins. In this case, it is preferable to locate the cargo handling platform with its seaside front slightly backward from that of the breasting dolphins so that the ship berthing force does not act directly on the cargo handling platform dolphin.

② The layout of dolphins should be designed in such a way that the longitudinal axis of dolphins becomes parallel to the prevailing directions of winds, waves, and tidal currents. This layout helps ease ship maneuvering during berthing and unberthing and reduces the external forces acting on the dolphins when the ship is moored. If the directions of winds, waves, etc. are different from those of coastlines, and when dolphins need to be constructed near the coast, the dolphins are usually laid out parallel to the coastline for the usage of the water area because dolphins that are designed for medium-sized or smaller ships near the coast should not have a large wind load. Furthermore, positions that can secure the planning depth at the water depth of the ground are advantageous.

③ Dolphins should be laid out in such a way that avoids adverse effects on the anchorage of other ships in anchorage areas and on the navigation of other ships in navigation channels.

(3) Although mooring posts may be sometimes installed on land when the coast is near to connect mooring ropes for berthing and mooring, mooring posts are usually installed on mooring dolphins. Mooring dolphins are normally set at a 45° angle from the rope bitts on the bow and stern of a ship and with a certain setback from the front face of the breasting dolphins. The number of mooring dolphins is decided on the basis of the tractive force of ships, but two to four units are usually adopted based on previous construction examples. It is also possible to install mooring posts on breasting dolphins and use them for the mooring of design ships and small ships.

(4) The distance between breasting dolphins is closely related to the overall length (L) of the design ships. **Fig. 5.6.2** shows the relationship between the breasting dolphin interval and the water depth derived from past construction data for reference.

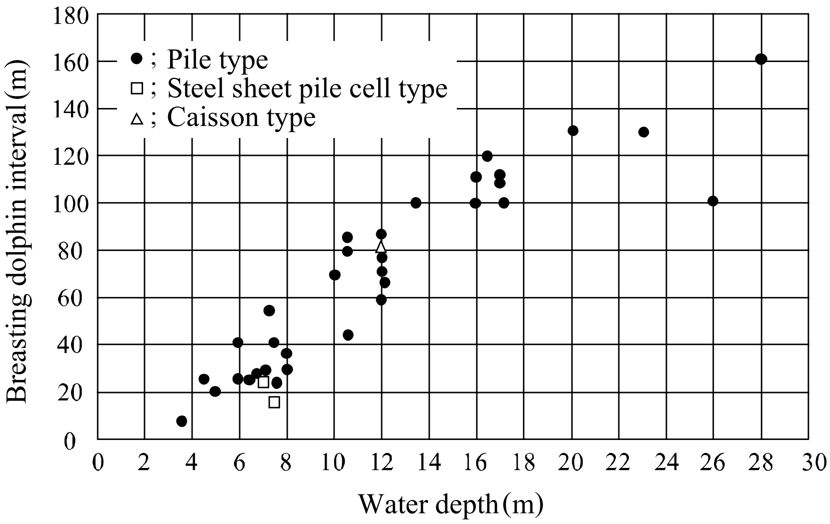


Fig. 5.6.2 Distance between Breasting Dolphins

5.6.3 Actions

(1) For the calculation of the reaction force from the fenders onto the dolphins, refer to **Part II, Chapter 8, 2.2 Action Caused by Ship Berthing** and **Part III, Chapter 5, 9.2 Fender Equipment**.

(2) For the calculation of the tractive force of ships, refer to **Part II, Chapter 8, 2.4 Action due to Traction by Ships**.

(3) For the calculation of vertical loads due to self-weight and live load, refer to **Part II, Chapter 10, Self-Weight and Surcharge** and **Part III, Chapter 5, 5.2.3 Actions** for open-type wharves on vertical piles.

(4) For the action due to earthquakes, refer to **Part II, Chapter 6, Earthquakes** and **Part III, Chapter 5, 5.2.3 Actions** in this chapter for open-type wharves on vertical piles.

(5) For the calculation of dynamic water pressure during an earthquake, refer to **Part II, Chapter 4, 3.2 Dynamic Water Pressure**.

(6) For the calculation of wind pressure forces acting on cargo handling equipment, refer to **Part II, Chapter 2, 2.3 Wind Pressure**.

5.6.4 Performance Verification

(1) Pile Type Dolphins

① For the performance verification of pile type dolphins, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Pile**r.

② The characteristic value of the seismic coefficient for the verification used in the performance verification of pile type dolphins in variable situations with respect to Level 1 earthquake ground motion shall be appropriately calculated by considering the structural characteristics. For the calculation of the seismic coefficient for the verification of pile type dolphins, refer to **Part III, Chapter 5, 5.2.3(2)⑭ Ground Motions used in the Performance Verification of Seismic Resistance and Seismic Coefficient for Verification**.

③ In the case of pile type dolphins, the berthing energy may normally be calculated on the assumption that it is absorbed by the deformations of the fenders and the piles.

④ Large tankers are usually berthed at a slant angle with the dolphin alignment line. Considering that the characteristics of fenders vary depending on the berthing angle, it is recommended to use the characteristics curve appropriate to the berthing angle. Furthermore, a slanting berthing may cause some of the fenders attached to a breasting dolphin to not absorb the berthing energy effectively. Therefore, it is preferable to examine carefully the fenders that will come into contact with the hull of the ship in consideration of the berthing angle.

⑤ For the structure of the joints of the pile heads, **Part III, Chapter 5, 5.2.6 Performance Verification of Structural Members** can be applied with modification as necessary. If steel pipe piles are used, it is preferable to improve the joints with superstructures and to apply filling with concrete around the L.W.L. to avoid pile deformation due to collisions with driftwood or small ships. If material other than concrete is used for the superstructures, verification should be performed appropriately in consideration of the material properties.

⑥ For ancillary facilities such as mooring posts, fenders, skirt guards, and ladders, refer to **Part III, Chapter 5, 9 Ancillary of Mooring Facilities** in this chapter. For connecting piled piers, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Piles**.

(2) Steel Cell Type Dolphins

① For the performance verification of steel cell type dolphins, refer to **Part III, Chapter 5, 2.9 Cellular-Bulkhead Quaywalls with Embedded Sections**.

② The characteristic value of the seismic coefficient for the verification for the performance verification of steel cell type dolphins in variable situations with respect to Level 1 earthquake ground motion shall be appropriately calculated by considering the structural characteristics. The characteristic value of the seismic coefficient for verification of steel cell type dolphins may be calculated in accordance with **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls** when soil pressure is the acting force or in accordance with **Part III, Chapter 4, 3.1 Gravity-Type Breakwaters (Composite Breakwaters)** when soil pressure is not the acting force.

③ For the foundations of cargo handling equipment and mooring posts, refer to **Part III, Chapter 2, 3.4 Pile Foundations** and **9.19 Foundations for Cargo Handling Equipment**.

④ In the case of a cylindrical cell type dolphin, the equivalent wall width can be calculated using **equation (5.6.1)**.

$$B = \sqrt{3}R \tag{5.6.1}$$

where

B : equivalent wall width (m);

R : radius of cylindrical cell (m).

⑤ In general, steel cell type dolphins have reinforced concrete superstructures for the whole top area, and a crown made from steel sheet piles or steel plates is embedded on them. When heavy goods, such as cargo handling equipment, are placed on this structure, a significantly large compressive force acts on the steel sheet piles or steel plates via the superstructures. Buckling may occur on the crown if the weight is not relieved. One countermeasure involves driving piles in the inner filling of the cells for support as pile foundations.

⑥ For the performance verification of structural members, refer to **Part III, Chapter 5, 5.6.4(1) Pile Type Dolphins**. For the verification of members of steel cells, refer to **Part III, Chapter 5, 2.9 Cellular-Bulkhead Quaywalls with Embedded Sections**.

(3) Caisson Type Dolphins

① For the performance verification of caisson type dolphins, refer to **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls** in this chapter.

② The characteristic value of the seismic coefficient for the verification for the performance verification of caisson type dolphins in variable situations with respect to Level 1 earthquake ground motion shall be appropriately calculated by considering the structural characteristics. The characteristic value of the seismic coefficient for the verification of caisson type dolphins may be calculated in accordance with **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls** when soil pressure is the acting force or in accordance with **Part III, Chapter 4, 3.1 Gravity-Type Breakwaters (Composite Breakwaters)** when soil pressure is not the acting force.

③ The rotation of a caisson occurs when an eccentric external force acts on a dolphin. The examination of stability against rotation must be made even when the stability against sliding and overturning, as well as against the failure of the foundation ground due to insufficient bearing capacity, are found to be satisfactory because the confirmation of the stability with respect to these items does not guarantee that the caisson is safe against rotation. In this case, in calculating the resistance force, attention should be given to the friction force of the caisson bottom, which should be proportional to the bottom reaction force, as described in **Part III, Chapter 2, 2.2 Caissons**.

④ For the performance verification of structural members, refer to **Part III, Chapter 5, 5.6.4(1) Pile Type Dolphins**. Furthermore, for the verification of caisson members, refer to **Part III, Chapter 2, 2.2 Caissons**.

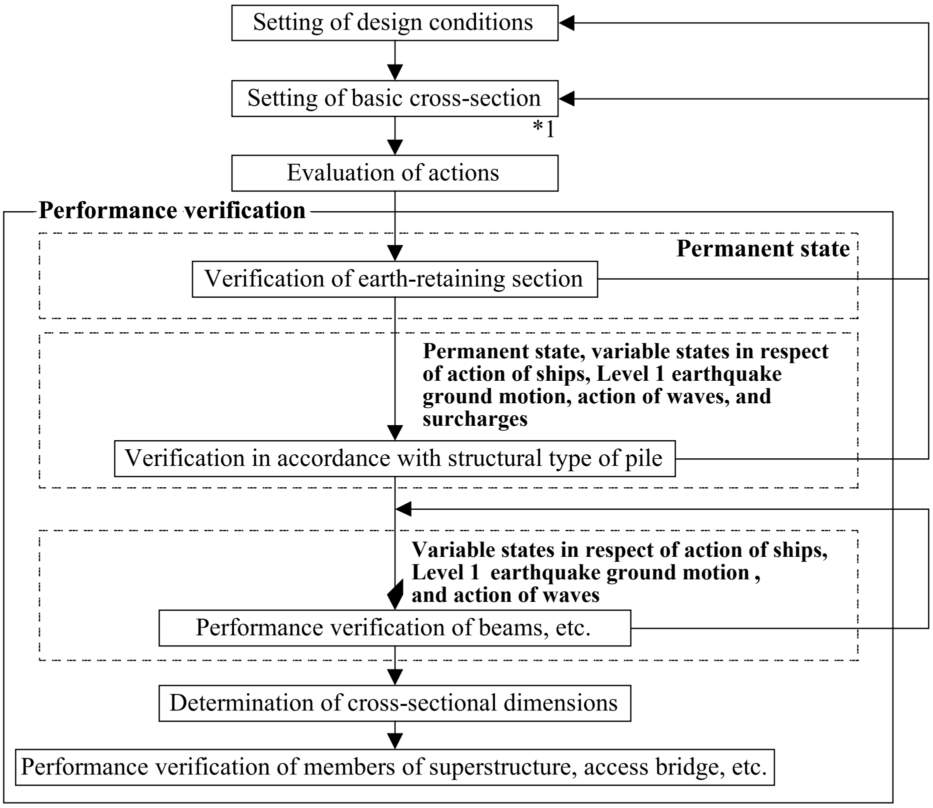
5.7 Detached Piers

5.7.1 General

(1) The following may be applied to the performance verification of detached piers comprising the detached pier and the earth-retaining section.

(2) Detached piers are foundations such as a rail-mounted portal bridge crane, constructed at locations with adequate water depth, and used as mooring facilities. In general, a detached pier needs no floor structure and consists of a beam and piles that support the beam.

(3) An example of the procedure of performance verification of detached piers is shown in **Fig. 5.7.1**.



*1: The evaluation of the effect of liquefaction, settlement, etc. is not indicated; therefore it is necessary to consider this separately.

Fig. 5.7.1 Example of Procedure of Performance Verification of Detached Piers

(4) For consideration for the maintenance of detached piers, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Piles** with paying due consideration to the structural characteristics.

(5) An example of a cross-section of a detached pier is shown in **Fig. 5.7.2**.

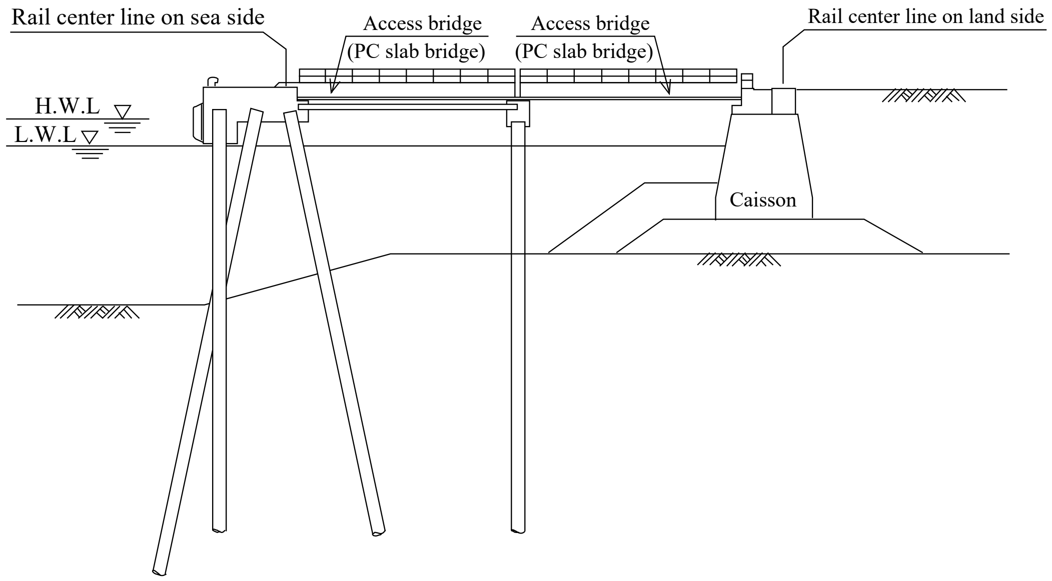


Fig. 5.7.2 Example of Cross-section of a Detached Pier

5.7.2 Setting of Basic Cross-section

(1) The distance between a detached pier and the land as well as the rail interval and the spacing of piles in the face line direction shall be set based on the economic efficiency, constructability, etc. with due consideration given to the dimensions of the mounted cranes, the sea bed, etc.

(2) In general, simple beams are adopted with due consideration given to the uneven settlement of piles.

(3) The performance verification of the detached pier shall be conducted so that it is stable against all the actions on the piles and beams. In addition, it is preferable for the detached pier that a structure is decided with consideration of dimensions of portal bridge crane, the traveling characteristics, and the settlement of rails after installation.

(4) Rail mounted cranes are installed on detached piers; therefore, it is preferable that the structure shall have a small deformation.

(5) In some cases, both rails of cargo handling equipment are laid on a detached pier. In other cases, only one rail is laid on a detached pier while the other rail is laid on the earth-retaining section. In the latter case, it is preferable to construct the foundation of the fixed legs of cargo handling equipment on the land side.

(6) It is necessary to pay adequate attention to the deformation of the earth-retaining section due to the action of earthquakes.

5.7.3 Actions

(1) For the wheel loads of cargo handling equipment, refer to **Part II, Chapter 10, 3.2 Live Load**.

(2) For tractive forces of ship, refer to **Part II, Chapter 8, 2.4 Action due to Traction by Ships**.

(3) For the self-weight of superstructures and self-weight of piles, refer to **Part II, Chapter 10, 2 Self-Weight**, and **Part II, Chapter 10, 3 Surcharge**.

(4) For fender reactions, refer to **Part II, Chapter 8, 2.2 Action Caused by Ship Berthing**, **Part II, Chapter 8, 2.3 Action Caused by Ship Motions**.

(5) For wind loads acting on cargo handling equipment and superstructures, refer to **Part II, Chapter 2, 2.3 Wind Pressure**.

(6) For the ground motions acting on cargo handling equipment, superstructures, and piles, refer to **Part II, Chapter 6, 2 Seismic Action**.

(7) The characteristic value of the seismic coefficient for verification for the performance verification of the detached piers against the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated with due consideration given to the structural characteristics. For the calculation of the seismic

coefficient for verification of detached piers, refer to **Part III, Chapter 5, 5.2.3(2) ⑭ Ground Motions used in the Performance Verification of Seismic Resistance and Seismic Coefficient for Verification**.

(8) For the performance verification of the detached piers, it is preferable to consider wave forces and uplift pressure when necessary.

(9) For the performance verification of the beams, braking forces on cargo handling equipment shall be considered as a horizontal force, but for piles, braking forces on cargo handling equipment shall be considered, as necessary.

(10) For a live load acting on the access bridges and the floor slabs, 5.0 kN/m^2 may be assumed.

5.7.4 Performance Verification

(1) The performance verification of the piles of the detached piers may be carried out by appropriately selecting items from performance criteria of **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles**, **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Piles**, **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls**, and **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections** r, in accordance with the structure type. In addition, the performance verification of the earth-retaining part may be carried out by appropriately selecting items from performance criteria of **Part III, Chapter 5, 2.2 Gravity-type Quaywalls**, **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**, and **Part III, Chapter 5, 2.4 Cantilevered Sheet Pile Quaywalls** and, in addition, refer to the following.

(2) Performance Verification of Piles

Performance Verification of piles shall be carried out appropriately in accordance with the structure type.

(3) Performance Verification of Beams

① The performance verification of beams shall be conducted so that they are safe against vertical as well as horizontal forces and loads.

② Structural elements with sufficient strength against the designated vertical and horizontal forces shall be used for the beams of a detached pier because the crane rails for a crane are directly installed on the beams. In the examination of vertical loads, due consideration shall be given to the increase in the wheel loads due to the wind load or seismic force on cargo handling equipment.

③ When both legs of the bridge crane are fixed ones, the horizontal load acting on each leg is determined by distributing the total horizontal load to each leg based on the proportion of the wheel load. When the bridge crane has a fixed leg and a suspended leg, the whole horizontal load shall be borne by the fixed leg for making the design on the safer side. However, at the same time, the horizontal force being one-half of the force acting on one fixed leg in the case of both legs being fixed shall be borne by the suspended leg.

(4) Performance Verification of Earth-Retaining Sections

① Performance verification of earth-retaining sections shall be carried out appropriately in accordance with the structure type.

② If one rail of the cargo handling equipment is laid at the back of the earth-retaining section, performance verification may be carried out in accordance with **Part III, Chapter 5, 9.19 Foundations for Cargo Handling Equipment**.

5.7.5 Performance Verification of Structural Members

(1) Superstructure

For performance verification of superstructures, refer to **Part III, Chapter 5, 5.2.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situation**.

(2) Access Bridge

For performance verification of the access bridges, refer to the **Specifications and Commentary for Highway Bridges**[23)] and **Technical Standards and Commentary of Elevated Pedestrian Crossing Facilities**[24)].

5.7.6 Structural Detail

(1) Detached piers are generally equipped with ancillary facilities such as fenders, mooring posts, and access bridges.

(2) Access bridges are provided at one to two locations per berth. In addition, if there is not enough space for rope-handling work when ships depart, slabs are required for safety.

(3) For fenders and mooring posts, refer to **Part III, Chapter 5, 9 Ancillary of Mooring Facilities**.

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6 Floating Piers

[Ministerial Ordinance] (Performance Requirements for Floating Piers)

Article 30

1 The performance requirements for floating piers shall be as prescribed respectively in the following items in consideration of the structural type:

(1) The requirements specified by the Ministry of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable the safe and smooth mooring of ships, embarkation and disembarkation of people, and handling of cargo.

(2) Damage, etc. due to the actions of self-weight, variable waves, Level 1 earthquake ground motion, ship berthing, traction by ships, surcharge loads, etc. shall not impair the function of the floating piers, and shall not adversely affect the continuous use of the floating piers.

2 In addition to the provisions of the preceding paragraph, the performance requirements for floating piers in the place where there is a risk of having serious impact on human lives, property, or socioeconomic activity by damage to the floating piers shall be such that the structural stability of the floating piers is not seriously affected even in cases where the function of the floating piers is impaired by design tsunamis, accidental waves, etc.

[Public Notice] (Performance Criteria of Floating Piers)

Article 56

1 The provisions of Article 48, paragraph (1) (excluding item (ii)) apply mutatis mutandis to the performance criteria of floating piers.

2 In addition to the provisions in the preceding paragraph, the performance criteria of floating piers shall be as prescribed respectively in the following items in consideration of the structural type:

(1) The floating pier shall have the dimensions necessary for the containment of their movements and tilting within the allowable range in consideration of the usage conditions.

(2) The risk of capsizing of the floating body under the variable situation in which the dominating action is variable waves shall be equal to or less than the threshold level.

(3) The floating pier shall have the freeboard required for the dimensions of the design ships and the usage conditions.

(4) The following criteria shall be satisfied under the variable situation in which the dominating actions are variable waves, Level 1 earthquake ground motions, ship berthing, traction by ships, and surcharge loads:

(a) The risk of impairing the integrity of the members of the floating body shall be equal to or less than the threshold level.

(b) The risk of impairing the integrity of the members of the mooring equipment of the floating body and losing the structural stability shall be equal to or less than the threshold level.

3 In addition to the provisions of the preceding two paragraphs, the performance criteria of floating piers for which there is a risk of having serious impact on human lives, property, or socioeconomic activity by damage to the facilities shall be such that the degree of damage under the accidental situation in which the dominating actions are design tsunamis or accidental waves is equal to or less than the threshold level.

4 The provisions of Articles 65 and Article 95 apply mutatis mutandis to the performance criteria of the access facilities of the floating body by taking into account the usage conditions.

[Interpretation]

11. Mooring facilities

(10) Performance Criteria of Floating Piers (Article 30, Paragraph 1 of the Ministerial Ordinance and the interpretation related to 56, Paragraph 2 of the Public Notice)

① **Common for floating piers**

(a) The performance requirement for floating piers shall be serviceability. The serviceability mentioned here indicates that the applicable floating pier should have the necessary dimensions such that the amount of motions of the floating body and the amount of its tilting are within the allowable range and that it should have the necessary freeboard based on the dimensions of design ships and its usage conditions.

(b) When the freeboard is set, the dimensions of the design ships and the envisaged usage conditions shall be considered to allow the safe and efficient embarkation and disembarkation of passengers and the safe and efficient handling of cargos.

(c) The performance requirement for floating piers under the variable situation in which the dominating actions are variable waves, L1 earthquake ground motions, ship berthing and traction by ships, and/or surcharges shall be serviceability. The performance verification items and standard indexes to determine the limit values for such actions shall be as shown in **Attached Table 11-26**. The indexes for determining the limit values for the performance verification items shall be appropriately set.

Attached Table 11-26. Performance Verification Items and Standard Indexes for Determination of Limit Values for the Structural Stability of Floating Piers and the Soundness of Members in Each Design State (Excluding Accidental Situations)

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirement</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index for determination of limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="4">30</td><td rowspan="4">1</td><td rowspan="4">2</td><td rowspan="4">56</td><td rowspan="4">2</td><td>2</td><td rowspan="4">Serviceability</td><td rowspan="4">Variable</td><td>Variable waves</td><td>Self-weight, wind, water pressure, water flow</td><td>Capsizing of floating body</td><td>–</td></tr>
<tr><td>4a</td><td>[Level 1 earthquake ground motion]</td><td>(Self-weight, wind, water pressure, water flow)</td><td>Soundness of members of floating body</td><td>–</td></tr>
<tr><td rowspan="2">4b</td><td>[Ship berthing and traction by ships]</td><td>(Self-weight, support reactions of connecting facilities, wind, water pressure, water flow, surcharges)</td><td>Soundness of members of mooring equipment</td><td>–</td></tr>
<tr><td>[Surcharges]</td><td>(Self-weight, wind, water pressure, water flow)</td><td>Structural stability of mooring equipment</td><td>–</td></tr>
</table>

Note: The descriptions [ ] under dominating action indicate that such dominating action is used as a substitute for the design state.

Note: The descriptions ( ) under non-dominating action indicate that such action is used as a substitute based on the dominating action.

(d) Regarding **Attached Table 11-26**, for the performance verification against the capsizing of floating bodies, standard indexes for determining the limit values for capsizing shall be appropriately set by considering the conditions of use of the floating body and the natural conditions. For the performance verification of the structural members of a floating body and floating equipment, standard indexes for determining the limit values for their soundness and stability shall be appropriately set based on the structural type and material of the members.

(e) The performance verification items and standard indexes to determine the limit values for the soundness of the structural members of mooring equipment with mooring ropes under the variable situation in which the dominating actions are variable waves, ship berthing, and/or traction by ships shall be as shown in **Attached Table 11-27**.

Attached Table 11 -27. Performance Verification Items and Standard Indexes for Determination of Limit Values for the Soundness of the Structural Members of the Mooring Equipment of Floating Piers with Mooring Ropes in Each Design State(Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirement	Desig state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
30	1	2	56	2	4b	Serviceability	Variable	Variable waves (Ship berthing and traction by ships)	Self-weight, wind, water pressure, water flow (Self-weight, support reactions of connecting facilities, wind, water pressure, water flow, surcharges)	Yielding of mooring ropes	Design yield stress
										Stability of mooring anchors, etc.	Resistance force of mooring anchors, etc. (horizontal, vertical)

Note: The descriptions [ ] under dominating action indicate that such dominating action is used as a substitute for the design state.
Note: The descriptions ( ) under non-dominating action indicate that such action is used as a substitute based on the dominating action.
Note: The verification of the stability of mooring anchors, etc. refers to verifying that the tensile forces acting on the mooring anchors do not exceed the resistance force.

(f) The term “mooring anchors, etc.” shown in **Attached Table 11-27** is used as a general term for equipment placed on the seabed to retain floating bodies and includes sinkers in addition to mooring anchors.

② **Floating piers of facilities prepared for accidental incidents** (Article 30, Paragraph 2 of the Ministerial Ordinance and the interpretation related to Article 56, Paragraph 3 of the Public Notice)

(a) The performance requirement for floating piers under the accidental situation in which the dominating actions are design tsunamis and/or accidental waves shall be safety. The performance verification items and standard indexes to determine the limit values for such actions shall be as shown in **Attached Table 11-28**.

Attached Table 11-28. Performance Verification Items and Standard Indexes for Determination of Limit Values for the Floating Piers of Facilities Prepared for Accidental Incidents under Accidental Situations

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
30	2	–	56	3	–	Safety	Accidental	Design tsunami	Self-weight, wind, water pressure, water flow	Yielding of mooring ropes	Design yield stress
								Accidental waves		Stability of mooring anchors, etc.	Resistance force of mooring anchors, etc. (horizontal, vertical)

(b) The verification of the stability of mooring anchors, etc. under accidental situation in which the dominating actions are design tsunamis and/or accidental waves shall be examined with the drifting of floating structures taken into account, which may be caused by design tsunamis and/or accidental waves, in order to ensure that it will not make a serious impact on the surroundings.

③ **Access facilities** (Article 30, Paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 56, Paragraph 4 of the Public Notice)

The performance criteria for vehicle ramp, which is an ancillary equipment of the mooring facilities defined in Article 65 of the Standard Public Notice, and the performance criteria of fixed facilities for embarkation and disembarkation of passengers defined in Article 95 of the Standard Public Notice shall apply to the performance criteria of the access facilities of floating piers in accordance with the envisaged conditions of use of the floating pier. The access facilities of floating piers are those that function between a floating body and the land or between floating bodies as passageways of passengers or vehicles, such as access bridges, gang ways, and adjustment towers.

6.1 Fundamentals of Performance Verification

(1) The performance verification procedures for floating piers shall be applied to those with floating bodies that are moored by mooring ropes (e.g., chains, wires, etc.), dolphins, mooring piles, etc. (hereinafter “pontoons”). Furthermore, the Japanese Fire Service Law (Law 186 in 1948), Japanese **Building Standards Law** (Law 201 in 1950), and/or Japanese **Vessel Safety Law** (Law 11 in 1933) apply to some floating structures.

(2) The methods for verifying the performance of floating piers shall be applied to the floating piers installed in places where the actions from waves, tidal currents, and wind are relatively weak. Floating piers are not normally used in locations where the waves or currents are large but are frequently used in locations where the wave height is 1 m or less and where the current is 0.5 m/s or less.

(3) **Figs. 6.1.1** and **6.1.2** show the notation of the respective parts of a floating pier and a pontoon which is a main structure of the floating pier. As shown in the figure, a floating pier comprises pontoons, an access bridge that connects the pontoons with land, gang ways that interconnect the pontoons, mooring ropes that moor the pontoons, mooring anchors, and other elements.

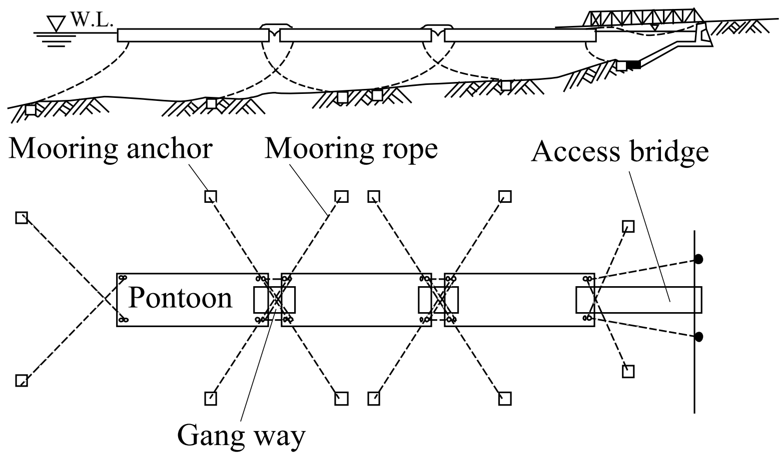

Fig. 6.1.1. Notation of Respective Parts of Floating Pier

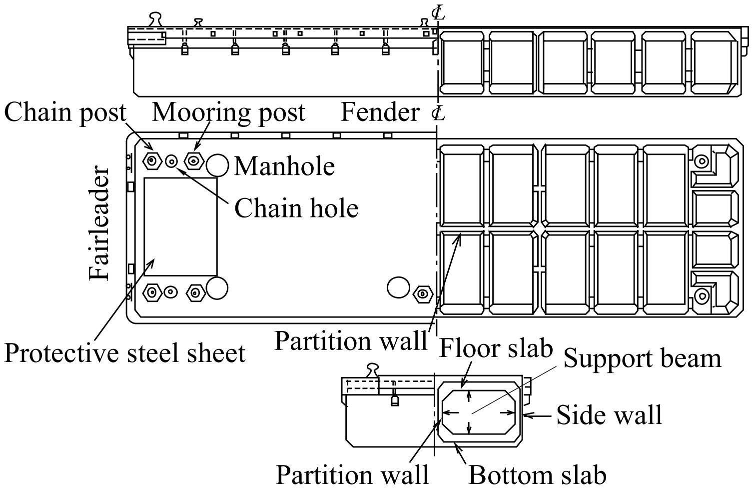

Fig. 6.1.2. Notation of Respective Parts of Pontoon

(4) Floating piers moored by methods other than mooring ropes (e.g., dolphin-fender method) and large-scale floating piers have recently been constructed. **Fig. 6.1.3** and **Fig. 6.1.4** illustrate examples of the overall structure of a large-scale floating pier and its pontoon structure, respectively.

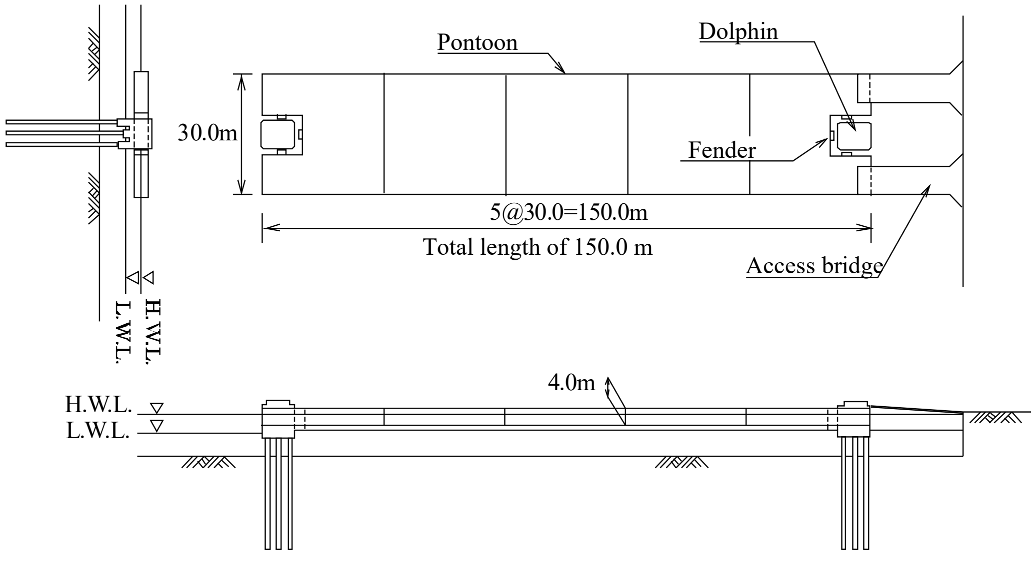

Fig. 6.1.3. Example of Overall Structure of a Large-Scale Floating Pier (Dolphin Mooring Method)

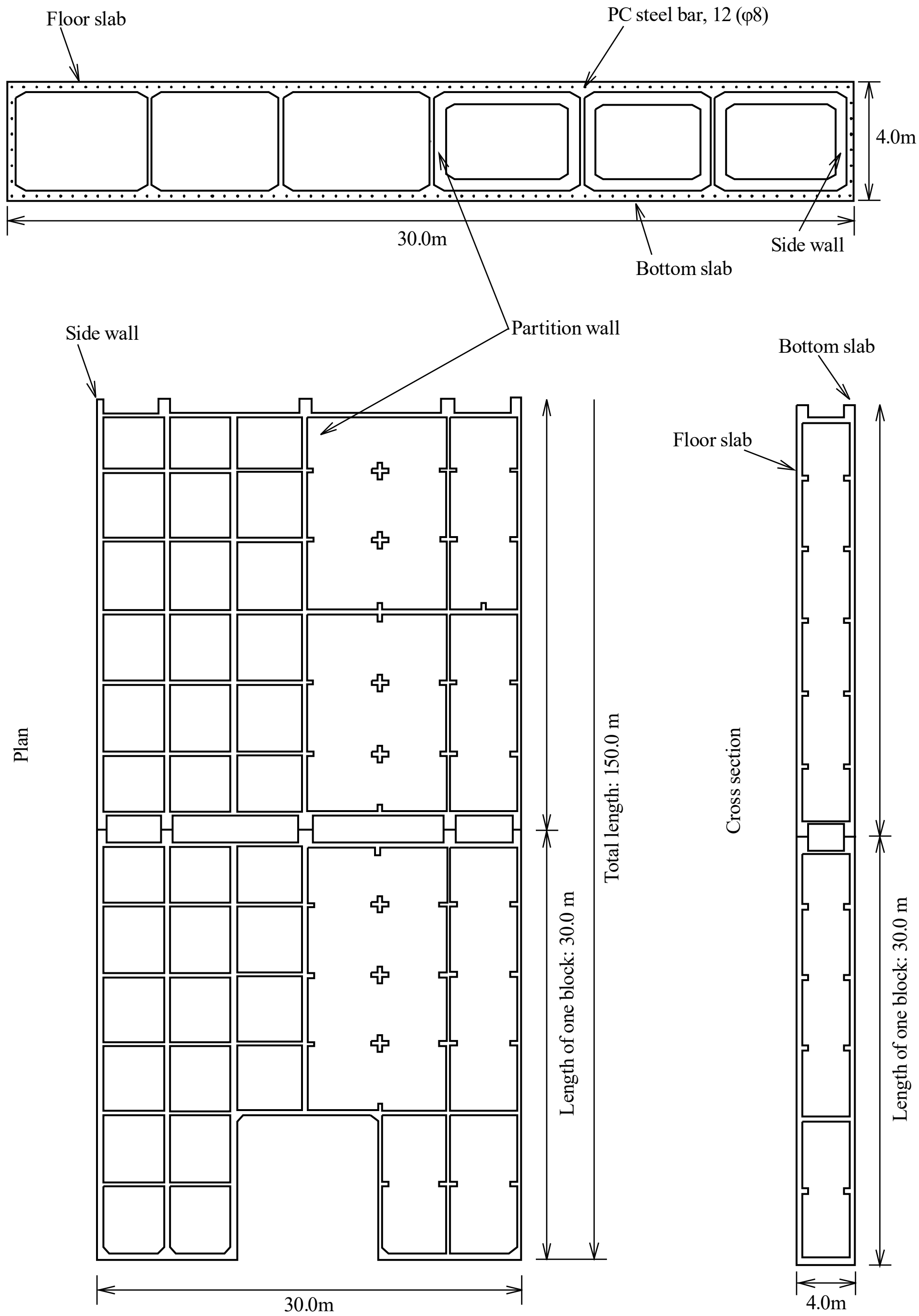

Fig. 6.1.4. Example of Pontoon Structure of a Large-Scale Floating Pier

(5) **The Technical Manual for Floating Structure**[1)] shall be used as a reference for the performance verification of floating piers. Furthermore, **Part II, Chapter 2, 6.4 Wave Force Acting on Structures near the Water Surface**, **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions**, and **Part III, Chapter 4, 3.10 Floating Breakwaters**, etc. shall be used as a reference as necessary. Regarding relatively small floating piers using mooring piles, the **Manual for Design and Construction of Floating Piers (draft)**[2)] can be used as a reference.

(6) For the performance verification of floating piers, the freeboard of the floating pier shall be appropriately set by considering the dimensions of the design ships and the envisaged conditions of use to secure the safety of people and cargos and to allow the efficient embarkation and disembarkation of passengers and the efficient handling of cargos.

(7) Floating piers shall be stable and safe, and their durability should be ensured because they are used for transporting cargos and passengers. Mooring systems such as mooring ropes and mooring anchors shall have sufficient resistance capacity against working actions.

(8) In setting the cross-sectional dimensions of floating bodies of floating piers, the amount of motions of the floating body, the amount of its tilting, etc. shall be appropriately verified to be within the allowable range in accordance with the envisaged conditions of use as necessary.

(9) **Fig. 6.1.5** shows an example of the procedure of the performance verification of floating piers.

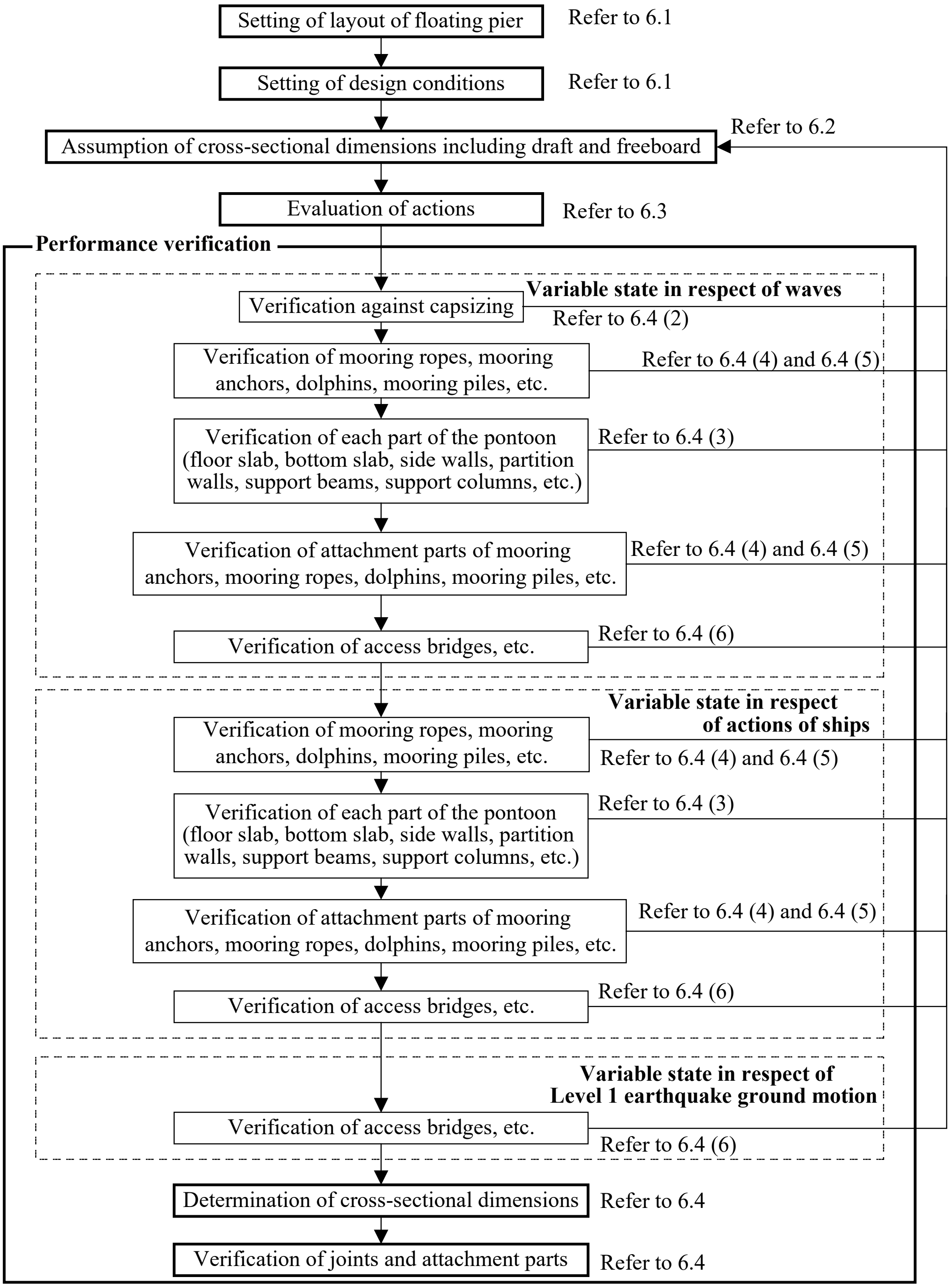

Fig. 6.1.5. Example of Performance Verification Procedure for Floating Piers

(10) The installation locations and arrangement of floating piers shall be determined by considering the types and sizes of design ships, depth of water, water flow, waves, wind, soil of the sea floor, and other factors. The arrangement of floating piers includes jetty type arrangement and parallel-type arrangement.[3)] Generally, the jetty type arrangement is preferable due to the lower construction cost and ease of mooring ships. One to three pontoons are usually adopted in Japan when the pontoons are connected. However, recently, some large-scale floating piers have only one pontoon.

(11) When pontoons oscillate subjected to waves and when they roll due to deviations in the vertical directions of the two pontoons, one may hit the other and make a hole on it. To avoid such a problem, two pontoons shall be strongly united, or fenders shall be installed between pontoons. The length of the gang way between the pontoons may be approximately double the interval between the pontoons. Generally, chains are used to unite neighboring pontoons. At this time, the chains are installed onto the chain posts via the fairleaders.

(12) The types of pontoons are broadly classified as follows on the basis of raw materials: pontoons made from reinforced concrete, steel, PC, FRP, and wood and hybrid pontoons. The characteristics of each type are shown below:

① Reinforced concrete pontoons with excellent durability draw deep; therefore, they usually do not oscillate to a great extent. The construction, maintenance, and repair costs are lower than those for steel pontoons, but they are vulnerable to impact, and their imperviousness is rather low. Therefore, to bolster imperviousness, the content of concrete needs to be higher and reliable construction is required. Regarding reinforcing bars, the use of many small-diameter bars is recommended to enhance the resistance to impact.

② Impact-resistant steel pontoons are easier to manufacture and repair, but they corrode. Therefore, their durability is lower than that of reinforced concrete pontoons. However, they draw lighter than reinforced concrete pontoons; therefore, they are not significantly affected by water flow.

③ PC pontoons with excellent imperviousness resist cracking compared with reinforced concrete pontoons, and it is also an advantage that thickness of the members can be thinner.

④ Lightweight FRP pontoons draw light; therefore, they are unstable but are highly durable and easier to install. They are currently used for small-scale floating piers (e.g., marinas).

⑤ The construction cost of wood pontoons is low, but their imperviousness is inferior, and their durability is low because they are vulnerable to decay and insect damage. To secure their imperviousness and antisepticize them, they need to be often pulled up and repaired.

⑥ RC hybrid pontoons refer to steel floating bodies that are surrounded by protective reinforced concrete. The steel members and concrete members work as one body to resist loads. Their imperviousness is high, and they resist corrosion. The weight of this type of a floating body is between that of a reinforced concrete pontoon and that of a steel pontoon.

⑦ For PC hybrid pontoons, PC inner members (support beams and partition walls) are combined with steel materials. The weight of a floating body in this type can be lighter than that of a PC pontoon. Their imperviousness is high, and they resist corrosion.

(13) For the raw materials of pontoons, the **Standard Specifications for Concrete Structures [Design]**,[4] the **Rules for the Survey and Construction of Steel Ships; Guidance for the Survey and Construction of Steel Ships Part Q (Steel Barges)**,[5] and other standards shall be used as a reference.

(14) There are several types of mooring methods such as the chain or wire method, in which mooring ropes are used, and the dolphin-fender method, in which dolphins are used.[1] For small-scale floating piers, a mooring method that uses mooring piles is often used. The method to be applied shall be appropriately selected on the basis of the performance required for floating piers and other factors.

(15) Access bridges are movable bridges. There are two types of such bridges: One bridge has a land side that is secured with hinges, and its other side with rollers is on a pontoon; the other bridge is hung with an adjustment tower.[3] At a place where motions due to waves is minimal, the former type can be used without using adjustment towers. The length and width of an access bridge shall be appropriately determined by considering the conditions of use of the floating pier.

6.2 Setting of Basic Cross-section

(1) A pontoon shall have a surface area and freeboard appropriate for its purpose of utilization. The dimensions of a pontoon shall make it stable against the external actions on it.

(2) The length of many pontoons is 20 to 40 m, the width is less than 15 m, and the height is 2 to 4 m. Recently, large-scale floating piers have been constructed.

(3) As standard dimensions of various sections in a pontoon, the length of a single side of a floor slab, side wall, bottom slab, and partition wall is 1 to 3 m. The thickness of the side wall and bottom slab of a reinforced concrete

pontoon is often 15 to 20 cm and that of a floor slab and partition wall is 10 to 20 cm. For steel pontoons, the thickness of them is often 6 to 10 mm. The ratio of the side to the length of each slab shall desirably be a value close to one.

(4) The freeboard of a pontoon shall be set to an appropriate height to provide good conditions for cargo handling and passenger use when it is fully loaded and lightly loaded with cargo and passengers. Normally, the height is set to approximately 1.0 m. The freeboard can be calculated using **equation (6.2.1)**.

$$h' = d - \frac{W_1}{\gamma_W A} \tag{6.2.1}$$

where

h' : freeboard (m)

d : pontoon height (m)

W_1 : pontoon weight (kN)

γ_W : unit weight of seawater (kN/m^3)

A : horizontal cross-sectional area of the pontoon (m^2)

(5) In the case of a reinforced concrete pontoon, the dimensions shall be determined by considering the imperviousness of the concrete.

(6) To enable the facilities to exert its functions, the mooring method of pontoons of a floating pier shall be appropriately determined on the basis of the scale of the pontoons, water depth at the installation location, soil of the sea floor, and other natural conditions.

(7) Regarding the types of mooring methods, normally a chain method or a wire method is used in fairly deep water. For shallow water depths, an intermediate buoy method, an intermediate sinker method, or a dolphin-fender method are mainly used.[1] The mooring method shall be selected on the basis of a comparison of the functions and stability of the floating pier and the characteristics of the mooring facilities.

(8) The dimensions and incline of access bridges and gang ways shall be appropriately determined by considering the capacity required from floating piers for transporting passengers and cargos.

(9) For the width and incline of access bridges and gang ways on which vehicles pass, **Part III, Chapter 5, 9.8 Vehicle Loading Facilities** shall be referred to.

(10) The incline of access bridges and gang ways on which passengers pass is often 5% to 20% at the L.W.L.

(11) Regarding the dimensions of access bridges and gang ways, the width is often 2 to 6 m, the span length for access bridges is 10 to 30 m, and the span length for gang ways is 2 to 6 m, according to the examples constructed in the past.

6.3 Actions

(1) Among the actions applied to pontoons, the live load shall be determined on the basis of the type of passengers and cargos that will be transported using pontoons.

(2) When pontoons are subject to a reaction force from the access bridges, they tilt; therefore, ballast is placed as a counterweight in some cases. The weight shall be determined such that it matches the reaction force caused by the self-weight of an access bridge, and it works to make the pontoon horizontal.

(3) The fender reaction force, wave force, current force, etc. do not need to be considered unless necessary. However, when there is an anticipated risk that the pontoon may be subjected to wave actions, the following forces shall be considered: the wave forces exerted upon the stationary pontoon that is assumed to be rigidly fixed in position and the fluid forces due to the motions of the pontoon.[6] For these forces, **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions** shall be referred to. In this case, the mooring forces should be calculated by considering the motions of the pontoon.

(4) A sidewalk live load of 5.0 kN/m^2 is commonly used for floating piers, which are mainly used for passenger ships. For the live load, the vehicle load shall be taken into account when floating piers are used for ferries and when

vehicles are allowed to get on them. For vehicle loads used for performance verification of floor slabs, the T-load specified in the **Specifications and Commentaries for Highway Bridges, Part I Common**[7] can be generally used.

(5) The fender reaction forces used in the performance verification of mooring ropes, etc. can be calculated by referring to **Part II, Chapter 8, 2.2 Actions Caused by Ship Berthing** and **Part II, Chapter 8, 2.3 Actions Caused by Ship Motions**. Furthermore, for the tractive forces of ships, refer to **Part II, Chapter 8, 2.4 Actions Caused by Traction of Ships** shall be referred to.

(6) The wave forces used in the performance verification of mooring ropes, etc. can be calculated by an appropriate method by referring to **Part II, Chapter 2, 6.4 Wave Force Acting on Structures near the Water Surface** and **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions**. At this time, the drag coefficient for cubes may be used. The area over which the drag force acts can be taken to be the area below the still water surface. The abovementioned wave forces are those that act on a stationary pontoon. However, if the natural period of the motions of the pontoon is close to the period of the waves, resonance may occur, thus causing large forces in the mooring ropes. This point should be carefully considered. In particular, for floating piers located in places where it is envisaged that swells and other long period waves penetrate, it is preferable that a motion analysis of the moored floating bodies be performed using a numerical simulation method.[8]

(7) In the performance verification of mooring ropes, etc., when the mooring systems are not tense, the influence of wave drift force (higher-order component) is large in addition to wave-exciting force; therefore, it should be appropriately evaluated.

(8) The water flow velocity used in the performance verification of mooring ropes, etc. can be determined on the basis of the instructions in **Part II, Chapter 2, 7 Water Currents**, but it is desirable to determine it through actual measurements. Furthermore, the tidal current force can be calculated by referring to **Part II, Chapter 2, 7.2 Fluid Force due to Currents**. The drag coefficient can be calculated in a similar way as that for wave forces.

6.4 Performance Verification

(1) Normally the following items shall be examined for the verification of the stability of floating piers.

① Pontoon stability

② Stability of each part of the pontoon

③ Stability of the mooring system (e.g., mooring ropes, mooring anchors, dolphins, and mooring piles)

④ Stability of access bridges and gang ways

(2) Performance Verification of the Stability of Pontoons

① The structural stability levels required for the pontoons shall be appropriately secured in accordance with the conditions of use and other conditions. In examining the stability of a pontoon, the following requirements must be satisfied:

(a) The pontoon must satisfy the stability condition of a floating body with the required freeboard against the actions of the reaction force from the access bridge supporting point, full surcharge on the deck, and even against the presence of some water inside the pontoon owing to leakage of the pontoon.

(b) Even when the full surcharge is placed on only one side of the deck divided by the longitudinal symmetrical axis of the pontoon and the reaction force from an access bridge supporting point acts on this side, if the bridge is attached there, the pontoon must satisfy the stability condition of a floating body. Furthermore, the inclination of the deck must be equal to or less than 1:10 with the smallest freeboard of 0 or more.

② Generally, the height of the water accumulated inside the pontoon by leakage shall be 10% of the height of the pontoon in the examination of pontoon stability. In most cases, the freeboard to be maintained is approximately 0.5 m.

③ When subjected to a uniformly distributed load, the pontoon can be regarded stable if **equation (6.4.1)** is satisfied. **Fig. 6.4.1** illustrates the stability of a pontoon subjected to an eccentric load.

$$\frac{I\gamma_W}{W} - \overline{\mathrm{CG}} > 0 \tag{6.4.1}$$

where

I : geometrical moment of the inertia of the cross-sectional area at the still water level with respect to the longitudinal axis of the pontoon (m^4)

W : weight of the pontoon and uniformly distributed load (kN)

γ_W : unit weight of seawater (kN/m^3)

C : center of buoyancy of the pontoon

G : center of gravity of the pontoon

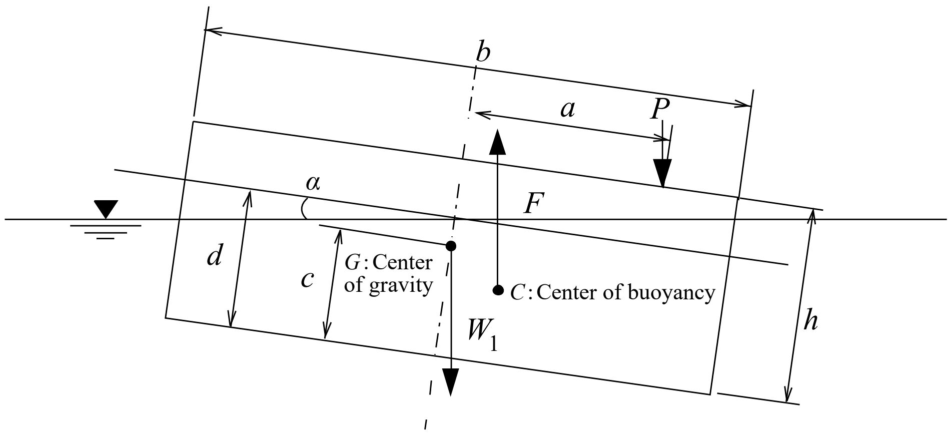

Fig. 6.4.1. Stability of Pontoon Subjected to Eccentric Load

When the pontoon is partially filled with water by leakage, the pontoon can be regarded stable when **equation (6.4.2)** is satisfied. W, I, C, and G of the equation shall refer to the states for water leakage inside the pontoon.

$$\frac{\gamma_W}{W}\left(I - \sum i\right) - \overline{\mathrm{CG}} > 0 \tag{6.4.2}$$

where

i : geometrical moment of the inertia of the water surface inside each chamber with respect to its central axis parallel to the rotation axis of the pontoon (m^4)

When subjected to an eccentric load, it shall be checked if the value of tan α obtained by solving **equation (6.4.3)** satisfies **equation (6.4.4)**. α is generally very small; therefore, cos$^2\alpha$ in **equation (6.4.3)** can be 1-tan$^2\alpha$ approximately.

$$(W_1 + P)\left\{\frac{b^2 \tan\alpha}{12d\cos^2\alpha} - \left(\frac{b^2}{24d}\tan^2\alpha + c - \frac{d}{2}\right)\tan\alpha\right\} - p\{a + (h-c)\tan\alpha\} = 0 \tag{6.4.3}$$

$$\left.\begin{aligned} &\tan\alpha < \frac{2(h-d)}{b} \\ &\tan\alpha < \frac{1}{10} \end{aligned}\right\} \tag{6.4.4}$$

where

W_1 : weight of the pontoon (kN)

P : total force of the eccentric load (kN)

b : width of the pontoon (m)

h : height of the pontoon (m)

d : draft of the pontoon when P is applied to the center of the pontoon (m)

c : height of the center of gravity of the pontoon measured from the bottom (m)

a : deviation of P from the center axis of the pontoon (m)

α : inclination angle of the pontoon (°)

④ Regarding concepts on the stability of pontoons by water leakage, the **Theoretical Naval Architect**[9) can be used as a reference.

(3) Performance Verification of the Stability of Each Part of a Pontoon

① The stresses, etc. generated in the structural parts of the pontoon shall be examined by using an appropriate method selected by considering the usage conditions of the pontoon, external actions on the respective parts, their structural characteristics etc.

② A floor slab can normally be verified for performance as a two-way slab fixed on four sides with support beams and side walls against the actions that yield the largest stress out of the following combinations of actions:

(a) When only a static load acts on a pontoon

(static load) + (self-weight)

(b) When a live load acts on a pontoon

(live load) + (self-weight)

(c) When the supporting point of an access bridge is set on a pontoon without adjustment tower

(supporting point reaction force of an access bridge) + (self-weight)

③ A side wall can normally be verified for performance as a two-way slab fixed on four sides with a floor slab, a bottom slab, and side walls or support beams against hydrostatic pressure acting when the pontoon submerges by 0.5 m above the deck.

④ A bottom slab can normally be verified for performance as a two-way slab fixed on four sides with side walls or support beams against hydrostatic pressure acting when the pontoon submerges by 0.5 m above the deck.

⑤ A partition wall can normally be verified for performance as a slab fixed on four sides against water pressure acting when one compartment has become fully waterlogged.

⑥ The support beams of the floor slab, bottom slab, and side walls and the center support columns can normally be calculated as a rigid frame box against water pressure acting when the maximum load is on the floor slab of the pontoon and the draft of the pontoon is equal to its height. Furthermore, the performance of the center support columns and the support beams of the side walls can be verified as members that receive bending force and compressive force in the axial direction. Support beams can be regarded T-beams, but their performance can be generally verified as rectangle beams.

⑦ The effective spans of slabs shall be intervals between the centers of support beams, floor slabs, side walls, bottom slabs, etc.

⑧ For the bending moment of a four-side fixed slab that receives a uniformly distributed load, [**Reference (Part III), Chapter 4, 2 Numerical Table for Bending Moment of Slab** shall be used. The bending moment of a four-side fixed slab, when it receives trapezoidal water pressure, can be evaluated as follows: the water pressure is divided into a uniformly distributed load of 5.0 kN/m^2 and a triangular load that is zero at the upper end of the pontoon and that increases by 10.0 kN/m^2 for every 1.0 m, and then both loads are added.

⑨ The live load can be regarded a T-load, and performance can be verified by handling the live load as a concentrated load. For the performance verification of slabs, the **Specifications and Commentaries for**

Highway Bridges, Part II Steel Bridges and Steel Members[10] and the **Specifications and Commentaries for Highway Bridges, Part III Concrete Bridges and Concrete Members**[11] shall be referred to.

⑩ The supporting point reaction force of an access bridge can be acted at the center of the slab as a concentrated load. Furthermore, the weight of a steel plate to be constructed under the supporting points of an access bridge shall be considered as self-weight.

⑪ When the wave actions are considered, the calculations of section forces can be made using Müller's equation[12] and other rules. When it is necessary to consider the motions of the floating body, wave parameters, the effect of water depth, etc., the method with cross-sectional division by Ueda et al. [6) 13) 14)] can be used.

⑫ The imperviousness of concrete must be fully ensured. Surface coating with epoxy resin and polyurethane resin can be applied as measures for imperviousness.

(4) Performance Verification of the Stability of Mooring Ropes and Other Mooring Equipment

① The structure of mooring ropes shall be examined by using an appropriate method in such a way that the ropes can hold securely a pontoon in position under the action of whichever force is the largest among the fender reaction force generated during berthing, the tractive force of the ship, and the wave force, with the addition of the tidal current force to each of the aforementioned forces.

② Fundamentally, the chain method is described here because it is assumed to be the most applied. For other mooring methods, **Reference 1)** shall be referred to.

③ Chains are usually installed onto the pontoon's chain posts at the four corners through the chain holes, and the pontoon is secured to the sea floor with mooring anchors.

④ The chains are normally crossed under a pontoon as shown in **Fig. 6.1.1** such that they do not hinder ships from berthing. However, the chains may rub against each other and may abrade depending on the crossing way; therefore, attention is required.

⑤ The length of a mooring rope is usually five times the water depth plus the tidal range. When a chain is stretched, the following points shall be considered:

(a) The chain shall not be overstretched during high tide because overstretching can exert excessive tension force on the chain.

(b) There shall be no interference with ship berthing during high tide.

(c) Sufficient anchor holding power shall be ensured for mooring anchors during high tide.

(d) The amount of horizontal movement of the pontoon during low tide shall be small.

⑥ The anchor holding power of steel mooring anchors is significantly reduced when the angle between the chain at the attachment part and the horizontal surface is 3° or higher.

⑦ There are chains with studs and without studs. Appropriate chains shall be selected for floating piers. For example, **Flash Butt Welded Anchor Chains (JIS F 3303)** can be used.

⑧ The diameters of chains shall be determined such that they will not break owing to the actions specified in **Part III, Chapter 5, 6.3 Actions** at high tide. The allowable tension of chains at this time can be one-third of the breaking test load specified in **JIS F 3303** mentioned above.[1] The weight of chains can be the minimum weight specified in **JIS F 3303** mentioned above.

⑨ In the determination of the diameter of the chain, careful consideration shall be given to the abrasion, corrosion, and biofouling of the chain. Furthermore, appropriate maintenance work shall be performed on the chain, including periodical checks and replacement as necessary.

⑩ When determining the chain diameter with a numerical simulation of motions, the characteristics of the displacement-restoration force relationship of the mooring system shall be determined using an appropriate method such as catenary theory, etc.[15]

⑪ The maximum tension acting on each chain is ideally calculated by using dynamic analysis on the chain and the pontoon, but static analysis may also be used. A chain can normally be verified for performance on the condition that only one chain is assumed to resist all external actions as shown in **Fig. 6.4.2**.

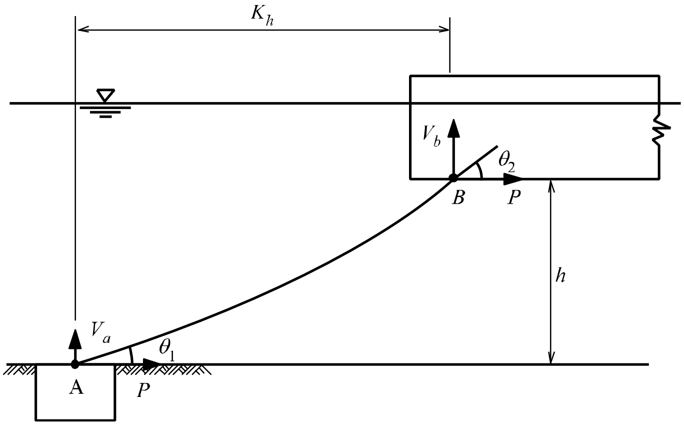

Fig. 6.4.2. Performance Verification of Mooring Rope

Assuming that the chain forms a catenary line, the maximum tension acting on the chain is given by **equation (6.4.5)**.

$$T = P \sec \theta_2 \tag{6.4.5}$$

where

T : maximum tension acting on the chain (kN)

P : horizontal external action (kN)

θ_2 : angle that the chain makes with the horizontal plane at the attachment point between the chain and the pontoon (°)

The horizontal force acting on the mooring anchor is the same as the horizontal force acting on the pontoon, and the vertical force acting on the anchor is given by **equation (6.4.6)**.

$$V_a = P \tan \theta_1 \tag{6.4.6}$$

where

V_a : vertical force acting on the mooring anchor (kN)

θ_1 : angle that the chain makes with the horizontal plane at the attachment point between the mooring anchor and the chain (°)

The vertical force acting on the attachment point between the chain and the pontoon can be expressed by **equation (6.4.7)**.

$$V_b = P \tan \theta_2 \tag{6.4.7}$$

where

V_b : vertical force acting on the attachment point between the chain and the pontoon (kN)

Angles θ_1 and θ_2 can be calculated using **equation (6.4.8)** by assuming a chain length l and a chain weight per unit length w.

$$\left.\begin{aligned} l &= \frac{P}{w}\left(\tan\theta_2 - \tan\theta_1\right) \\ h &= \frac{P}{w}\left(\sec\theta_2 - \sec\theta_1\right) \end{aligned}\right\} \tag{6.4.8}$$

where

l : length of the chain (m)

h : water depth under the bottom of the pontoon (m)

w : weight per unit length of the chain in water (kN/m)

The horizontal distance between a mooring anchor and the pontoon can be expressed by **equation (6.4.9)** when a horizontal force acts on the pontoon. By using this equation, the amount of the horizontal movement of the pontoon from its stationary position under no horizontal force can be calculated.

$$K_h = \frac{P}{w}\left\{\sinh^{-1}\left(\tan\theta_2\right) - \sinh^{-1}\left(\tan\theta_1\right)\right\} \tag{6.4.9}$$

where

K_h : horizontal distance between the mooring anchor and the attachment point between the pontoon and the chain (m)

Considering that the catenary line of the chain with normal diameter can be approximately represented with a straight line, it can be assumed in **equations (6.4.5)** to **(6.4.9)** that $\theta_2 = \theta_1 = \sin^{-1}(h/l)$, and K_h can be approximately expressed by $:K_h = \sqrt{l^2 - h^2}$

(5) Performance Verification of the Stability of Mooring Anchors

① A mooring anchor shall be capable of providing the resistance forces required to keep the pontoon stable against the maximum tension acting on the mooring rope and shall have an appropriate stability.

② For the performance verification of the stability of mooring anchors, **equation (6.4.10)** can be used. Furthermore, the adjustment factor can be taken to be an appropriate value equal to or greater than 1.2.

$$\left.\begin{aligned} R_h &\geq mP \\ R_v &\geq mV_a \end{aligned}\right\} \tag{6.4.10}$$

where

R_h : horizontal resistance force of the mooring anchor (kN)

R_v : vertical resistance force of the mooring anchor (kN)

P : horizontal force acting on the mooring anchor (kN)

V_a : vertical force acting on the mooring anchor (kN)

m : adjustment factor

$V_a = P\tan\theta_1$ can be used for the calculation.

③ The following forces are normally considered the resistance forces of a mooring anchor, but in-situ stability tests are preferred for a mooring anchor:

(a) In the case of concrete block:

1) For clay ground:

- Horizontal resistance force R_h: cohesion of the surfaces of the bottom and sides, the difference between the passive and active earth pressures
- Vertical resistance force R_v: weight in water, effective overburden weight in water

2) For sand ground:

- Horizontal resistance force R_h: bottom friction force, the difference between the passive and active earth pressures
- Vertical resistance force R_v: weight in water, effective overburden weight in water

The vertical force used in the calculation of the bottom friction force is the difference between the weight of the block in water and the vertical component of the chain tension acting on the block.

(b) In the case of steel mooring anchor:

- Horizontal resistance force R_h: holding power
- Vertical resistance force R_v: weight in water

The holding power of a steel mooring anchor T_A can be calculated by **equation (6.4.11)**.

$$\left.\begin{array}{ll} \text{On soft mud:} & T_A = 17W_A^{2/3} \\ \text{On hard mud:} & T_A = 10W_A^{2/3} \\ \text{On sand:} & T_A = 3W_A \\ \text{On flat rock:} & T_A = 0.4W_A \end{array}\right\} \qquad \textbf{(6.4.11)}$$

where

T_A : holding power of the mooring anchor (kN)

W_A : weight of the mooring anchor in water (kN)

④ Generally, concrete blocks are used as mooring anchors. Steel mooring anchors are often used for sandy soil.

⑤ When concrete blocks are used as mooring anchors, they should be embedded under the sea floor. The weight of many concrete blocks is approximately 150 to 700 kN. Concrete blocks that were reinforced with additional bars were used in some cases.

⑥ When a rectangular solid anchor block is deeply embedded in cohesive soil, Hansen obtained **equation (6.4.12)** for the horizontal resistance force by assuming the slip surface around the block. [16)]

$$P = 11.4ch \qquad \textbf{(6.4.12)}$$

where

P : resistance force of the block per unit width (kN/m)

c : cohesion of the cohesive soil (kN/m^2)

h : height of the block (m)

Mackenzie experimentally obtained **equation (6.4.13)** for blocks embedded to a depth of 12 times or more the height of the block. [16)]

$$P = 8.5ch \qquad \textbf{(6.4.13)}$$

⑦ For the mooring anchors of concrete blocks, anchor rings are generally installed as shown in **Fig. 6.4.3**. For the performance verification of anchor rings, **Part III, Chapter 2, 2 Structural Members** shall be referred to.

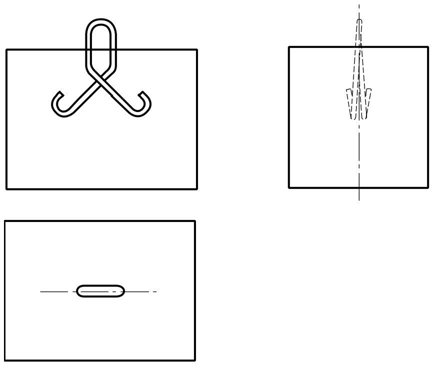

Fig. 6.4.3. Example of Concrete Mooring Anchor

(6) Performance Verification of the Stability of Access Bridges and Gang Ways

① The performance of access bridges and gang ways can be verified by referring to the **Specifications and Commentaries for Highway Bridges, Part II Steel Bridges and Steel Members**[10)] and the **Technical Standard and Commentary of Grade Separation Facilities for Pedestrians**.[17)]

② In the performance verification of access bridges and gang ways, attention shall be desirably paid so that elderly people and physically impaired persons can safely move on wheelchairs and other similar equipment. Furthermore, for floating piers on which low-floor vehicles may pass, sufficient separation distance shall be provided such that the floors of vehicles do not come into contact with the access bridges and gang ways.

③ Generally, steel access bridges are used for connections to the land. Pony trusses or plate girders are often used. Gang ways between pontoons are often steel-plate girders, I-beams, pony trusses, or slabs.

④ For vehicle loads, the **Specifications and Commentaries for Highway Bridges, Part I Common**[7)] shall be referred to.

⑤ As bearings of access bridges connecting to the land, when the pontoons do not oscillate considerably, hinges are used on the land side, roller bearings are used on the pontoon side, or adjustment towers are used for hanging. For the pontoons that greatly oscillate, access bridges are simply placed between steel protection plates installed onto the pontoons and the land coasts, and chains are normally used as anchors for the pontoons and the land coasts to prevent the access bridges from falling down. To improve passage from the access bridges to the pontoons, apron plates are installed at the ends of the access bridges on the pontoon side when necessary. Steel plates are usually installed for protection onto pontoon floor slabs with which roller bearings come into contact. Furthermore, as bearings of gang ways between pontoons, hinges are used on one side, and roller bearings are used on the other side in most cases. There were some cases in which the hinges broke because forces acted on the access bridges within the horizontal planes owing to the motions of the pontoons; therefore, attention is required.

⑥ For access bridges connecting to the land, adjustment towers are installed on the access bridges on the pontoon side in some cases to reduce the supporting point reaction force acting on the pontoons and to wind up the access bridges in stormy weather. There are some types of adjustment towers: one is an adjustment tower that is manually modified on the basis of tide level at all times; another is an adjustment tower for which the most weight of an access bridge is hung with the counterweights of the adjustment tower, and a part is placed on the pontoon to allow the bridge to automatically move up and down on the basis of tide level. Many adjustment towers are made from reinforced concrete and steel frames. An adjustment tower has steel pulleys, counterweights, and hanging materials. **Fig. 6.4.4** illustrates an example of an adjustment tower. For adjustment towers, the stability required to satisfy the functions against the reaction force of access bridges and the actions of earthquake ground motion shall be secured.

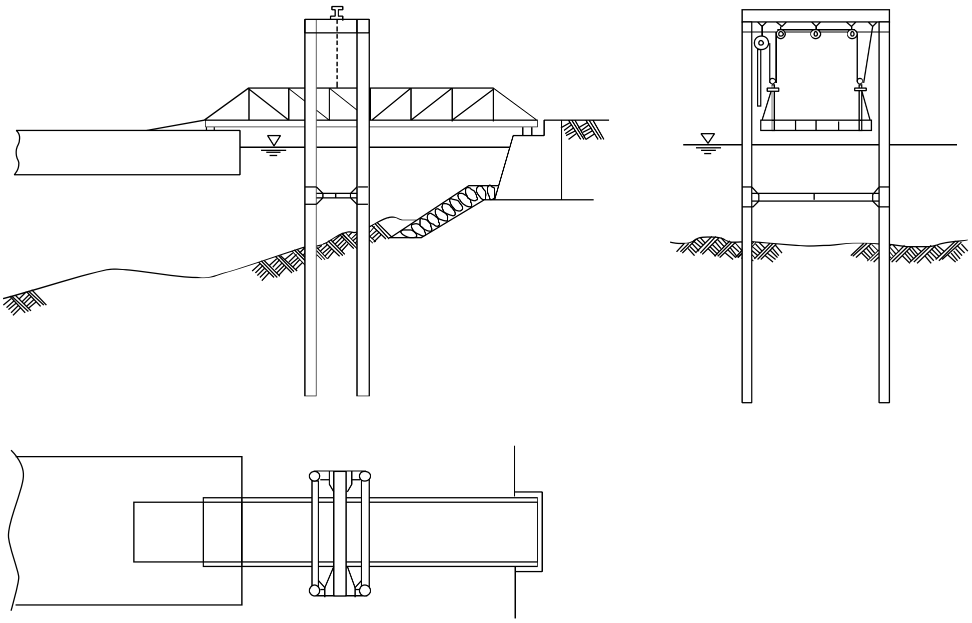

Fig. 6.4.4 Example of Adjustment Tower

6.5 Structural Specifications

Handrails shall be installed onto access bridges and gang ways as safety facilities. Curbing shall be installed on them as necessary.

[References]

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7 Shallow Draft Wharves

(English translation of this section from Japanese version is currently being prepared.)

7.1 Common for Shallow Draft Wharves

(English translation of this section from Japanese version is currently being prepared.)

7.2 Actions

(English translation of this section from Japanese version is currently being prepared.)

7.3 Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

8 Boat Lift Yards

[Ministerial Ordinance] (Performance Requirements for Boat Lift Yards)

Article 32

The performance requirements for boat lift yards shall be as prescribed respectively in the following items in consideration of the structural type:

(1) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable the safe and smooth lifting and launching of boats.

(2) Damage to boat lift yards, etc. due to self-weight, earth pressure, water pressure, variable waves, berthing and traction of boats, Level 1 earthquake ground motions, surcharge loads, etc. shall not impair the function of the boat lift yards and shall not adversely affect the continuous use of the boat lift yard.

[Public Notice] (Performance Criteria for Boat Lift Yards)

Article 58

1 The performance criteria for boat lift yards shall be as prescribed respectively in the following items:

(1) The boat lift yard shall have the necessary water depth and length in consideration of the dimensions of the design ships.

(2) The boat lift yard shall have the necessary crown height in consideration of the tidal range, the dimensions of the design ships and the usage conditions.

(3) The boat lift yard shall have the necessary ancillary equipment in consideration of the usage conditions.

2 The provisions of Article 49 through Article 52 apply mutatis mutandis to the performance criteria of the front wall portion of the boat lift yard in consideration of the structural type.

3 The performance criteria for the pavement of the boat lift yard shall be as prescribed respectively in the following items:

(1) The pavement of the boat lift yard shall have the dimensions necessary for enabling the safe and smooth handling of boats.

(2) The risk of impairing the integrity of the pavement under the variable situation, in which the dominating action is the surcharge load, shall be equal to or less than the threshold level.

(3) The risk of impairing the integrity of the pavement of the slip way under the variable situation, in which the dominating actions are water pressure and variable waves, shall be equal to or less than the threshold level.

[Interpretation]

11. Mooring facilities

(12) Performance Criteria for Boat Lift Yards (Article 32 of the Ministerial Ordinance and the interpretation related to Article 58 of the Public Notice)

① The provisions of Article 49 (performance criteria of gravity-type wharfs) through Article 52 (performance criteria of cell-type wharfs) in the Reference Public Notice and their interpretations shall be applied with modification as necessary to the performance criteria and interpretations for the front wall portions of boat lift yards in consideration of the structural type. Necessary performance verification items for the front wall portions of boat lift yards shall appropriately be selected from those defined in the articles.

② The pavements of boat lift yards shall have serviceability as their performance requirement in variable situations where the dominating actions are surcharges. **Attached Table 11–29** shows performance verification items and standard indexes to determine limit values for such actions. For the performance verification of the pavements of boat lift yards, standard indexes to determine limit values for the integrity shall be appropriately set based on the material quality and other factors.

Attached Table 11–29 Performance Verification Items and Standard Indexes to Determine Limit Values for the Pavements of Boat Lift Yards in Various Design Situations (excluding accidental situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
32	1	2	58	3	2	Serviceability	Variable	Repeated surcharges	–	Soundness of the pavement	–
					3			Water pressure [Variable waves]	–	Soundness of the pavement of the slip way	–

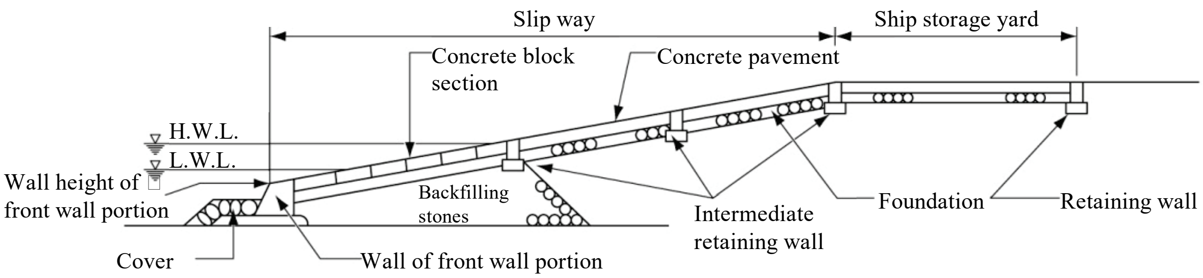
Note: The descriptions [xx] under Dominating action indicate that such dominating action is used as a substitute for the design situation.

8.1 Fundamentals of Performance Verification

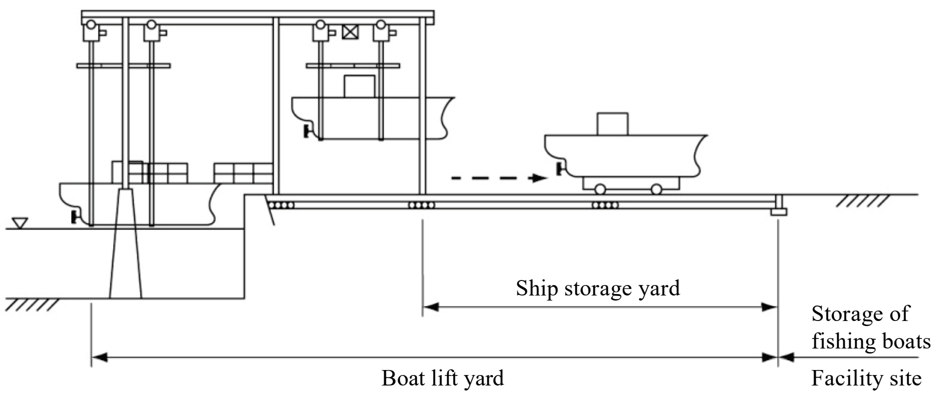
(1) A boat lift yard is a facility used to retrieve ships to the land and launch to the sea for such purposes as repair, refuge from storm waves and storm surges, and land storage of ships during winter.

(2) In many cases, rails or cradles are employed in the retrieving and launching of ships of 30 tons or larger in gross tonnage, but the provisions in this section can be applied to the performance verifications of the facilities used to lift and launch ships smaller than 30 tons in gross tonnage directly over the slope of slip way.

(3) The structure of boat lift yards is broadly divided into the pull-up type (slip way type) and lifting type. **Fig. 8.1.1** shows main components of both structural types.



(a) Pull-up type (slip way type)



(b) Lifting type

Fig. 8.1.1 Boat Lift Yard

(4) As fishing boats have been becoming more energy efficient, small ships have also been following this trend. Ships smaller than 30 tons in gross tonnage require rail-type slip ways in some cases. Rails are required because larger keels are used because of larger propellers and such keels get out from the bottoms of ships. This makes it impossible to directly pull up such ships onto a slope and lower them.

(5) In addition to improved energy efficiency of small ships, the number of lift-type docking and undocking facilities has also been increasing to make docking and undocking quick and to ensure safety.

8.2 Performance Verification

For performance verification of boat lift yards, **Design Guidelines for Fishing Port and Fishing Ground Facilities, Part 6 Mooring Facilities**[1] can be referred to.

8.3 Location Selection of Boat Lift Yard

(1) Location of boat lift yards needs to be determined in such a way that the following requirements are satisfied:

① The front water area is calm.

② The front water area is free from siltation or scouring.

③ Navigation and anchorage of other ships are not hindered.

④ There is an adequate space in the background for the work for ship lifting and launching as well as for ship storage.

(2) A boat lift yard is a slope, so waves easily go up it and they hinder the usage and moreover may cause disaster, so a calm place shall be selected to install a yard.

(3) A place for which the water area at the front would be easily buried because of littoral drift or river sediment load or it would be easily scoured because of water flow or waves requires maintenance and repair work, so such place shall be avoided as much as possible.

(4) Usually, on the back of a boat lift yard, hoists, rails, repair facilities, and other equipment are installed and fishing equipment is temporarily placed. Also, vehicles drive in there or the space is used for other purposes, so a sufficiently large site shall desirably be secured.

(5) The water area at the front shall be sufficiently large such that retrieving and launching of ships and installed rails and sliding way do not hinter the sailing and anchoring of other ships.

(6) At a place where waves going up the slip way in stormy weather may flow into the space on the back, scouring prevention work is required on the back of the slip way.

8.4 Dimensions of Each Part

8.4.1 Requirements for Serviceability

(1) Water depth and length

(a) Length

In setting the length of slip ways for the performance verification, the dimensions of the design ships shall be appropriately considered.

(b) Water depth

In setting the water depth of slip ways for the performance verification of boat lift yards, the dimensions of the design ships and the envisaged conditions of use of the facility shall be appropriately considered.

(2) Crown height

In setting the crown height of slip ways for the performance verification of boat lift yards, the dimensions of the design ships and the envisaged conditions of use of the facility shall be appropriately considered to enable the slip ways to be safely and efficiently used.

(3) Ancillary facilities

In the performance verification of boat lift yards, appropriate consideration is required for ancillary facilities to enable the boat lift yards to be safely and efficiently used. The provisions of **Reference Ministerial Ordinance, Article 33** (Performance Required for Facilities Incidental to Mooring Facilities) shall be applied with modification as necessary to the performance required for ancillary facilities. The settings in **Reference Public Notice, Article 60** to **Article 74** shall be applied with modification as necessary to the performance criteria based on the type of an ancillary facility.

(4) Others

① **Extension of slip ways**

In the performance verification of boat lift yards having slip ways, the extension of a slip way shall be appropriately set considering the dimensions of the design ships and the envisaged conditions of the use of the facility such that it does not hinder the use by design ships.

② **Area of the space on the back of a slip way**

In the performance verification of boat lift yards having slip ways, the area of the space on the back of the slip way shall be appropriately set considering the dimensions of the design ships and the envisaged conditions of the use of the facility such that it does not hinder the use by design ships.

③ **Slope angle of the slip way**

In the performance verification of boat lift yards having slip ways, the slope angle of the slip way shall be appropriately set considering the dimensions and shape of the design ships, the ground conditions, the tidal range, and the envisaged conditions of the use of the facility in order to enable smooth retrieving and launching of ships.

④ **Area of the anchorage at the front**

In the performance verification of boat lift yards, the area of the anchorage at the front shall be appropriately set considering the dimensions of the design ships and the envisaged conditions of the use of the facility so that design ships can be safely retrieved and launched and such that it does not hinder the sailing of other ships.

8.4.2 Height of Each Part

(1) It is preferable that the crown height of the front wall of the slip way section be located at a level lower than the mean monthly lowest water level (L.W.L.) by the draft of the design ships. This requirement indicates that it is necessary to lift ships even at the low water of neaps. The draft of the ship should be the light draft for the case of repair, refuge, and wintertime storage and should be the full-load draft for the case of lifting small fishing boats filled with catches. For boat lift yards that are to be constructed in the areas where tidal ranges are small or for the boat lift yards that are to be used even at the low-water springs during high waves, it is possible to lower the crown height of the front wall further.

(2) The crown height of the ship storage yard can be determined by applying **Part III, Chapter 5, 2.1.1 Dimensions of Quaywalls**. However, when the ship storage area is located adjacent to a quaywall, the crown height of the ship storage area can be set equal to the crown height of the quaywall to facilitate ease of use. In cases where waves are high in the water area in front of the boat lift yard, consideration of the wave runup height is preferable.

(3) It is preferable not to change the gradient of the slip ways considering the convenience of retrieving and launching of ships.

(4) If providing a point at which the gradient changes on the slip ways is unavoidable, due to the deep depth of water or constraint of available ground area, it is preferable that the position of the point of gradient change be set considering the heights of the following:

① **When the slip way consists of two different surfaces**

Near M.S.L. - H.W.L.

② **When the slip way consists of three different surfaces**

First point : Near L.W.L.

Second point: Near H.W.L.

(5) When waves are high in the water area in front of the boat lift yard, the crown height of the ship storage yard shall preferably be determined considering the wave runup height. In addition, in determination of the crown height of the ship storage yard, influence of abnormal tide levels and ground subsidence shall be appropriately considered.

(6) The submerged section of a slip way is usually covered with concrete blocks as the structure in many cases. In such a case, the height of the boundary between the cast-in-place concrete and concrete block section can be near M.S.L.

(7) For the relationship between the wave height at the front and wave runup height, **Part II, Chapter 2, 4 Waves** can be referred to.

8.4.3 Extension of Boat Lift Yards and Areas on the Back

(1) The extension of the slip way of a boat lift yard shall be calculated from the use of the yard and shall desirably be determined considering the facility layout of the entire port.

(2) The area of space on the back is to place retrieved ships, and usually, it is a flat space area. However, when the crown height of the ship storage yard is high, a part at the top of the slope is sometimes included. As the length of this section of the slip way, it is desirable to add approximately 5 m to the total length of design ships.

(3) Retrieved ships shall desirably be arranged at intervals of approximately 2 m in the longitudinal direction and approximately 1 m in the horizontal direction.

8.4.4 Front Water Depth

(1) The depth of water in front of the slip way may be determined referring to the sum of the draft of the design ship and a margin of 0.5 m.

(2) The required water depth at the front varies depending on the crown height of the wall of the front wall portion and ship retrieving and launching procedures. Generally, when wire ropes are used to launch a ship slowly, the ship does not draw deeper than the maximum draft. When a ship is slid on the slope to launch it, it may draw deeper than the maximum draft, so attention is required.

8.4.5 Gradient of Slip way

(1) The gradient of the slip way shall be determined appropriately in consideration of the shape of the design ships, the characteristics of foundation, and the tidal range, so that the lifting and launching of ships can be performed smoothly.

(2) When the slip way is to be utilized by small ships, it is preferable to have a slope with a single gradient. Single-gradient slopes are frequently used in slip ways for human power-based ship lifting in shallow waters. For this type of slip way, a slope inclination of 1:6 to 1:12 may be used as a reference.

(3) When the water in front of the slip way is deep or the area of the construction site is limited, the slip way may be built with two or more gradients. When this is the case, a two-gradient slip way may be adopted when the crown height of the front wall is about -2.0 m, and a three-gradient slip way may be adopted when the crown height of the front wall is lower than -2.0 m. The following values may be used as reference gradients:

① **When the slip way consists of two different surfaces:**

Front slope: 1:6 to 1:8

Rear slope: 1:8 to 1:12

② **When the slip way consists of three different surfaces:**

Front slope: an inclination steeper than 1:6

Central slope: 1:6 to 1:8

Rear slope: 1:8 to 1:12

(4) The submerged section can be a steep slope. However, seaweed gets on the low-water section and that makes the slope slippery when a ship is retrieved, so a gentle slope is desirable. If a steep slope is unavoidably used, footholds or other similar materials shall desirably be installed on the sides of the slip way to prevent slippage.

(5) At a section at which the gradient changes, larger force is required to pull up a ship, so the gradient shall not desirably be changed much. If the gradient is unavoidably changed largely, the step shall be smoothened using a curve with the large radius of curvature.

(6) For docking and undocking of ships with the total tonnage of 30 tons or more, rails are often used. In such a case, the slip way often consists of two different surfaces or three different surfaces. The gradient of the rear slope is gentler than 1:12 in some cases.

8.4.6 Area of Front Basin

(1) The basin in front of a boat lift yard shall have an appropriate area that allows for efficient operation of ship retrieving and launching without damage to the ships and safe and efficient navigation of nearby ships.

(2) When the ship is launched to the sea by sliding over the slip way, the ship runs over a certain distance after touching the water with the speed gained during the launch. This distance is more than about 5 times of the ship's length overall, although it varies depending on the gradient of slope, slip way friction, and launching distance. However, because the ship attains its maneuverability after moving a distance about 4 to 6 times of its length, it is sufficient to secure a distance about 5 times of the ship's length overall from the waterfront line of the slip way to the other end of the basin. When strong tidal currents exist, it is preferable to add an appropriate margin.

(3) When the ship is launched to the sea gently by wire ropes, a distance of about 3 times of the ship's length overall will suffice to secure the required width of water area.

8.5 Walls and Pavements of Front Wall Portions

8.5.1 Walls of Front Wall Portions

(1) The structure of walls of the front wall portions of boat lift yards shall be appropriately set based on the main dimensions of ships that use the yards, the crown height of the walls of the front wall portions, the methods to pull up such ships, and other factors.

(2) For the performance verification of the walls of the front wall portions of boat lift yards, performance verification for similar structures can be referred to, based on the structure of the walls.

(3) There are several types of wall structures for front wall portions such as blocks, cast-in-place concrete, and sheet piles.

(4) When the crown height of the wall of a front wall portion is high, the weight of a ship may concentratedly apply to the wall of the front wall portion, so particular attention is required for the bearing power of the foundation.

(5) If the foundation is scoured by incoming waves from the front wall portion or the return flow of waves that went up the slope, which may break not only the wall of the front wall portion but also the slope. Therefore, at a place with great waves, sufficient foot protection and covering are desired for the wall of the front wall portion.

8.5.2 Pavements

(1) Most pavements are cement concrete. The thickness of concrete slabs is 20 to 35 cm for cast-in-place concrete, and the intervals between joints are approximately 5 to 10 m. For precast concrete blocks, the size is 2 × 2 m and the thickness is approximately 30 cm in many cases. However, when the wave height is high or in recovery from disaster, **Fig. 8.1.2** can be used to determine the thickness of blocks.[2)] The roadbeds shall desirably be sufficiently compacted to prevent differential settlement. Usually, the thickness is approximately 30 cm.

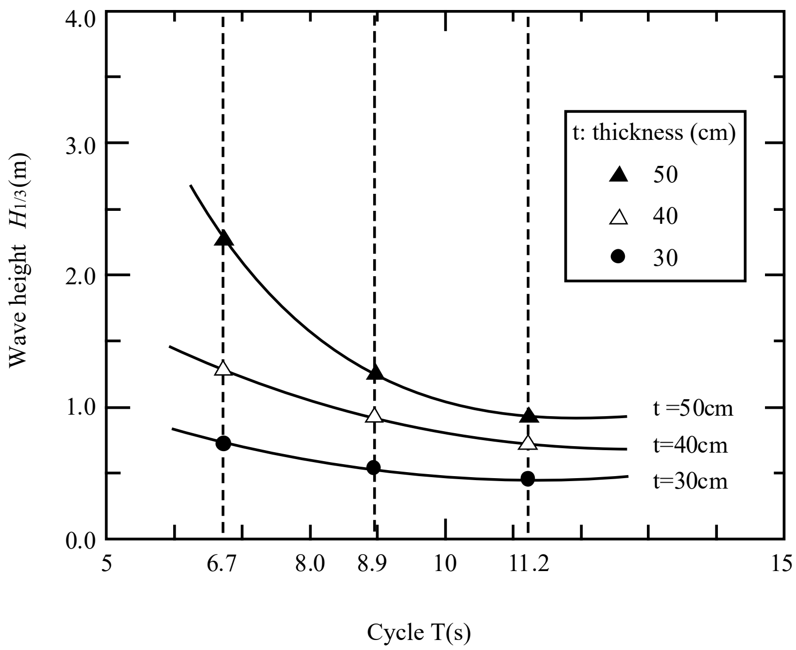


Fig. 8.1.2 Required Thickness of Pavements based on the Cycle and Wave Height t[2)]

(2) The raw materials of the foundation may be drawn out from the joints of the pavement, which may cause differential settlement and may break the pavement. Therefore, pavements shall be constructed with less joints as the structure. The materials are often drawn out at a slope where waves go up, in particular, so cast-in-place pavements shall desirably be constructed as much as possible.

(3) Cast-in-place pavements are difficult to construct under water, so precast concrete blocks are laid out. Such a structure shall desirably prevent sand from flowing out by using half lap joints as joints and by putting asphalt in between the joints. In addition, sand invasion prevention plates shall be used for joints between cast-in-place pavements and precast concrete blocks.

(4) The joints of pavements that may be easily broken by waves are connected with tie bars, or intermediate retaining walls are installed and the edges are cut in some cases. In addition, intermediate retaining walls are often installed at the boundaries between the concrete block section and concrete section. At the ends of pavements, retaining walls shall desirably be installed because if the site on the back is scoured, the foundation of the pavement may flow out.

(5) A point at which the gradient changes easily breaks due to concentration of the weight of a ship or other factors, so intermediate retaining walls, piles, and other similar materials are sometimes used for reinforcement.

(6) If there is a risk of a slope settling because the ground is soft, attention shall be paid to prevent differential settlement through rolling compaction of the foundation, soil improvement, and other measures.

(7) In performance verification of pavements, the necessary stability shall be secured against actions from ships and other elements.

[References]

1) Fisheries Agency: Reference for the Design of Fisheries Facilities, 2015 (in Japanese), http://www.jfa.maff.go.jp/j/gyoko_gyozyo/g_thema/sub52.html

2) Kimura., K.: Design method of plastering blocks for slip way, Journal of Public Works Research Institute (PWRI), Hokkaido Regional Development Bureau, No. 369、1984

9 Ancillary Equipment of Mooring Facilities

9.1 Mooring Posts, Bollards and Mooring Rings

[Public Notice] (Performance Criteria of Mooring Posts, Bollards and Mooring Rings)

Article 60

The performance criteria of mooring posts, bollards and mooring rings shall be as prescribed respectively in the following items:

(1) The mooring posts, bollards and mooring rings shall be located appropriately so as to enable the safe and smooth mooring of ships and cargo handling operations in consideration of the positions of the mooring ropes for the ships using the mooring facilities.

(2) The risk of impairing the integrity of the members of mooring posts, bollards and mooring rings and losing their structural stability shall be equal to or less than the threshold level under the variable situation in which the dominating action is the traction by ships.

[Interpretation]

11. Mooring Facilities

(13) Performance Criteria of Mooring Posts, Bollards and Mooring Rings (Article 33 of the Ministerial Ordinance, and the interpretation related to Article 60 of the Public Notice**)**

Serviceability shall be the performance requirement of mooring posts, bollards and mooring rings under the variable situation in which the dominating action is traction by ships. The performance verification items and standard indexes to determine the limit values for such actions shall be as shown in **Attached Table 11-30**. For the performance verification of the members of mooring posts, bollards and mooring rings, the standard indexes for determining the limit values for the soundness shall be appropriately set according to the kind of materials.

Attached Table 11-30 Performance Verification Items and Standard Indexes for Determination of Limit Values of Mooring Posts, Bollards and Mooring Rings in Each Design State (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
33	1	2	60	–	2	Serviceability	Variable	Traction by ships	–	Soundness of members of mooring posts, bollards and mooring rings	–
								Traction by ships	Self-weight	Stability of structures of mooring posts, bollards and mooring rings (sliding of superstructures)	Action-to-resistance ratio for sliding
										Stability of structures of mooring posts, bollards and mooring rings (overturning of superstructures)	Action-to-resistance ratio for overturning

9.1.1 Fundamentals of Performance Verification

(1) Mooring posts, bollards and mooring rings shall have structures that are safe against tractive forces by ships acting on them. They shall also have such sizes and shapes that they will not interfere with operations for mooring and unmooring ships.[1)]

(2) For the performance verification of mooring posts, bollards and mooring rings, **References2)** and **3)** can be used.

9.1.2 Layout of Mooring Posts, Bollards and Mooring Rings

(1) Mooring posts, bollards and mooring rings shall be located appropriately so as to enable the safe and smooth mooring of ships and cargo handling operations in consideration of the positions of the mooring ropes for the ships using the mooring facilities concerned.

(2) In general, bollards are installed close to the berth face line for mooring ships under ordinary conditions or for berthing and unberthing ships, whereas mooring posts are installed around both ends of the berth and as far away from the berth face line as possible for mooring ships in storm conditions.

(3) For positions and names of mooring ropes for a moored ship, **Part III, Chapter 5**, **2.1.1 Dimensions of Wharves** shall be referred to.

(4) It is common for posts to be installed close to the berth face line if they are intended to be used for mooring ships under ordinary conditions or for berthing and unberthing of ships. Otherwise, mooring ropes will traverse an apron and interfere with operations for loading and unloading ships. In principle, bollards shall be selected for use as posts in such positions, because mooring ropes may be stretched at a large angle of elevation. However, for mooring facilities to be used by small ships, mooring posts are often selected for use as posts installed close to the berth face line, because the crowns of small-ship mooring facilities are at almost the same level as decks of ships and thus mooring ropes are unlikely to be stretched at an extremely large angle of elevation. For the distance intervals between bollards and the minimum number of bollards installed per berth, the values given in **Table 9.1.1**. can be used as a reference[4)] When the gross tonnages of design ships are 200,000 tons or more, the distance intervals between bollards and the number of bollards installed may be determined by reference to the placement of bollards for ships with capacities of 150,000 tons or more and less than 200,000 tons.

Table 9.1.1 Placement of Bollards

Gross tonnage of design ship (ton)		Maximum interval between bollards (m)	Minimum number of bollards installed per berth (unit)
	Less than 2,000	10-15	4
2,000 or more	and less than 5,000	20	6
5,000 or more	and less than 20,000	25	6
20,000 or more	and less than 50,000	35	8
50,000 or more	and less than 100,000	45	8
100,000 or more	and less than 150,000	45	10
150,000 or more	and less than 200,000	45	12

(5) At small-ship mooring facilities where mooring ropes are unlikely to be stretched at a large angle of elevation, there are cases where mooring posts are installed at intervals of 10 to 20 m, instead of installing bollards. It is common to moor a small ship by stretching mooring ropes from the bow and stern of the ship to a quaywall, so mooring rings and other mooring devices that have strength equivalent to that of bollards may be installed at intervals of 5 to 10 m, in place of bollards. Such mooring devices can be installed on either the top or the side of mooring facilities. When they are installed on the side, they shall be positioned at an appropriate height with due consideration of the tide level.

(6) When posts are installed only in positions close to the berth face line at mooring facilities where ships are moored even in a storm, they cannot effectively work against the forces acting on a ship from the side. Therefore, posts shall also be installed away from the berth face line so that mooring ropes can be stretched as perpendicularly to the center line of a ship as possible and will not interfere with traffic for operations for loading and unloading ships. Mooring posts can be selected for use as posts in such positions, because mooring ropes hung on them are unlikely

to be stretched at a large angle of elevation. Mooring posts are positioned according to the conditions of use by ships. In general, they are installed in such positions as to allow mooring ropes to be stretched as perpendicularly to the center line of a ship as possible and, thus, effectively work against the forces acting on the body of a ship from the side. It is common to install two mooring posts on one berth. Bow ropes and stern ropes of a ship are stretched so that each rope makes a small angle with the center line of the ship to control the movement of the ship in the direction of its center line. It is preferable to install bollards so that the angle is kept larger than 25° to 30°. **Fig. 9.1.1** shows typical examples of arrangements of mooring posts. There are some cases where it was decided not to install mooring posts at a port where strong wind blows only in a certain direction and never come from the berth side (the land side) or at a port where no ship is moored in strong wind.

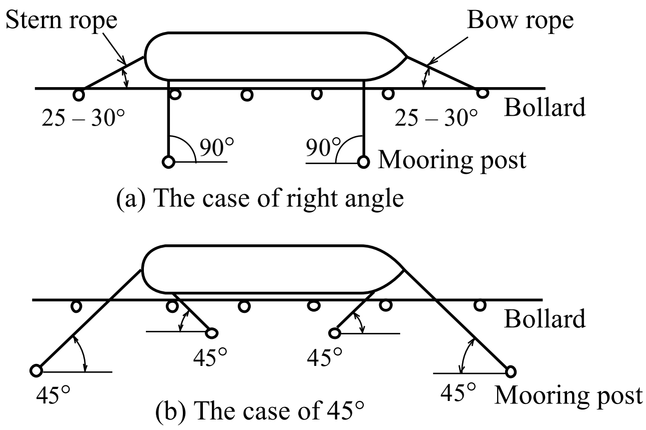


Fig. 9.1.1 Angles of Mooring Ropes against Quaywall Face Line

(7) There are cases where the mooring ropes stretched from two adjacently moored ships are hung on one mooring post or bollard installed at the junction of two berths. Since the ropes are stretched from different directions and their resultant force is not much larger than the tractive force from either of the ships, there is no need to install a larger-sized mooring post or bollard at the junction of two berths. However, to ensure safe release of mooring ropes for unberthing ships, it is preferable to install two bollards. At large mooring facilities, there are cases where bow ropes and stern ropes are stretched from both sides of a ship and, thus, there are four or more ropes from each of the bow and the stern. In view of this, it is preferable to install pairs of bollards at points to hang these ropes.

(8) From the aspect of safety in mooring and unmooring ships, bollards should be installed as close to the berth face line as possible and kept at a certain distance from curbing.[1)] It should be noted that, when bollards are positioned on the land side of curbing, mooring ropes hung on the bollards are likely to interfere with the curbing and thereby get damaged and/or bounce up. It is preferable that each mooring post or bollard be surrounded by a flat area with no obstacles or differences in level so that mooring ropes will not get damaged when rubbing against the mooring post or bollard and can be hung on and released from it smoothly.

9.1.3 Actions

(1) The tractive forces by design ships shall be appropriately calculated considering the berthing and mooring conditions of ships. For setting the tractive forces by design ships, **Part II, Chapter 8, 2.4 Actions Caused by Traction of Ships** shall be referred to.

(2) The verification of stability of superstructures against sliding and overturning shall be performed in terms of tractive forces from the most dangerous traction angles. The traction angles of the most dangerous tractive forces can be calculated using **equations (9.1.1)** and **(9.1.2)**. The envisaged ranges of traction angles depending on conditions such as the dimensions of design ships and tide levels need to be given due consideration. **Fig. 9.1.2** illustrates actions caused by the tractive force in sliding or overturning of a superstructure. The subscript k indicates the characteristic value in the following equations.

① The case of sliding (Refer to **Fig. 9.1.2 (a)**.)

$$\theta_k = \tan^{-1}\left(\frac{f}{m}\right) \quad \textbf{(9.1.1)}$$

where

θ : traction angle (rad)

f : friction coefficient

m : adjustment factor; m = 1.0 shall be used.

② The case of overturning (Refer to **Fig. 9.1.2 (b)**.)

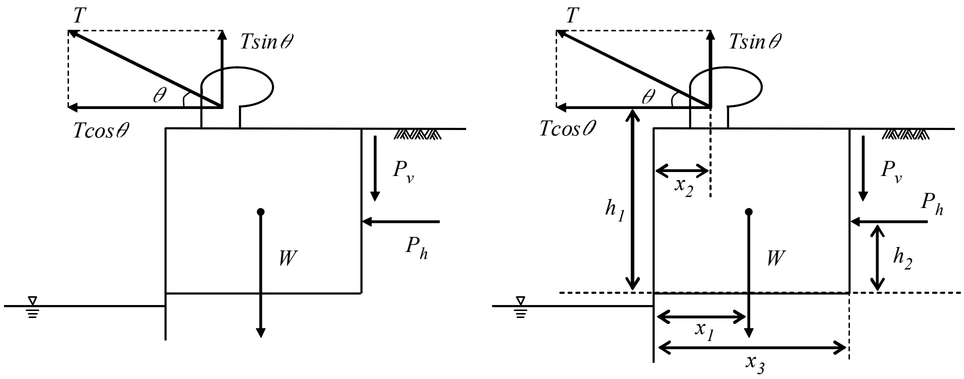
$$\theta_k = \tan^{-1}\left(\frac{x_2}{h_1}\right) \quad \textbf{(9.1.2)}$$

where

θ : traction angle (rad)

x_2 : distance from face line of quaywall to tractive force acting point (m)

h_1 : distance from bottom of superstructure to tractive force acting point (m)



(a) Sliding (b) Overturing

Fig. 9.1.2 Actions Caused by Tractive Forces

9.1.4 Performance Verification

(1) The performance of superstructures on which mooring posts, bollards and mooring rings are installed shall be verified in terms of the stability against sliding and overturning. When a superstructure is constructed behind a quaywall, not on the quaywall face line, the stability of the superstructure against sliding and overturning shall be verified by appropriately considering the forces of active earth pressure and passive earth pressure as components of the resultant vertical earth pressure and the resultant horizontal earth pressure.

① **Verification of stability against sliding**

The following equation can be used for verifying the stability against sliding of the superstructures on which mooring posts, bollards and mooring rings are installed.

$$f(W + P_v - T\sin\theta) \geq m(T\cos\theta + P_h) \quad \textbf{(9.1.3)}$$

where

f : friction coefficient

W : weight of superstructure (kN/m)

θ : traction angle (rad)

T : tractive force (kN/m)

P_v : resultant vertical earth pressure acting on superstructure (kN/m)

P_h : resultant horizontal earth pressure acting on superstructure (kN/m)

m : adjustment factor; m = 1.0 shall be used

② **Verification of stability against overturning**

The following equation can be used for verifying the stability against overturning of the superstructures on which mooring posts, bollards and mooring rings are installed.

$$x_1 W + x_3 P_v \geq m(h_1 T \cos\theta + x_2 T \sin\theta + h_2 P_h) \qquad \textbf{(9.1.4)}$$

where

W : weight of superstructure (kN/m)

θ : traction angle (rad)

T : tractive force (kN/m)

P_v : resultant vertical earth pressure acting on superstructure (kN/m)

P_h : resultant horizontal earth pressure acting on superstructure (kN/m)

x_1 : distance from face line of quaywall to superstructure weight acting point (m)

x_2 : distance from face line of quaywall to tractive force acting point (m)

x_3 : distance from face line of quaywall to acting point of resultant vertical earth pressure (m)

h_1 : distance from bottom of superstructure to tractive force acting point (m)

h_2 : distance from bottom of superstructure to acting point of resultant horizontal earth pressure (m)

m : adjustment factor; m = 1.1 shall be used.

(2) When a superstructure equipped with mooring posts, bollards and mooring rings is constructed on a sheet pile quaywall, a mooring dolphin or other mooring facilities, it is connected to the heads of steel sheet piles, steel pipe piles or the like. Therefore, the verification of its stability shall be performed by using an appropriate method with due consideration of structural characteristics of the mooring facilities.

9.2 Fender Systems

[Public Notice] (Performance Criteria of Fender Systems)

Article 61

The performance criteria of fender systems shall be as prescribed respectively in the following items:

(1) The fender systems shall be located appropriately and provided with the necessary dimensions so as to enable the safe and smooth berthing and mooring of ships in consideration of the environmental conditions to which the systems are subjected, the berthing and mooring conditions of ships, and the structural type of mooring facilities.

(2) The risk that the berthing energy of ships may exceed the absorbed energy of the fender system under the variable situation, in which the dominating action is ship berthing, shall be equal to or less than the threshold level.

[Interpretation]

11. Mooring Facilities

(14) Performance Criteria of Fender Systems (Article 33 of the Ministerial Ordinance and the interpretation related to Article 61 of the Public Notice)

Serviceability shall be the performance requirement of fender systems under the variable situation in which the dominating action is ship berthing. The performance verification items and standard indexes to determine the limit values for such actions shall be as shown in **Attached Table 11-31**.

Attached Table 11-31 Performance Verification Items and Standard Indexes for Determination of Limit Values of Fender Systems in Each Design State

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for Determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
33	1	2	61	-	2	Serviceability	Variable	Ship berthing	–	Berthing energy against fender system	Absorbed energy of fender system

9.2.1 Fundamentals of Performance Verification

(1) To verify the performance of a fender system, its installation position and dimensions shall be appropriately determined by considering the environmental conditions to which the system is subjected, the berthing and mooring conditions of ships, and the structural type of the mooring facility to ensure that the ships are berthed and moored safely and smoothly.

(2) When a ship is berthed to a mooring facility or when a moored ship motions owing to the actions of wind and waves, a berthing force and impact forces are generated between the ship and the mooring facility. To prevent damage to the ship's hull and the mooring facility due to the generated forces, fender systems shall be installed on the mooring facility in principle. However, fender systems are not always required in cases where small ships, certain types of ferries, and other ships provided with fender equipment, such as ship fenders or tires, are maneuvered very carefully during berthing by considering the fender equipment's energy absorption capacity; thus, the berthing force is relatively small.

(3) Rubber fenders and pneumatic fenders are commonly selected for use in fender systems. Other types of fenders, such as foam type, hydraulic type, gravity type, pile type, and timber type, are also used.[5)]

(4) The procedure for performance verification of rubber fenders, pneumatic fenders, and pile type fenders is shown in **Fig. 9.2.1** as an example.

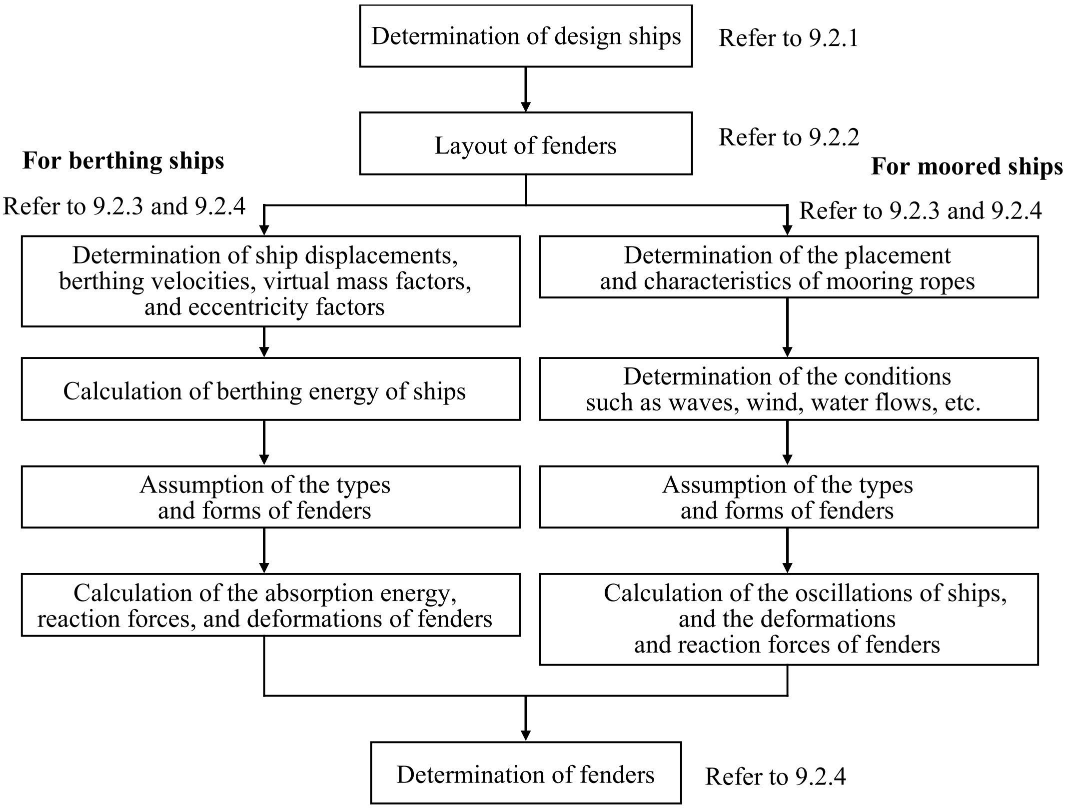


Fig. 9.2.1 Example of Performance Verification Procedure for Fenders

(5) The performance of fenders significantly affects the construction costs of mooring facilities, the maintenance costs after construction, and the efficiency of loading and unloading ships. Therefore, it is preferable to consider not only the construction costs of the fenders but also the comprehensive costs of all the aforementioned factors when selecting fenders. In general, the structures of mooring facilities, such as piled piers and dolphins, are significantly affected by the reaction forces of fenders; thus, the total construction costs of such mooring facilities may be reduced by selecting high-performance fenders even if they are expensive. By contrast, the structures of gravity-type and sheet-pile-type mooring facilities are not affected by the fender reaction forces caused by ships; thus, the performance of fenders does not affect the construction costs of those mooring facilities. However, by considering the maintenance costs after the completion of construction, the selection of easy-to-maintain fenders may result in cost reduction in the long run even if their initial cost is high. There are also cases in which high-performance fenders should be selected to allow the berthing of ships even under relatively severe oceanographic and meteorological conditions and to reduce the motions of the moored ships, thereby improving the efficiency of loading and unloading ships.

9.2.2 Layout of Fenders[5) 6)]

(1) Fenders shall be appropriately placed so that the ships have no direct contact with the mooring facilities before the fenders absorb a certain amount of berthing energy.

(2) Fenders are normally placed at 5 to 20-m intervals. When a ship berths, a point near the bow or stern initially contacts the mooring facility. Since the ship has a curved surface on the side facing the mooring facility, the fenders placed at excessively long intervals cause direct contact between a part of the ship's hull and a part of the mooring facility, where no fender is placed, before the fenders absorb a sufficient amount of berthing energy. Intervals of approximately 5 m normally cause no problem. However, when the intervals are 10 m or more and when a part of the ship's hull may directly contact a part of the mooring facility where no fender is placed, it is preferable to construct concrete superstructures so that the parts where the fenders are placed project from other parts by 0.2 to 0.5 m. Fenders should be placed at intervals of approximately 1/5 to 1/6 of the length of the parallel side of design ships.[7)]

(3) In case of large mooring facilities to be berthed by small ships, where the fenders for large ships are placed at long intervals and the fenders for small ships are placed in between them, the front surfaces of the fenders for small ships shall be set back from those for large ships to some extent. If the front surfaces of the fenders for small ships are not adequately set back, large ships may contact the small ship fenders that have small energy absorption capacity while being berthed, causing a significant increase in the reaction forces of the small ship fenders.

(4) In majority of the cases, timber fenders are placed continuously on the front surface of a mooring facility. There are also cases in which timber fenders are concentrated at intervals of 8 to 13 m.

(5) Particular attention shall be paid to the layout of the fenders for piled piers and other mooring facilities, where the dominating action is the berthing forces of ships.

(6) The installation heights of fenders shall be carefully determined by considering the design ships, which may include cement tankers and other ships with very low gunwale, as well as ferries and other ships with very high gunwale.

(7) When a small ship is berthed at a mooring facility located in a place, where a large height difference can be observed between high and low tides or where there are large waves, the side of the ship may come into direct contact with the mooring facility with no cushioning by fenders; further, the gunwale of the ship may get caught by protruding fenders. Thus, it is necessary to carefully determine the installation heights of the fenders. To prevent these problems, the fenders may be placed horizontally in two lines, or vertical fenders may be placed.

(8) For mooring facilities used by container ships and car carriers, especially ships with large flare, it is preferable to take measures to prevent the ship's side from coming into direct contact with mooring posts, bollards, container cranes, or other cargo handling equipment.

(9) From the aspect of safety while mooring and unmooring ships, mooring ropes may interfere with the top or bottom of a fender, especially a fender with a fender board.[1)] To prevent such interference, it is necessary to take measures for the top of a fender board; for example, selecting the structure that has no hangers or other protruding parts or providing a fender board with chains to prevent it from catching mooring ropes. It is also necessary to take measures for the bottom of a fender board; for example, providing it with steel members that prevent mooring ropes from getting caught underneath the fender board or adjusting the shape and/or installation height of the fender board so that its bottom will not be exposed above the sea surface at low tide. It is preferable to determine the positions of the fenders on the face line of a quaywall relative to the mooring posts or bollards by considering the actual operations for mooring and unmooring ships.

9.2.3 Actions

(1) For calculating the berthing energy of ships, **Part II**, **Chapter 8**, **2.2 Actions Caused by Ship Berthing** shall be referred to.

(2) To calculate the berthing force, it is common to initially draw a curve of load versus energy absorbed by a mooring facility and a curve of load versus energy absorbed by the whole part of one fender, and then to obtain a curve of load versus absorbed energy that shows the sum of the absorbed energy E_{f1} caused by the fender deformation and the absorbed energy E_{f2} caused by the mooring facility deformation, as shown in **Fig. 9.2.2**. The berthing force P for a given berthing energy E_f can be determined from the obtained curve. For gravity-type mooring facilities or other rigid mooring facilities, it shall be assumed that no energy is absorbed by the deformation of the main body of the mooring facility in general.

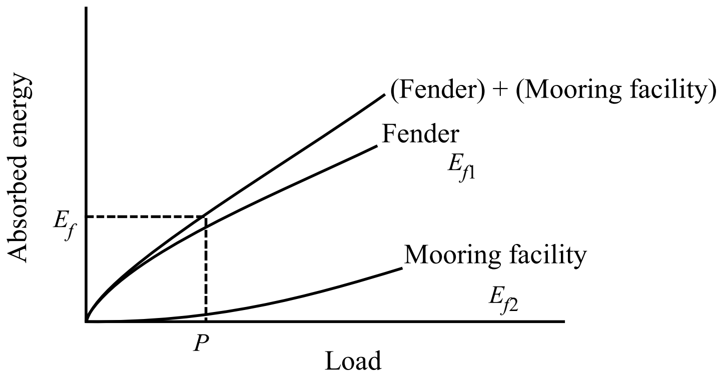


Fig. 9.2.2 Relation between Load and Absorbed Energy

(3) At mooring facilities exposed to wave actions, ships oscillate in both the horizontal and vertical directions owing to waves. These motions may cause excessive shear deformation in fenders in addition to the normal compressive deformation, and there were some cases in which fenders broke because of these deformations. There were also some cases in which fenders broke because of rough berthing of small ships at large mooring facilities available to both large and small ships. Since the coefficient of friction between a dry rubber material and an iron material is approximately 0.3 to 0.4, the shearing force, assumed as the friction force, is estimated to be approximately 30 to 40% of the reaction force of fenders at mooring facilities similar to those mentioned above. There was a case in which portable pneumatic fenders were selected for a port exposed to strong swells so that the fenders would not be damaged by the shearing force acting on them. It must be noted that some of the constant reaction type fenders exhibit the characteristic that the compression reaction force decreases when the shearing force acts on them.

(4) The fenders shall be equipped with a fender board or the like, as necessary, to reduce the load (surface pressure) per unit area to prevent the berthing force and other forces from acting on ships as a concentrated load. A synthetic resin plate or the like may be attached on the front surface of a fender board to reduce the shearing force acting on the fender.

9.2.4 Performance Verification

(1) An appropriate type of fenders shall be selected by considering the following items:

① Structural characteristics of the mooring facilities and the ships using them.

② For the mooring facilities exposed to wave actions, the motions of the moored ships and the ship berthing conditions such as the berthing angles.

③ Effects of the reaction forces of the fenders generated during ship berthing on the structures of the mooring facilities.

④ Variation ranges of the physical characteristic values of fenders due to manufacturing variation, dynamic characteristics, thermal characteristics, and other factors.

(2) Mooring facilities, such as the gravity-type mooring facilities, sheet-pile-type mooring facilities, and mooring facilities with relieving platforms, exhibit sufficient resistance against normal berthing forces. By contrast, piled piers, dolphins, detached piers, and other mooring facilities with a flexible structure, especially mooring facilities constructed on vertical piles, exhibit relatively small resistance against horizontal forces; therefore, the berthing forces must be lower than the tolerable load level. For the performance verification of the resistance of the piled piers, dolphins, and detached piers against berthing of ships, **Part III, Chapter 5, 5 Piled Piers of this Chapter** shall be referred to.

(3) The berthing energy is absorbed by the deformation of a ship's hull and the deformation of a mooring facility. However, the small energy absorbed by the deformation of the ship's hull is commonly disregarded.

① The deformation of a ship's hull can be classified as local deformation or whole deformation.

(a) Local deformation (Local deformation at the shell and ribs of the hull)

(b) Whole deformation (Strength of the entire hull against side bend)

② The deformation of a mooring facility can be classified as the deformation of the main body of the mooring facility or the deformation of one or more fenders.

(a) Deformation of the main body of the mooring facility (Deformation of a piled pier, dolphin, or other mooring facility due to ship berthing)

(b) Deformation of one or more fenders (Deformation of fenders due to ship berthing)

(4) The energy absorption by the deformation of a mooring facility can be given as follows:

① It shall be assumed that there is no energy absorption by the deformation of the main bodies of such rigid mooring facilities as gravity-type mooring facilities, sheet-pile-type mooring facilities, mooring facilities with relieving platforms, and cellular-bulkhead mooring facilities in general.

② Piled piers, dolphins, detached piers, and similar mooring facilities are classified into two types: with a rigid structure and with a flexible structure. There is no energy absorption by the deformation of the former type facilities. However, energy absorption can be observed by the deformation of the latter type facilities because of their flexibility, and the energy absorption can be generally given by **equation (9.2.1)**.

$$E_1 = \int_0^{Y_1} g(y_1)dy_1 \tag{9.2.1}$$

where

E_1 : energy absorbed by the deformation of the main body of the mooring facility (kJ)

Y_1 : maximum displacement of the main body of the mooring facility (m)

y_1 : displacement of the main body of the mooring facility (m)

$g(y_1)$: characteristics of the reaction force caused by the deformation of the main body of the mooring facility (kN)

Mooring facilities having a flexible structure are normally manufactured using steel materials. Since their performance required for the actions caused by the berthing forces of ships is serviceability and the responses are within an elastic limit, there is a linear relation between the deflection and reaction forces of such mooring facilities. When a mooring facility and its fenders completely absorb the berthing energy of a ship, the energy absorbed by the mooring facility can be expressed by **equation (9.2.2)**, where C denotes the spring constant of the mooring facility.

$$E_1 = \frac{1}{2}CY_1^2 \tag{9.2.2}$$

The same shall apply to the energy absorbed by the pile-type fenders.

③ The single pile structure (SPS) is a type of structure expected to absorb the berthing energy through the deformation of the piles made from high strength steel. In the performance verification of the berthing dolphins that use SPS, it is necessary to evaluate the amount of energy absorption by considering the residual deformation of the piles due to repeated berthing. As shown in **Fig. 9.2.3**, the amount of energy that can be absorbed by piles during ship berthing can be calculated from the displacement obtained by subtracting the residual displacement from the loading point displacement.[8)]

The loading point displacement with the considered residual displacement can be calculated from **equation (9.2.3)**.

$$y_{top} = A_1 y_0 + A_2 i_0 h + \frac{Ph^3}{3EI} \tag{9.2.3}$$

where

y_{top} : loading point displacement (m)

y_0 : pile displacement at sea bottom at the time of initial loading (m)

i_0 : pile deflection angle at sea bottom at the time of initial loading (rad)

P : horizontal load (kN)

h : height of loading point (m)

EI : flexural rigidity of the pile (kNm^2)

A_1, A_2 : coefficients of influence of repeated loading

The time of initial loading refers to the situation in which the pile is exposed to the largest ever load.

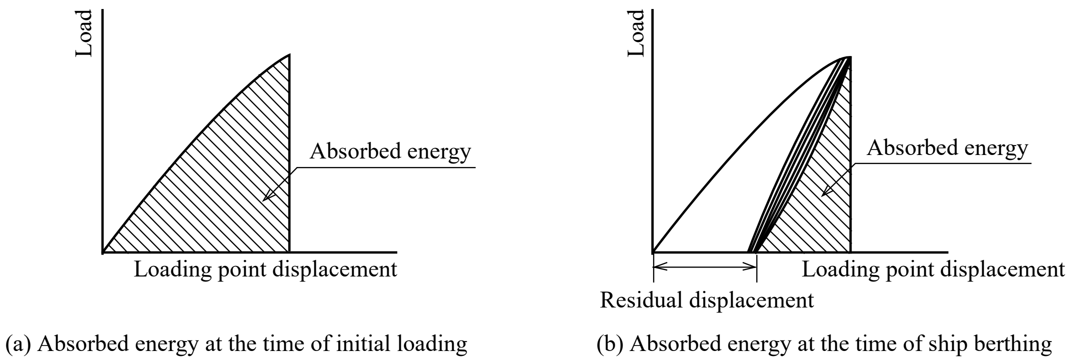


Fig. 9.2.3 Energy Absorbed by Deformation of Piles

Table 9.2.1 presents the values of the coefficients of influence of repeated loading that were obtained from the results of the in-situ full-scale loading tests[9] and the model tests.[10]

Table 9.2.1 Values of the Coefficients of Influence of Repeated Loading [8]

	For obtaining the maximum displacement	For obtaining the energy absorbed by the deformation of piles	For obtaining the residual displacement
A_1	1.4	0.4	0.8
A_2	1.2	0.6	0.5

(5) For mooring facilities with a rigid structure, where there is no energy absorption by the deformation of their main bodies, the energy absorbed by a fender can be calculated using the following equation:

$$E_s = \phi E_{cat} \geq E_f \tag{9.2.4}$$

where

E_s : energy absorbed by the fender (kJ)

ϕ : manufacturing error (tolerance) of the fender

E_{cat} : specified value of the energy absorbed by the fender (kJ)

E_f : berthing energy of the ship (kJ)

The characteristic value of the berthing energy of a ship can be expressed by **equation (2.2.1)** given in **Part II, Chapter 8, 2.2 Actions Caused by Ship Berthing**.

(6) There are various types of rubber fenders, including V-shaped, circular hollow, and rectangular hollow types. They differ from each other in terms of the relation between the reaction force and deformation as well as the energy absorption rate. Fender manufacturers' catalogs shall be referred to regarding the graphs of the amount of energy absorption versus the deformation and those of the reaction force versus the deformation for each type of fender.

Constant reaction type fenders, such as V-shaped fenders, are characterized by the low reaction forces and high energy absorption rates. However, it must be kept in mind that the total reaction force of such fenders may become large when a ship simultaneously comes in contact with two to three fenders. This is because the reaction force of each fender increases almost to the maximum level when it contains absorbed energy equivalent to one-third of its design capacity.

(7) The factors that cause variations in the characteristics of rubber fenders include the manufacturing variation, deterioration over time, dynamic characteristics (i.e., velocity-dependent characteristics), creep characteristics, repetition characteristics (i.e., compression frequency-dependent characteristics), oblique compression characteristics, and thermal characteristics. For fenders used for mooring floating structures, these factors are important in evaluating their safety as mooring equipment. For fenders used for mooring ships, it is also appropriate to verify the performance of the fenders by considering the factors, including the manufacturing variation, dynamic characteristics (i.e., velocity-dependent characteristics), oblique compression characteristics, and thermal characteristics. For example, when the manufacturing variation (tolerance) of a fender is ±10%, it is preferable to decrease the characteristic (performance) values presented in its catalog by 10% for calculating the amount of energy absorption and to increase the values by 10% for evaluating the reaction force of the fender that acts on the mooring facility. With regard to the dynamic characteristics, it is preferable to confirm that the reaction force of a fender at the time of ship berthing will not exceed the standard capacity shown in the catalog issued by its manufacturer by considering the berthing velocity of the ships. It should also be borne in mind that the fender reaction force becomes higher in a low-temperature environment than that in the standard temperature environment.

(8) A working group of the World Association for Waterborne Transport Infrastructure (PIANC) has recommended to correct the amount of energy absorption and the reaction force in the standard environment using the velocity correction factor and the temperature correction factor when selecting a fender by considering the fact that its characteristics will vary depending on the ship berthing velocity, the ambient temperature, and other conditions of the actual environment in which the fender will be used. The working group has also published guidelines[11) 12)] for selecting a fender using these correction factors. The actual values of the velocity correction factor and the temperature correction factor should be checked with the fender manufacturers as they vary depending on the ship berthing velocity, the ambient temperature, and the kind of rubber used for the fender. It should also be borne in mind that the reaction force acting on a mooring facility during the berthing of a small ship at a high berthing velocity may be larger than that during the berthing of a large ship at a low berthing velocity.

(9) The berthing force of a ship may cause permanent deformation of the shell of the ship; hence, it is necessary to select a fender carefully.[13) 14)] It is preferable to attach fender boards on the front surfaces of fenders as necessary to reduce the loads on the ships. Since the damage to the shell of a ship is affected by not only the magnitude of berthing force but also the structural strength of the shell, it is preferable to increase the contact area of each fender so that it contacts two ribs of the ship at the same time. The guidelines for designing fenders[11) 12)] recommend that the maximum allowable surface pressure for each type of ship should be approximately 200 to 400 kN/m^2. For the effects of the reaction force of fenders on the shell structure, **References 13) 15)** and **16)** can be used.

(10) Fenders must also be safe against the shearing force generated by the friction that occurs in the direction of the face line of a mooring facility due to oblique berthing of the ships. This force can be normally calculated using the equation suggested by Vasco Costa[17)]. When a ship is berthing to a mooring facility at an angle of 6 to 14º with the face line of the mooring facility, this force becomes 10 to 25% of the berthing force of the ship.

(11) According to the simulation results of the motions of moored ships[7) 18)], the deformation of fenders owing to the motions of a moored ship can be larger than the deformation owing to the berthing force of the ship when the period of waves acting on the ship is long due to swells, when waves act on the side of the ship's hull perpendicularly, or when the value of E/δ_a, which is the ratio of the amount of energy absorption by a fender E to the allowable deformation δ_a, is large. Therefore, it is advisable to select a fender that has a small value of E/δ_a, i.e., a fender that has the largest value of allowable deformation δ_a among the fenders which are equivalent with respect to the amount of energy absorption E.

9.3 Skirt Guards

9.3.1 General

(1) The facility shall be equipped with appropriate skirt guards when there is a risk of a small ship getting into an empty space beneath the slab of a piled pier, dolphin, or other mooring facility.

(2) The majority of skirt guards have precast slabs or section steel members, which are placed in the form of a wall, comb, or grid in appropriate positions by considering the tidal range and other conditions.

9.4 Lighting Facilities

[Public Notice] (Performance Criteria of Lighting Facilities)

Article 62

The performance criteria for lighting facilities shall be such that appropriate lighting facilities are installed so as to enable the safe and smooth use of mooring facilities where cargo handling operations, berthing and unberthing of ships, and entry and exit of people occur in consideration of the usage conditions of the mooring facilities.

9.4.1 General

(1) Appropriate lighting facilities should be provided at mooring facilities and related facilities where cargo handling works such as loading, unloading and transfer, berthing/unberthing of ships, and use by passengers and others are performed at night in consideration of the use conditions of the concerned mooring facilities.

(2) The description here may be applied to the installation, improvement, and maintenance of lighting facilities at wharves where cargo handling, berthing and unberthing, passenger use, etc., are performed at night. The lighting facilities for other facilities shall comply with the descriptions here and the standards separately provided for respective facilities.

(3) Many lighting facilities are designed these days to highlight the night views of structures, parks, watersides, etc., in urban fringes and tourist sites to meet social needs for lighting and other facilities in port facilities. In these cases, not only illumination but also light colors and color-rendering property are needed to give people pleasure, familiarity, and peace of mind. On the contrary, given that lighting facilities have come into wide use, it is essential to consider the adverse effects of lighting on the surroundings and on energy savings. The performance verification of lighting facilities should fully take into account these demands. Properly examine lighting functions and individually take necessary measures suited to individual facilities in coastal areas where people interact, such as amenity-oriented revetments, marinas, parks, and promenades.

9.4.2 Performance Verification Items for Lighting Facilities

(1) In designing lighting facilities, the locations of lamp fittings shall be determined by appropriately selecting lighting methods, light sources, and lighting apparatuses in consideration of the following items according to the installation locations of the lighting facilities. Furthermore, lighting facilities that have possible influences on sea surfaces should be designed in a way that prevents its interference with ship navigation at sea.

① Standard intensity of illumination
② Distribution of illumination
③ Glare
④ Color and color-rendering property
⑤ Obstacle light and energy saving

9.4.3 Standard Intensity of Illumination

(1) General

① The standard intensity of illumination is an average horizontal-plane illumination and is defined as the minimum value for safely and effectively using the facilities concerned. The objective generally used in designing lighting facilities is illumination. Horizontal illumination is the illumination of a floor surface or a ground surface. The average horizontal illumination is the average value of that illumination.

② The illumination of lighting facilities should be properly determined on the basis of varieties and systems of work to enable the facilities concerned to be used safely and smoothly.

③ The International Commission on Illumination (CIE) has been examining the criteria of illumination and has published **GUIDE FOR LIGHTING EXTERIOR WORK AREAS.** The guide includes the recommendations for the regulation values of maintenance illumination and the uniformity ratios of illumination, glare, and color-rendering property.

④ The performance verification of lighting facilities can refer to the standard intensity of illumination described here because it has been determined by taking into consideration the following laws and regulations, actual situations of lighting facilities at domestic and international ports, and other reference materials. However, given that the standard intensity of illumination described here is the minimum value, it can be increased as needed.

(a) **Ordinance on Industrial Safety and Health** (Ordinance of the Ministry of Labor No. 32, September 30, 1972)

(b) **Order for Enforcement of the Parking Place Act** (Cabinet Order No. 340, December 13, 1957)

(c) **General Rules of Recommended Lighting Levels** (JIS Z 9110: 2010)

(d) **Lighting for Roads** (JIS Z 9111)

(e) **Lighting of Tunnels for Motorized Traffic** (JIS Z 9116)

(f) **Standards and Commentaries for the Installation of Road Lighting Facilities**[19)]

(g) **Lighting of Outdoor Work Places**[20)]

(h) **Lighting of Indoor Work Places** (JIS Z 9125: 2007)

(i) **Lighting of Outdoor Work Places** (JIS Z 9126: 2010)

(j) **Standard and Design Guide for Lighting of Indoor Work Places** (JCIE-002 2009)

(k) **Maintenance Factors and Maintenance Planning in Lighting Design, Third Edition** (JIEG-001 (2005))

(l) **Maintenance Factors and Maintenance Planning in Lighting Design, Third Edition, Enlarged Edition for LED Lighting** (JIEG-001 (2013))

(2) Standard intensity of illumination for outdoor lighting

① The values shown in **Table 9.4.1** may be used for the standard intensity of illumination of each type of outdoor facility.

(a) Mooring facilities for passengers, vehicles, and pleasure boats and general cargo and container berths

The lighting for aprons on these facilities is required to show relatively advanced visual information, such as the clothes and facial expressions of passengers and the types, shapes, and colors of vehicles or cargoes, and meet a certain level of comfort. Thus, the standard intensity of illumination is set at 50 lx with reference to the **General Rules of Recommended Lighting Levels (JIS Z 9110)** and actual measurements at existing lighting facilities (**Table 9.4.1**).

Table 9.4.1 Standard Intensity of Illumination for Outdoor Lighting

Facility			Standard intensity of illumination (lx)
Wharf	Apron	Mooring facilities for passengers, vehicles, and pleasure boats and general cargo and container berths	50
		Slipways for pleasure boats and aprons for handling dangerous goods using pipelines	30
		Aprons for simple work using pipelines and belt conveyors	20
	Yard	Container yards, general cargo storage, handling yards, and cargo transfer yards	20
	Path	Passenger gates and vehicle gates	75
		Passenger paths and vehicle paths	50
		Other paths	20
	Security	All facilities	1–5

Facility			Standard intensity of illumination (lx)
Road and Park	Road	Main roads	20
		Other roads	10
	Parking lots	For ferries	20
		Others	10
	Park and green space	Garden paths	3

(b) The slipways for pleasure boats and aprons for handling dangerous goods using pipelines

The standard intensity of illumination of the slipways for pleasure boats is set at 30 lx because these slipways are not required to show detailed visual information, such as shapes and colors. The standard intensity of illumination of the aprons for handling dangerous goods is also set at 30 lx in consideration of their safety.

(c) Aprons for simple work

The standard intensity of illumination of 20 lx is set for aprons where simple cargo handling work is executed using pipelines or belt conveyors.

③ **Container yards**

(a) In container yards, containers are transferred and stacked by straddle carriers and transfer cranes. In cargo sorting area, cargoes are transferred and loaded and unloaded from trucks by forklifts. The work procedures in container and cargo sorting areas are similar to those in inland truck yards. Thus, the standard intensity of illumination in the container and cargo handling yards is set at 20 lx with reference to the **General Rules of Recommended Righting Revels (JIS Z 9110)** and actual measurements at existing lighting facilities (**Table 9.4.1**).

(b) The standard intensity of illumination is not necessarily applied to all areas of the container yards. Container yards can be divided into zones in accordance with lighting objects, and different values of standard intensity of illumination can be set depending on the importance of work executed in respective zones, e.g., chassis travel paths and container transship points. Furthermore, in storage yards where containers are stacked, the standard intensity of illumination is not necessarily applied to the places between the stacked containers.[21) and 22)]

④ **Paths**

(a) Gates and paths for passengers and vehicles

Lighting facilities are required for movable bridges used by passengers and vehicles when they board passenger ships or ferries. Although vehicles have head lamps, the glare of head lamps may cause traffic controllers to make erroneous guidance, thereby leading to accidents such as falls into the sea. Thus, the standard intensity of illumination for paths and gates is set at 50 and 75 lx, respectively with reference to the **General rules of recommended lighting levels (JIS Z 9110)** and actual measurements at existing lighting facilities (**Table 9.4.1**).

(b) Other paths

The standard intensity of illumination for other paths, including pedestrian paths, for workers is set at 20 lx because they are at less risk of accidents than the paths for passengers and vehicles.

⑤ **Lighting for security control**

Facilities that are not used at night also require lighting for crime prevention and security control. Thus, the standard intensity of illumination for the security purpose is set at 1 to 5 lx.

⑥ **Roads**

The standard intensity of illumination for roads on and around wharves should be determined with consideration to the followings:

(a) Work conditions on wharves (necessity and frequency of night work)

(b) Traffic conditions (traffic volume, traveling speeds, and types of vehicles)

(c) Road conditions (types of alignments, structures, and pavement)

(d) Topographic conditions

(e) Economic effects.

Thus, the standard intensity of illumination for main roads and other roads is set at 20 and 10 lx, respectively, so that the above conditions can be fulfilled to the greatest possible extent with reference to the **General rules of recommended lighting levels (JIS Z 9110), Lighting for Roads (JIS Z 9111), Standards and Commentaries for the Installation of Road Lighting Facilities**, and actual measurements at existing lighting facilities. For lighting in tunnels, refer to the **Lighting of Tunnels for Motorized Traffic (JIS Z 9116)** and the **Standards and Commentaries for the Installation of Road Lighting Facilities.**

⑦ **Parking lots**

Even at slow speeds, vehicles need to be carefully maneuvered in parking lots. In particular, vehicles need lighting to accurately identify the locations of other parked vehicle. From the viewpoint of crime prevention, a certain level of intensity of illumination is required in parking lots so that vehicles and persons can be identified.

Thus, the standard intensity of illumination for the parking lots in ferry wharves and other parking lots is set at 20 and 10 lx, respectively, with reference to the **Order for Enforcement of the Parking Place Act**, the **General rules of recommended lighting levels** and actual measurements at existing lighting facilities. **(JIS Z 9110: 2010)**.

⑧ **Parks and green spaces**

The lighting at parks as places of relief needs to provide the following types of brightness:

(a) Brightness to ensure safe routes where people walk

(b) Brightness of atmosphere enabling people to feel psychologically safe

(c) Local brightness to highlight the beauty of trees and objects inside parks

(d) Brightness to crime prevention

Thus, the standard intensity of illumination and lighting methods shall be appropriately determined by taking into consideration the locations and areas, use purposes and situations, and facilities and objects requiring lighting. The standard intensity of illumination for paths is set at 3 lx with reference to the **General rules of recommended lighting levels (JIS Z 9110)**, and actual measurements at existing lighting facilities (**Table 9.4.1**).

⑨ **Others**

Mooring facilities where ships berth or unberth at nighttime should be provided with lighting as needed to make the normal lines of berths or the locations of corner sections easily identifiable.

(3) Standard Intensity of Illumination for Indoor Lighting

① The values shown in **Table 9.4.2** can be used for the standard intensity of illumination of each type of indoor facility.

Table 9.4.2 Standard Intensity of Illumination of Indoor Lighting

Facility		Standard intensity of illumination (lx)
Passenger terminal	Waiting lounges	300
	Passenger boarding paths and gates	100
Shed and Warehouse	Cargo handling spaces for fishing boat berths	200
	Container freight stations and dedicated vehicle sheds	100
	Rough work sheds and warehouses	70
	Other sheds and warehouses	50

② **Passenger terminals**

Considering that waiting lounges are places for relaxation, they shall provide people with a comfortable environment where they can feel at ease. Thus, the standard intensity of illumination for waiting lounges is set at 300 lx with reference to the **General rules of recommended lighting levels (JIS Z 9110)**, the **Standard for the Design and Construction of Electric Workpieces (Power Utilities)**, and actual measurements at existing lighting facilities (**Table 9.4.2**). The standard intensity of illumination for paths and gates is set at 100 lx with particular attention to ensuring the safety at these facilities with reference to the **General Rules of Recommended Lighting Levels (JIS Z 9110)**, the **Standard for the Design and Construction of Electric Workpieces (Power Utilities)**, and actual measurements at existing lighting facilities.

③ **Sheds and warehouses**

The standard intensity of illumination for the cargo handling spaces of fishery berths is set at 200 lx to facilitate the accurate judgment of fish freshness. For areas such as container freight stations where complex cargo handling operations are executed and facilities such as dedicated vehicle sheds that are important to security, the standard intensity of illumination is set at 100 lx.

Furthermore, for areas in sheds and warehouses where cargo sorting work or other work requiring safety is executed, the standard intensity of illumination is set at 70 lx with reference to the **Ordinance on Industrial Safety and Health** and the **General Rules of Recommended Lighting Levels (JIS Z 9110)**.

For other sheds and warehouses, the standard intensity of illumination is set at 50 lx with reference to the **General Rules of Recommended Lighting Levels (JIS Z 9110)** and actual measurements at existing lighting facilities. The standard intensity of illumination for the administration offices attached to passenger terminals, sheds, and warehouses shall be appropriately set with reference to the **General Rules of Recommended Lighting Levels (JIS Z 9110)**.

(4) Methods for calculating intensity of illumination

① **Methods for calculating intensity of illumination**

The methods for calculating the intensity of illumination include the flux method and the point-by-point method. Given that the flux method can be expressed by a relatively simple equation, it has been used for calculating the required number of lighting apparatuses in general. The point-by-point method is capable of accurately calculating the intensity of illumination required for specific points; therefore, it has been used generally for examining the evenness of intensity of illumination in a manner that obtains the intensity of illumination at individual points in areas for which the required intensity of illumination is calculated by the flux method.

(a) Calculation of intensity of illumination by the flux method

The average intensity of illumination in an area to be lit can be calculated by **equation (9.4.1)**.

Average intensity of illumination

$$E = \frac{NFUM}{A} \tag{9.4.1}$$

where

E : average intensity of illumination (lx);

N : number of lighting apparatuses (units);

F : total light flux per light source (lm);

U : utilization factor;

M : maintenance factor;

A : area of a plane of illumination (m^2).

(b) Calculation of intensity of illumination by the point-by-point method[23)]

In lighting design, the point-by-point method can be generally used to obtain the evenness of the intensity of illumination (**equation (9.4.4)**).

In the point-by-point method, a plane of illumination is first divided into pieces of rectangles in a grid pattern, and the intensity of illumination at the centers of respective rectangles is then calculated. The direct horizontal illumination E_h of light from a light source L at point P on a plane is expressed by **equation (9.4.2)** (refer to **Fig. 9.4.1**).

$$E_h = \frac{I_\theta \cos\theta}{l^2} \tag{9.4.2}$$

where

E_h : direct horizontal illumination at point P (lx);

I_θ : brightness of light with an incident angle of θ (cd);

l : distance from a light source to point P (m);

θ : incident angle (°).

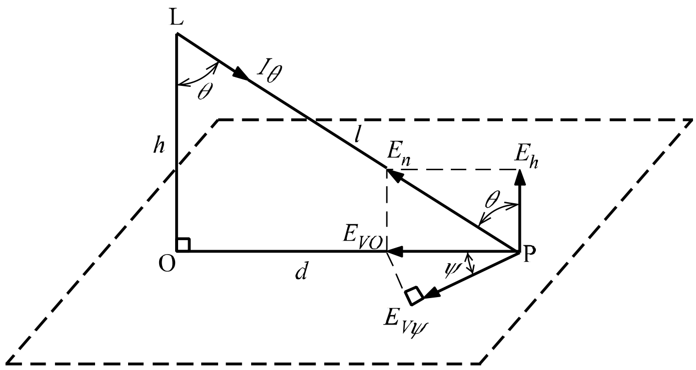


Fig. 9.4.1 Horizontal Illumination at Point P

The shape of each divided piece should be as close to a square as possible, and the finesses ratio of each piece is in the range of 0.5 to 2.0. The maximum length (p) of a side of each divided piece is the smallest length in equation **(9.4.3)**.

$$p \leq 0.2 \times 5^{\log d}, \quad p = 10 \tag{9.4.3}$$

where

d : width of a calculation range (m);

p : maximum value of a side of each divided piece (m).

The evenness (E_{av}) is the minimum average intensity of illumination, and the average intensity of illumination can be calculated by **equation (9.4.4)** (refer to **Fig. 9.4.2**).

$$E_{av} = \frac{E_1 + E_2 + \cdots + E_n}{n} \tag{9.4.4}$$

where

n : number of calculation points;

E_n : intensity of illumination at the center of nth divided piece.

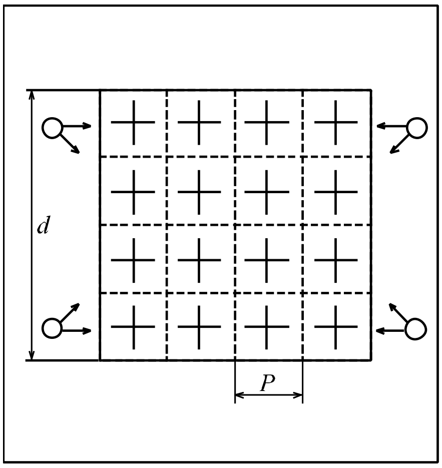


Fig. 9.4.2 Division of the Plane of Illumination in a Grid Pattern

② **Light ratio**

The light ratio is the ratio of light flux reaching the plane to be lighted to the whole flux of the light source. When calculating the intensity of illumination, it is preferable to take into consideration the fact that the utilization factors of indoor lighting varys depending on the efficiency of lighting apparatuses, areas of planes of illumination, situations inside rooms, and reflection ratios of respective sections inside rooms.

The light ratio of outdoor lighting can be calculated from the efficiency of lighting apparatuses and the areas of planes of illumination. The light ratio that have been practically used are in the range of 0.2 to 0.5 and are generally set at 0.4.

③ **Maintenance factors**

The maintenance factor is a value obtained by dividing the intensity of illumination of a lighting apparatus after a lapse of a certain period by the initial intensity of illumination. The total light flux of a light source inside a lighting apparatus is largest in an early stage and is gradually reduced as lighting time advances. The maintenance factors are also reduced when lighting apparatuses become dirty. The maintenance factors used for performance verification are determined on the basis of the assumption that the replacement of lamps and cleaning of lighting apparatuses are appropriately implemented such that maintenance factors are ensured. The maintenance factor (M) can be calculated by equation **(9.4.5)**.

$$M = M_a M_f M_d M_w \tag{9.4.5}$$

where

M_a : lumen maintenance factor of a light source;

M_f : residual factor of a light source;

M_d : partial maintenance factor due to dirt on a light source and a lighting apparatus;

M_w : partial maintenance factor due to an interior surface.

The lumen maintenance factors M_a and the residual factors M_f of light sources are determined by the characteristics of light sources; therefore, they are collectively called light source design lumen maintenance factors M_l. By contrast, partial maintenance factors due to dirt on light sources and lighting apparatuses are collectively called lighting apparatus design lumen maintenance factors M_d. The dirt and color degradation on interior surfaces cause the reductions in reflection ratios, thereby reducing the intensity illumination of interreflection components. However, the partial maintenance factors M_w are generally neglected in the calculation of maintenance factors because the contribution ratios of M_w to the reduction in the intensity of illumination are smaller than those of light sources and lighting apparatuses. Thus, the maintenance factors (M) can be calculated by equation **(9.4.6)**. For the lighting apparatus design lumen maintenance factors of high-intensity electric-discharge (HID) and LED lamps, refer to literature 23).

$$M = M_{\ell} M_d \tag{9.4.6}$$

where

M_l : a light source design lumen maintenance factor;

M_d : a lighting apparatus design lumen maintenance factor.

9.4.4 Performance Verification of Illumination Distribution

(1) Unfavorable illumination distribution on the planes of illumination not only causes passengers and workers to feel uncomfortable but also creates dark places that are difficult to see, thereby causing reduced work efficiency and accidents. Lighting design should be performed with consideration to the following items.

① Appropriate adjustment of the ratio between installation intervals and heights of lighting apparatuses to achieve favorable illumination distribution

② Auxiliary lighting apparatuses in cases of shady plants or cargoes

③ Adherence to the guiding average value for horizontal illumination and the recommended value for evenness proposed in **GUIDE FOR LIGHTING EXTERIOR WORKS AREAS** by CIE. In this guideline, evenness is defined as the ratio of the minimum illumination to the average illumination.

9.4.5 Performance Verification for Glare

(1) Glare is excessive brightness or excessive irregularity in brightness and causes people to have uncomfortable feelings and reductions in eyesight. Glare with respect to lighting facilities can be categorized into glare affecting ships and glare affecting passengers and workers.

① **Glare affecting ships**

Glare may prevent crews or pilots from identifying beacons and other ships at anchor and trigger erroneous ship handling, thus leading to accidental contact or collisions with other ships or berths. Therefore, the distribution of light and the installation locations of lighting facilities shall be carefully examined to ensure safety in ship navigation.

② **Glare affecting passengers and workers**

Glare may prevent passengers and workers from identifying cargoes and shipping tags and obstacles and may cause reduced work efficiency and fatigue. Therefore, lighting facilities shall be installed appropriately in full consideration of the interaction between the heights of the visual lines of passengers and workers and lighting apparatuses to prevent light from getting into people's eyes directly. In **GUIDE FOR LIGHTING EXTERIOR WORKS AREA**S, the CIE recommends to observe the values set for controlling glare to prevent glare from hindering visual work and traffic safety.

9.4.6 Performance Verification of Light Colors and Color-Rendering Properties

(1) The color property of light sources can be expressed by light colors and color-rendering property.

(2) Light colors

One of the ways to numerically rate the reddishness and bluishness of light is to use color temperature. Light becomes bluish and reddish when the color temperature is higher than 5,000 K and lower than 3,300 K, respectively. Generally, metal halide lamps have the highest color temperature, followed by white mercury and fluorescent lamps with intermediary color temperature, orangish white incandescent and high-pressure sodium lamps, and orangish-yellow low-pressure sodium lamp with the lowest color temperature.

(3) Color-rendering property

The color-rendering property is the effect of light from certain light sources on the changes in colors of objects in comparison with the colors of objects lit by light from a standard light source. Generally, electric, fluorescent, and metal halide lamps have favorable color-rendering properties, followed by fluorescent mercury and high-pressure sodium lamps. Clear mercury lamps have color reproductive properties that are sufficiently favorable for the greenish colors of leaves but are unfavorable for the colors of other objects. Considering that low-pressure sodium lamps are single spectrum light sources, they cannot be used for discerning colors.

(4) Color temperature and thermal sensation

The color temperature (K) is used to numerically express light colors. The degree of color temperature affects the thermal sensation, i.e., light colors come closer to red and bluish white because the color temperature is reduced and increased, respectively. **Table 9.4.3** shows the relationship between color temperature and thermal sensation.

Table 9.4.3. Relationship between Color Temperature and Thermal Sensation

Color temperature (K)	Thermal sensation
3,300 or less	Warm
3,300 to 5,300	Neutral
5,300 or higher	Cool

(5) Color-rendering property and average color-rendering indexes

Average color-rendering indexes (Ra) are used as typical indexes to represent the degree of color-rendering property. Ra is an average of rendering indexes with respect to eight prescribed test colors. The CIE shows the applicability of classified Ra to the different types of outdoor workplaces. **Table 9.4.4** shows the relationships of classes, ranges of Ra in respective classes, types of lamps, and applicable workplaces.

Table 9.4.4. Classification of Color-Rendering Property for Outdoor Lighting

Color-rendering property class	Average color-rendering index (Ra)	Type of lamp	Applicability
1	80 ≤ Ra Very good	Incandescent lamp	Applicable to workplaces requiring color differentiation
2	60≤ Ra < 80 Good	Fluorescent lamp, metal halide lamp, high-pressure sodium lamp with improved color-rendering property, LED lamp	
3	40 ≤ Ra < 60 Satisfactory	Mercury lamp	Applicable to workplaces of general work
4	20 ≤ Ra <40 Allowable	High-pressure sodium lamp	
5	Ra < 20	Low-pressure sodium lamp	Not applicable to workplaces where color differentiation is important

9.4.7 Performance Verification for Obstacle Light and Energy Saving

(1) Light leaking from outdoor lighting facilities affects the surrounding environments in terms of disturbance in astronomical observation, increased burden on the ecosystem, and glare interfering visual identification and causes loss of energy by lighting unnecessary objects. Considering that leaked light may cause social problems, it is preferable to give due consideration to the prevention of leaked light in lighting design.

9.4.8 Selection of Light Sources

(1) Light source for wharf lighting is preferably selected by considering the following requirements:

① The light source should have high efficiency and a long service life.

② The light source should be stable against the variations of ambient temperature.

③ The light source shall provide a good light color and good color-rendering performance.

④ The time of stabilization of the light after turning-on shall be short.

(2) Any light source other than a light bulb shall be used together with an appropriate stabilizer.

(3) Types of light sources

Fig. 9.4.3 shows the classification of light sources. **Table 9.4.5** shows the summary of the characteristics of the respective types of lamps. The light sources generally used on wharves are those classified as HID lamps with the following characteristics:

① **High-pressure sodium lamps**

High-pressure sodium lamps have lower efficiency than low-pressure sodium lamps but have a long service life, favorable start-up performance, and improved color-rendering property. They also have an orangish color and require pulse voltage to start lighting. There are two types of high-pressure sodium lamps: one is a starter stabilized type with a pulse generator (starter) integrated with a lamp; and the other is a dedicated stabilizer type with the pulse generator stored in a stabilizer. The former type has approximately 10% higher efficiency than the latter type.

② **Fluorescent mercury lamps**

Fluorescent mercury lamps have lower efficiency than high-pressure sodium lamps but have a good color-rendering property. These lamps have a white light color that is similar to fluorescent lamps and can be used for yard lighting.

③ **Metal halide lamps**

Metal halide lamps have a slightly shorter service life than high-pressure sodium lamps but can be used for yard lighting similar to high-pressure sodium lamps because they have a good color-rendering property. They have a whitish light color.

④ **LED lamps**

LED lamps are light sources with a whitish light color and lower color-rendering property than halogen lamps and incandescent lamps but have a favorable service life, efficiency, and economic performance. Although LED lamps have a long service life, their accessories need to be replaced after approximately 8 years.

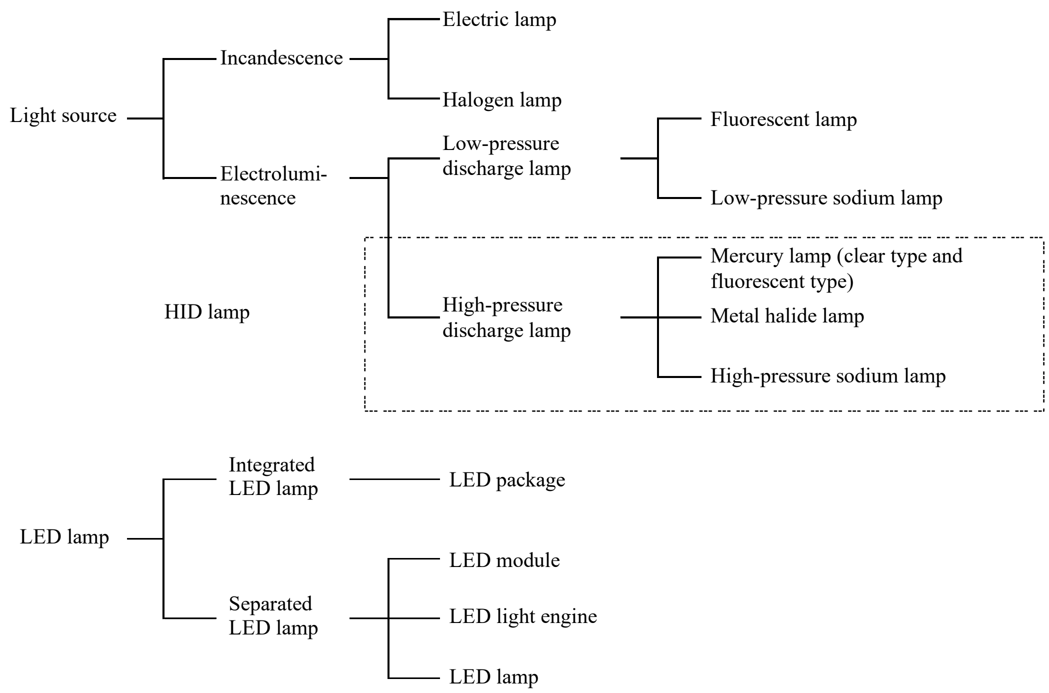


Fig. 9.4.3 Types of Light Sources

Table 9.4.5 Characteristics of Lamps

Characteristics / Type of lamp	Lamp efficiency (lm/w)	Light color (K)	Color-rendering property (Ra)	Service live (hours)	Ambient temperature dependency	Start-up performance	Restart performance	Modulation
Incandescent	Low 15 to 20	Orangish white 2,800	Good 100	Short 1,000 to 2,000	Stable	Instantaneous	Instantaneous	Easy
Halogen	Low 17 to 22	Orangish white 3,000 to 3,200	Good 100	Short 1,000 to 2,000	Stable	Instantaneous	Instantaneous	Easy
Fluorescent (white)	Medium 80 to 100	White 3,000 to 4,000	Slightly good 50 to 95	Long 6,000 to 12,000	Affected	Fast 2 to 3 seconds	Fast 2 to 3 seconds	Possible
Low-pressure sodium	Highest 100 to 180	Orangish yellow 1,700	Bad –	Normal 9,000	Stable	20 minutes	Slightly fast 10 seconds	Difficult
Mercury	Slightly low 40 to 60	White (bluish) 3,500 to 4,000	Normal 40 to 50	Long 9,000 to 12,000	Stable	8 minutes at normal temperature	Slightly slow Less than 10 minutes	Possible up to 50%
Metal halide	Medium 70 to 80	White 4,000 to 6,500	Good 70 to 90	Normal 6,000 to 9,000	Slightly affected	5 minutes at normal temperature	Slightly slow Less than 10 minutes	Difficult
High-pressure sodium	Medium 60 to 120	Orangish white 2,100	Normal 25 to 80	Long 9,000 to 12,000	Stable	5 to 10 minutes	Slightly fast 1 to 5 minutes	Possible up to 50%
White LED (equivalent to electric lamp)	Slightly high 60 to 150	Orangish white 2,800	Slightly good 70 to 80	Long up to 40,000	Stable	Instantaneous	Instantaneous	Easy

(4) Efficiency

It is preferable to enhance not only the efficiency of lamps but also the comprehensive efficiency of lighting facilities, including stabilizer and accessories.

(5) Ambient temperature

① **Influences on efficiency**

Most light sources are not affected by ambient temperature, except fluorescent lamps. The efficiency of fluorescent lamps is reduced not only with the increase in ambient temperature but also with the decrease in ambient temperature. Therefore, it is preferable to give due consideration to the fact that the degrees of influences of ambient temperature on the efficiency of fluorescent lamps vary depending on the structures of lighting apparatuses.

② **Influence on start-up and restart time**

Electric, high-pressure sodium, and low-pressure sodium lamps are not affected by ambient temperature, but fluorescent and metal halide lamps degrade their start-up performance when ambient temperature is reduced. In such cases, lamps and stabilizers shall be provided with special measures to prevent the degradation of start-up performance.

9.4.9 Selection of Apparatuses

(1) Outdoor lighting

① It is preferable to select apparatuses for outdoor lighting in consideration of the following requirements:

(a) Lighting apparatuses should have rainproof structures. Furthermore, they shall have explosion-proof structures for cases wherein large amounts of flammable dangerous goods are handled in the proximity of lighting apparatuses. In particular, outdoor lighting facilities using LED apparatuses should be provided with measures against low temperature and rainwater to prevent apparatuses from dew condensation.

(b) The materials used for lamp bodies, reflector surfaces, and illumination covers should have good quality, have high durability, and good resistance against deterioration and corrosion.

(c) Sockets should be compatible with respective light sources.

(d) Stabilizers and internal wiring should be capable of withstanding the expected increase in the temperature of apparatuses.

(e) Outdoor lighting apparatuses should have high efficiency.

(f) Luminous intensity distribution should be controlled appropriately in consideration of the use purposes of respective apparatuses.

② **Types of apparatuses**

Considering that wharves have a wide variety of outdoor facilities such as aprons, yards, roads, parking lots, parks, green areas, and squares, it is preferable to select lighting apparatuses that are appropriate for outdoor facilities. The main apparatuses of outdoor lighting facilities are as follows:

(a) Projectors

Projectors are apparatuses that have axisymmetric luminous intensity distribution to focus light on relatively narrow projection angles and are suitable for lighting wide areas. These apparatuses are installed on poles, steel towers, roofs, and walls of buildings. Some square projectors have asymmetric intensity distributions.

(b) Lighting apparatuses for roads

These lighting apparatuses are normally installed on poles with heights of 8 to 12 m.

(c) Others

There are other types of lighting apparatuses installed directly on or suspended from eaves of sheds.

③ **Structures of apparatuses**

Apparatuses shall have structures that facilitate the replacement, maintenance, and inspections of lamps; have opening and closing sections that are capable of being fastened by simple and reliable means; and are free from danger.

Sockets should have structures that can keep lamps from falling out or loosing connections even when apparatuses are subjected to vibrations. Furthermore, waterproof seals and internal wiring shall have resistance to temperature increase owing to heat radiation from apparatuses. It is also preferable that the materials and finishing of apparatuses, including poles and steel towers to which apparatuses need to be attached, have resistance to chloride-induced corrosion. Apparatuses installed in areas such as wharves that require particular attention to fires shall have explosion-proof structures that complying with the **Electrical Apparatus for Explosive Atmospheres in General Industry (JIS C 0903)**. Apparatuses with a risk of dazing passengers, workers, crews, or pilots with glare should have louvers or hoods to prevent glare.

④ **Waterproof structures of apparatuses**

Considering that apparatuses for outdoor lighting facilities are exposed to wind and rain, they should have waterproof structures, as stipulated in the **Test to Prove Protection against Ingress of Water and Degree of Protection (JIS C 0920)**.

⑤ **Outdoor lighting methods**

(a) Lighting method using high poles

This is a general lighting method for roads using 8 to 12 m poles with lighting apparatuses attached to them. This method requires a large number of poles when used for lighting wide areas such as parking lots and may hinder cargo handling work. Therefore, this method is suitable for small-scale parking lots, ferry boarding facilities, and places where no cargo handling work is executed.

(b) Method for lighting from high places

This is a method that uses structures with heights of 15 to 40 m and sizes larger than lighting poles so that wide areas can be illuminated with a small number of structures. This method is suitably used for yards, large-scale parking lots and other large areas with wide lighting ranges.

(c) Method for lighting from sheds

This is a method that uses buildings such as sheds if they exist near the places that require lighting in a manner that installs lighting apparatuses on roofs or side walls of buildings.

(d) Catenary lighting method

This is a method that uses overhead wires installed between poles or buildings at wide intervals (60 to 90 m) with lighting apparatuses suspended from the wires. This method requires a smaller number of poles than lighting method using high poles. Furthermore, this method enables lighting apparatuses to be moved closer to each other so that illumination distribution can be improved.

⑥ **Fig. 9.4.4** shows an example of the lighting method using high poles.

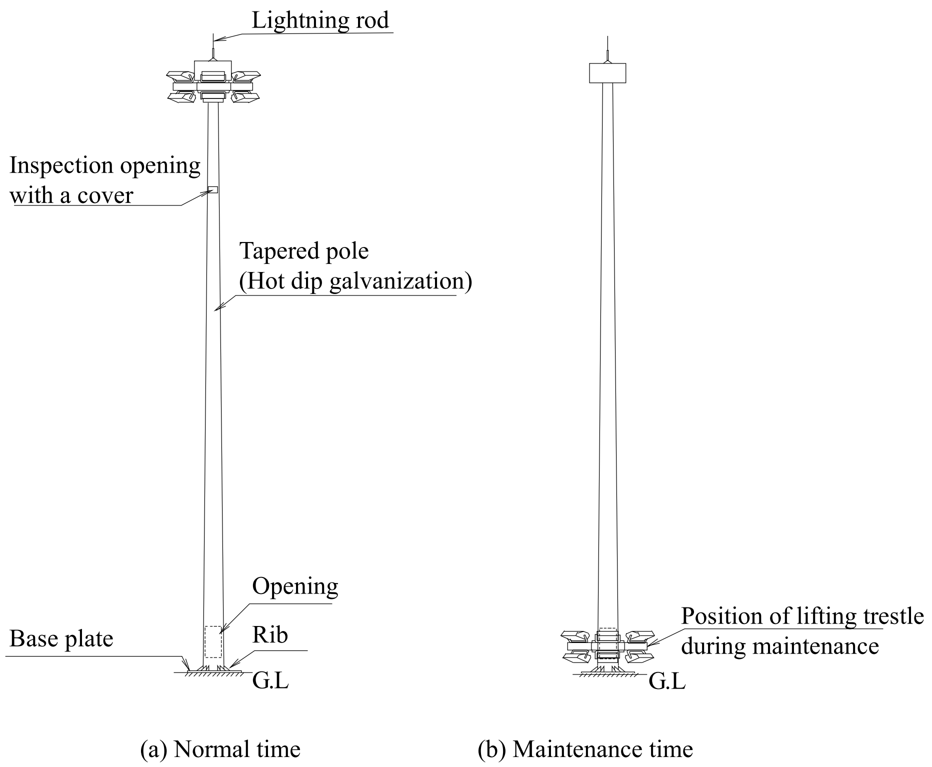


Fig. 9.4.4 Example of Lighting Method Using High Poles
(Lifting Type Mounted with Running Block and Balance Weight)

(2) Indoor lighting

① The selection of the apparatuses for indoor lighting should be made in consideration of the following requirements:

(a) Luminous intensity distribution should be controlled appropriately in consideration of the use purpose of respective apparatuses.

(b) Sockets should be compatible with respective light sources.

(c) Stabilizers and the internal wiring should be capable of withstanding the expected increase in the temperature of the equipment.

(d) Lighting apparatuses should have high efficiency.

② **Types of apparatuses**

General indoor lighting apparatus can be classified into ceiling mounted, ceiling embedded, and hanging types. Given that each type has axisymmetric or similar luminous intensity distributions, it is preferable to select appropriate apparatuses on the basis of their use purposes.

③ **Structures of apparatuses**

Apparatuses shall have structures that facilitate the replacement, maintenance, and inspections of lamps and are free from danger. Furthermore, stabilizers and internal wiring should be capable of withstanding the expected increase in the temperature of the apparatuses.

When selecting apparatuses for places where people rest, such as waiting lounges, it is also important to select apparatuses that are harmonized with buildings. Furthermore, apparatuses with resistance to corrosion are preferably selected for the types of sheds where apparatuses are subjected to corrosive gases.

④ **Indoor lighting methods**

(a) Method for lighting from ceilings

This method can be classified into the following three categories:

1) Ceiling mounted method

2) Ceiling embedded method

3) Hanging method

Among the three methods, the ceiling mounted method is the most economical and easiest method. Although the ceiling embedded method is economically disadvantageous, it is suitable for waiting lounges and paths where aesthetic value is important. The hanging method is suitable for sheds and warehouses with high ceilings.

(b) Floodlighting method

This method creates shadows behind cargoes, thus hindering cargo handling operations; therefore, is not suitable for these facilities.

9.4.10 Maintenance

(1) Inspections

① Inspections shall be periodically performed on the following:

(a) Lighting states

(b) Dirt on and damage to apparatuses

(c) Flaking states of paint

② Illumination intensity measurement should be periodically performed in a manner that selects plural measurement points at the typical places of respective facilities.

③ **Inspection frequency**

Although the inspection frequency shall be set by taking into consideration the types of lighting apparatuses, installation locations, meteorological, and oceanographic conditions, it is preferable to implement inspections at the following intervals depending on the inspection items:

(a) Lighting states: 1 month

(b) Other inspection items: 1 year

④ **Illumination intensity measurement points**

The standard illumination intensity is determined by obtaining the minimum value of average illumination intensity, and it requires a complex operation to confirm the standard illumination intensity. Therefore, the changes in illumination intensity are generally confirmed in a manner that preliminarily sets plural measurement points and monitors illumination intensity at these points.

⑤ **Illumination intensity measurement frequency**

It is preferable that illumination intensity is measured at the following intervals on the basis of the importance of facilities and the lengths of lighting time:

(a) Important facilities with long lighting times: 6 months

(b) Other facilities: 1 month

⑥ **Decline in light flux**

Light sources undergo a decline in light flux as lighting time advances. Therefore, lamps need to be replaced when light flux becomes lower than the design light flux. For design lumen maintenance factors, a reference can be made to literature 23).

(2) Cleaning and repair

① The lighting function of lighting facilities should be maintained via the implementation of repair and cleaning when inspections identify damage to lighting facilities, unlit lamps, and dirt on apparatuses.

② Given that dirt on the interior surfaces of lighting apparatuses reduces the illumination intensity on road surfaces, these apparatuses should be cleaned once they are identified to be dirty as a result of visual inspection or the measurement of illumination intensity.

③ Inspections should be implemented with particular focus on chloride-induced corrosion, and flaking paint should be repaired promptly.

④ It is necessary to identify the causes for the insulation failures of wiring and the failures in the control function of wiring devices and to remove such causes by performing repair to prevent unlit lamps.

⑤ As important data for future maintenance work, recoding ledgers should be prepared for respective installed lighting facilities to keep the records of structural types of lighting apparatuses, support columns, foundations, wiring devices, and management numbers of support columns. It is also preferable to keep the records of cleaning and repairs in the ledgers.

9.5 Staircases and Ladders

9.5.1 General

The intervals of staircases and ladders shall be appropriately set in accordance with the sizes and use states of facilities.

In facilities that are expected to be used by passengers when they embark on or disembark from ferries or other passenger ships, staircases are preferably installed at easily accessible places at one or more locations for every berth for emergency situations. Generally, staircases are preferably positioned at the anterior or posterior ends of respective wharves so that cargo handling work is not disturbed. Unlike ladders, staircases are used by passengers when they embark on and disembark from small ships or by workers when they load or unload small cargoes. Therefore, staircases need to have structures that ensure the safety of people. Ladders should also have structures that not only ensure the safety of passengers during their embarkation and disembarkation but also enable people in the water to easily climb up.

9.5.2 Performance Verification

(1) Staircases preferably have a width of 0.75 m or more, a rise of 20 cm, and a tread of 30 cm with concrete surfaces that are roughly finished.

(2) Generally, staircases should have landings. Staircases installed in areas with large tidal levels should have landings at shorter intervals than normal cases. The length of landings can be 1.5 m.

(3) When installing ladders on mooring facilities, ladders should be installed at the transition positions between berths so that the mooring of ships will not be disturbed. Generally, installation spaces are prepared by cutting out the portions of concrete superstructures of mooring facilities in the shapes of vertical trenches with a width of 75 cm and a depth of 30 cm, and ladders are placed in the trenches with a distance of approximately 20 cm from the cut concrete surfaces. Each spoke should have a downward gradient across it to prevent people from slipping and also have a one-way drainage gradient with a throating.

(4) It is preferable that spokes have a width of 45 cm and a vertical interval of 30 cm, with the lowermost spoke positioned below the L.W.L. Banisters are preferably embedded in wharves and extended 30 cm above the tops of mooring facilities and 45 cm inside normal lines or special jigs to ensure the safety of people.

(5) Generally, the surcharges on ladders shall be 1 kN per 1 m in both the vertical and horizontal directions. Mounting brackets should have special resistance to corrosion due to chloride damage and structures facilitating the repair of damaged or corrosive ladders when needed.

(6) The aprons at the foot of staircases and ladders should be protected from cargo handling machines by curbings and the like. In some cases, mooring posts or rings for small ships are used in place of curbings.

(7) There are two types of accommodation ladders: metallic and rubber. The selection of accommodation ladders shall be made by taking into consideration durability and resistance to berthing force because these ladders are subjected to corrosion and contact with small ships.

9.6 Lifesaving Facilities

[Public Notice] (Performance Criteria of Lifesaving Facilities)

Article 63

The performance criteria of lifesaving equipment shall be such that appropriate lifesaving equipment is provided and readily available as necessary so as to secure the safety of human beings on the mooring facilities to serve for passenger ships with gross tonnage equal to or larger than 500 tons.

9.6.1 General

(1) Lifesaving facilities refer to life rings and small ships.

(2) The types, shapes, installation locations, and materials of lifesaving facilities should be appropriately set to ensure the safety of users in accordance with the use conditions and structural characteristics of mooring facilities.

9.7 Curbings

[Public Notice] (Performance Criteria of Curbing)

Article 64

The performance criteria of curbing shall be as prescribed respectively in the following items:

(1) The curbing shall be installed at appropriate locations and shall be provided with the dimensions necessary for ensuring the safe use of mooring facilities while not hindering ship mooring and cargo handling in consideration of the structural types and usage conditions of the mooring facilities.

(2) The risk of impairing the integrity of curbing shall be equal to or less than the threshold level under the variable situation, in which the dominating action is collision of vehicles.

[Interpretation]

11. Mooring Facilities

(15) Performance Criteria of Curbing (Article 33 of the Ministerial Ordinance on Criteria and the interpretation related to Article 64 of the Public Notice)

The required performance of curbing under the variable action situation in which the dominant action is the impact of vehicles should focus on serviceability. **Attached Table 11-32** shows the performance verification items and standard indexes to determine limit values with respect to the action.

In the performance verification of curbing, the standard indexes to determine limit values with respect to their soundness shall be appropriately set in accordance with materials and the like.

Attached Table 11-32 Performance Verification Items and Standard Indexes to Determine Limit Values under Respective Design Situations (excluding accidental situation) of Curbing

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
33	1	2	64	–	2	Serviceability	Variable	Impact of vehicle	–	Soundness of curbing	–

9.7.1 General

(1) The structures, shapes, layouts, and materials of curbing should be set properly in such a way that the safety of users is ensured and that cargo handling work is not hindered in consideration of the structural characteristics and use conditions of mooring facilities.

(2) The materials generally used for curbing are concrete (with steel plate cladding), concrete (with cast iron cladding), prestressed concrete, resin concrete, synthetic resin, and square steel pipes.

(3) It is preferable to take comprehensive hazard prevention measures in a manner that combines curbings with warning signs, road markings, and barricades as needed. Particular attention is required for hazard prevention at the edges and boundaries of berths where vehicles are at high risk of falls.

(4) The locations and dimensions of curbings should be appropriately set to ensure the safety of users, smooth ship mooring, and cargo handling in accordance with the structures and use conditions of mooring facilities.

9.7.2 Performance Verification

(1) The distance intervals between curbings need to be shorter than the wheel treads of the cargo handling equipment and vehicles. They may be set at approximately 30 cm in general to drain rainwater from the aprons. However, it is preferable to set the intervals of curbing installed at both side of mooring posts at 1.5–2.5 m. In cases wherein vehicles are not expected to pass because fences or other barriers are set up to prohibit the passage of vehicles, there is no need to install curbing.

(2) It is preferable that the heights of curbing are separately set for dangerous and general zones depending on the degree of dangers due to vehicle falls (refer to **Table 9.7.1** and **Fig. 9.7.1**).

Table 9.7.1 Heights of Curbing

Degree of Danger	Height of curbing	Example
Dangerous zone	25 to 30 cm	Berth edges and berth boundary
General zone	15 to 20 cm	Areas other than dangerous zones

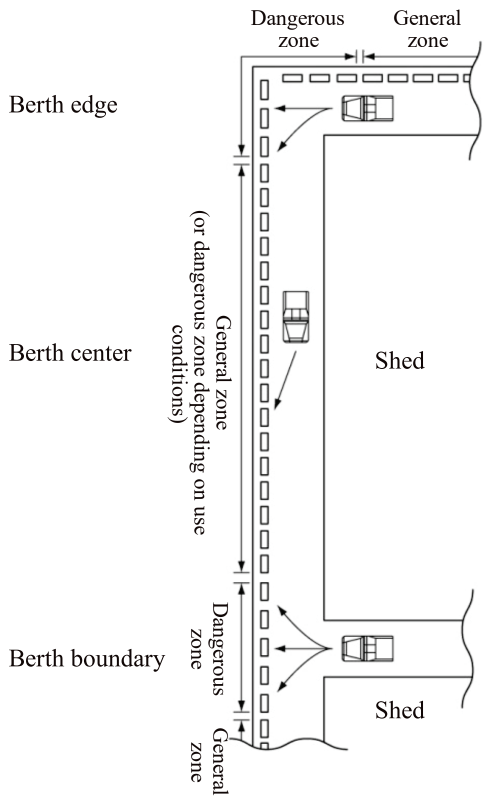


Fig. 9.7.1 Dangerous and General Zones on Passages

(3) Curbing is preferably installed at places approximately 10 cm landward from the face lines of wharves.

(4) The performance verification of curbing should be made by taking into consideration durability and visibility. Furthermore, refer to the **Curbing Design Manual**.[24)]

(5) Some curbing is installed with spaces between their bottom faces and road surfaces for the purpose of draining rainwater.

9.8 Vehicle Loading Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.8.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.9 Water Supply Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.9.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.10 Drainage Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.10.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.11 Fueling Facilities and Electric Power Supply Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.11.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.12 Passenger Boarding Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.12.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.13 Fences, Doors, Ropes, etc.

(English translation of this section from Japanese version is currently being prepared.)

9.13.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.14 Monitoring Equipment

(English translation of this section from Japanese version is currently being prepared.)

9.14.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.15 Rest Rooms

(English translation of this section from Japanese version is currently being prepared.)

9.15.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.16 Signs

(English translation of this section from Japanese version is currently being prepared.)

9.16.1 Placement of Signs and Marks

(English translation of this section from Japanese version is currently being prepared.)

9.16.2 Forms and Installation Sites of Signs

(English translation of this section from Japanese version is currently being prepared.)

9.16.3 Installation Locations of Signs

(English translation of this section from Japanese version is currently being prepared.)

9.16.4 Structure of Signs

(English translation of this section from Japanese version is currently being prepared.)

9.16.5 Raw Materials

(English translation of this section from Japanese version is currently being prepared.)

9.16.6 Maintenance and Management

(English translation of this section from Japanese version is currently being prepared.)

9.16.7 Guard Fences

(English translation of this section from Japanese version is currently being prepared.)

9.16.8 Barricades

(English translation of this section from Japanese version is currently being prepared.)

9.17 Fire-fighting Equipment and Alarm Systems

(English translation of this section from Japanese version is currently being prepared.)

9.17.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.18 Aprons

[Public Notice] (Performance Criteria for Aprons)

Article 73

The performance criteria for aprons shall be as prescribed respectively in the following items:

(1) Aprons shall be provided with the dimensions necessary for enabling the safe and smooth cargo handling operations.

(2) The surface of aprons shall be provided with the gradient necessary for draining rainwater and other surface water.

(3) Aprons shall be paved with appropriate materials in consideration of surcharge loads and the usage conditions of the mooring facilities.

(4) The risk of incurring damage to the pavement to the extent of affecting cargo handling operatons shall be equal to or less than the threshold level under the variable situation in which the dominating action is surcharge load.

[Interpretation]

11. Mooring Facilities

(17) Performance Criteria of Aprons (Article 33 of the Ministerial Ordinance and the interpretation related to Article 73 of the Public Notice)

① The required performance of aprons shall be usability. Here "usability" means the following.

a) Apron widths shall be properly set to ensure safe and smooth cargo handling.

b) Apron gradients shall be properly set to drain rainwater and other surface water.

c) Aprons shall be paved with proper materials taking account of the surcharges and the use conditions of mooring facilities.

② In addition to the above, the required performance of aprons, under the variable situation in which the dominating action is surcharge, shall be serviceability. The performance verification items and standard indexes to determine limit values with respect to the action shall be as shown in Attached Table 11-34. In the performance verification of apron pavement, the standard indexes to determine limit values with respect to the soundness shall be appropriately set in accordance with materials and the like.

Attached Table 11-34 Performance Verification Items and Standard Indexes to Determine Limit Values under Respective Design Situations (Excluding Accidental Situation) of Aprons

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
33	1	2	73	-	4	Serviceability	Variable	Surcharge	–	Soundness of pavement	–

9.18.1 General

The performance verification of aprons shall be carried out in terms of both dimensions and pavements.

9.18.2 Dimension of Aprons

(1) Apron widths

① The apron widths of ordinary mooring facilities may generally refer to the values shown in Table 9.18.1.

Table 9.18.1 Apron Widths

Berth water depth (m)	Apron width (m)
Less than 4.5	10
4.5 or more and less than 7.5	15
7.5 or more	20

② However, because required apron widths of container wharves and internal trade unit load terminals vary depending on the types of cranes, ships, and cargo handling methods, the widths shall be properly set, taking account of the traveling widths of cargo handling equipment and trailers as well as the actual situation of cargo handling operation.

③ The determination of apron widths of general cargo wharves shall normally take account of the spaces for cranes, temporary storage, cargo handling, and traffic paths. It is preferable that aprons have the widths of not less than 15 to 20 m when sheds are installed at the back and fork lifts are used, and not less than 10 to 15 m when roads and open storage yards are in the immediate vicinity and trucks are allowed to drive into the aprons for cargo handling operations directly to and from general cargo ships.

(2) Apron gradients

① Aprons are where cargo handling is performed and closely related to the conditions of cargo handling operation at the backyards, and hence transverse slopes need to be properly determined taking these conditions into consideration.

② Aprons normally have a down slope of 1 to 2% toward the sea. Shallow draft wharves have steep slopes. Aprons in snowy places often have relatively steep slopes for the easy removal of snow. In some cases, reverse slopes are used depending on the conditions of use of aprons and environmental consideration.

③ Since the settlement of backfilling may cause slopes to be reversed, construction should be carefully performed.

(3) Countermeasures against apron settlement

① Appropriate countermeasures shall be taken to prevent settlement due to sand washing-out or consolidation of lower landfill materials from hindering cargo handling operation and the vehicle traffic on aprons.

② In general, apron pavements are at risk for settlement due to the consolidation of the layers below the subgrade of the apron pavements. There are many other cases of settlement caused by the washing-out of landfill soil used as a part of the layers below the subgrade through the joints of quaywalls or compression of backfill materials. In many cases, settlement is considered to be the main reason for the failures of apron pavements.[28)] Therefore, it is preferable to consider preventive measures against the settlement of the layers below the subgrade such as sand washing-out prevention work and compaction of backfilling materials. There may be the cases where aprons are placed into service with temporary pavements until settlement subsides and full-scale pavements are implemented or aprons are provided with block pavements so as to facilitate future maintenance.

③ When aprons undergo settlement to such an extent that hinders cargo handling or vehicle traffic, repairs including leveling work shall be implemented. In the case of leveling work by constructing granular base courses on existing pavements or asphalt mixture layers on existing concrete pavements, proper drainage measures shall be taken against possible adverse impact of accumulated water due to the low permeability of existing pavements on leveling work.

④ Special attention is required when carrying out performance verification and executing paving work in the case where fluctuations of tide levels may cause the elevation of ground water levels higher than the subgrade of apron pavements.

9.18.3 Performance Verification

(1) General

① The types of apron pavements shall be properly selected in a comprehensive judgment taking account of the soil property below the subgrade, constructability, surrounding pavement conditions, cargo handling methods, economic efficiencies, and maintenance.

② The types of apron pavements include concrete, asphalt, semi-flexible, block, reinforced concrete, continuous reinforced concrete, prestressed concrete (PC), and interlocking block pavements. In addition, colored pavement is another type used when aesthetic property is of importance. The former two types of pavements have been often used for the aprons of mooring facilities, and the performance verification of these two types can be referred to **Part II, Chapter 5, 9.18.3 (4) Performance verification of concrete pavement** and **9.18.3 (5) Performance verification of asphalt pavement**.

③ Because there is no uniform way of selecting an optimal type of pavement, it shall be appropriately determined taking into consideration a variety of factors including the use conditions of aprons. When a concrete pavement is desirable but site conditions or other factors do not allow it to be constructed, the alternative choice is preferably a semi-flexible pavement or an asphalt pavement using special asphalt having dynamic stability equivalent to a semi-flexible mixture.

④ The characteristics of respective types of pavements are as follows.

(a) Concrete pavement

1) The advantages of concrete pavements

i. Concrete enables sufficient pavement structures to be constructed without being much affected by the bearing capacity of subgrade and its heterogeneous property and thereby reducing the thicknesses of base courses.

ii. Concrete pavements are extremely resistant to concentrated loads with large contact pressure and, therefore, advantageous to the use of truck cranes with outriggers.

iii. Concrete slabs have high durability and thereby extending the service life.

iv. Concrete surfaces have high abrasion resistance suitable for the scratching of cargo handling equipment.

2) The disadvantages of concrete pavements

i. Concrete pavements require a considerable number of days for curing after the completion of construction till the pavements are finally placed in service.

ii. Concrete slabs become difficult to repair once they start to be destructed, and demolition of destructed slabs requires large efforts.

iii. Even in the case of uneven settlement of the layers below the subgrade of pavements, concrete slabs hardly reflect the occurrence of such an incidence, and, therefore, even minor settlement can cause major destruction.

iv. There have been cases of pavement destruction induced by the contact between pavements and various structures.

(b) Asphalt pavement

1) Advantages of asphalt pavement

i. Paving work can be easily implemented in a phased manner and in accordance with the availability of construction investment. The phased implementation is advantageous to enhance consolidation settlement and strengthen subgrade while using aprons before implementing final paving.

ii. Asphalt pavements can alleviate the adverse effects of uneven settlement in the layers below subgrade to some extent without deteriorating serviceability.

iii. Asphalt pavements require a very short curing period, thereby enabling aprons to be used immediately after construction.

iv. Asphalt pavements are easy to be repaired.

2) Disadvantages of asphalt pavements

i. The service life of asphalt mixtures is relatively short.

ii. Asphalt pavements are weak against static loads with large contact pressure and repetitive loads applied to identical locations and are thereby being subjected to asperity and rutting.

iii. Asphalt pavements are susceptible to oil and heat and therefore require, for example, oil resistance surface treatment for the place with the risk of oil leaks.

iv. Asphalt pavements require complex construction management.

(c) Semi-flexible pavement

1) Advantages of semi-flexible pavements

i. Semi-flexible pavements are superior to asphalt pavements in terms of fluidity, oil and flame resistance.

ii. Semi-flexible pavements require shorter curing periods than concrete pavements.

2) Disadvantages of semi-flexible pavements

i. Semi-flexible pavements have shorter service life than concrete pavements and a risk of fine contraction cracks after construction due to the shrinkage when cement milk hardens or the fluctuations in external temperature.

ii. Semi-flexible pavements require longer curing periods than asphalt pavements because open-graded asphalt mixtures need to be constructed first before impregnating cement milk with them for final curing.

(d) Block pavements

1) Advantages of block pavements

i. Block pavements can alleviate the effects of uneven settlement in the layers below subgrade to some extent.

ii. The damage to block pavements due to settlement can be easily repaired at low costs.

iii. Block pavements enable aprons to be used immediately after construction.

2) Disadvantages of block pavements

i. The joints between blocks are subjected to damage and cause deterioration of vehicles' traveling performance.

ii. Block pavements require complex construction procedures.

(e) Reinforced concrete pavements

Reinforced concrete pavements have a mechanism to close cracks generated in concrete slabs with reinforcement bars arranged inside the slabs and to enable the interlocking effects of aggregates between crack surfaces to transfer loads. Although reinforced concrete pavements have larger strength than unreinforced concrete pavements, the slab thicknesses are normally identical in both cases.

(f) Continuous reinforced concrete pavements

Continuous reinforced concrete pavements use continuous longitudinal reinforcement bars to eliminate transverse joints on concrete slabs in a manner that controls the distribution of cracks in the transverse direction by the longitudinal reinforcement so as to reduce the crack widths of respective cracks. The thicknesses of concrete slabs are normally 80% to 90% of those of unreinforced concrete slabs.

(g) Prestressed concrete pavements

Prestressed concrete pavements achieve improved structural strength in a manner that reduces tensile stress generated in slabs by preliminarily applying compressive stress to concrete slabs.

(h) Interlocking block pavements

Interlocking block pavements are a type of block pavements and use high-strength concrete products formed into deformed blocks instead of rectangle blocks. The interlocking block pavement method can reduce deformation of pavement surfaces through the interlocking effects among blocks and has been frequently used for the aprons in many ports overseas.

⑤ The pier slabs normally have pavements to ensure flat traveling surfaces that can resist vehicle (traffic) loads and to prevent abrasion due to loads on the pier slabs. The pavements on pier slabs are classified into concrete pavement slabs placed directly on pier slabs and asphalt pavements comprising a base course and a surface course. Concrete pavement slabs are further classified into a single layer type where structural pier slabs and pavement slabs are cast simultaneously as integrated slabs with final surface finishing and a double layer type where pavement concrete is cast after the concrete of pier slabs hardens. It is necessary to pay attention to the surface finishing in the case of the single layer type and to the cracks on pavement concrete in the case of the double layer type.[29)]

(2) Fundamentals of performance verification

① The performance verification of apron pavements shall be generally such that pavement structures are stable under the surcharges by cargo handling vehicles and related equipment.

② **Fig. 9.18.1** shows an example of the performance verification procedures of apron pavements.

③ For the performance verification of apron pavements, reference can be made to the **Pavement Design and Construction Guide**[30)]; the **Manual for Pavement Design**[31)]; the **Manual for Airport Pavement Design**[32)]; and the **Manual for Airport Pavement Rehabilitation**[33)].

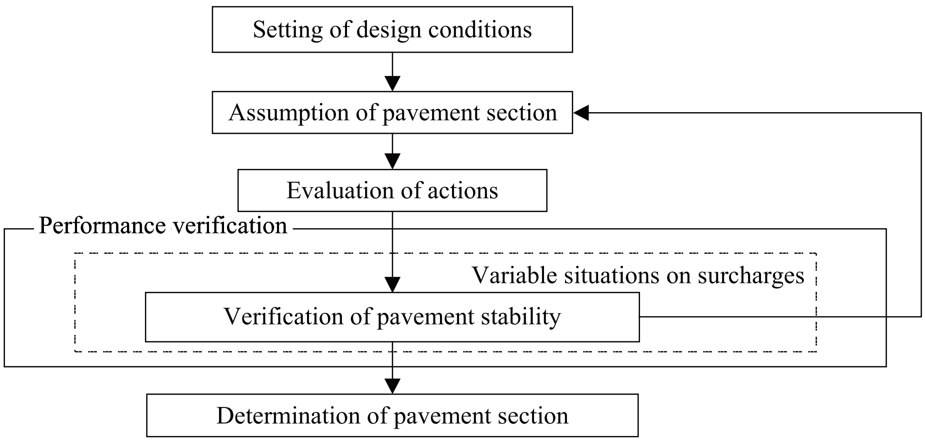


Fig. 9.18.1 Example of Procedures for the Performance Verification of Apron Pavements

(3) Actions

① Actions to be considered in the performance verification of apron pavements are generally the surcharges by mobile cranes (truck cranes, rough terrain cranes, and all terrain cranes), trucks, tractor trailers, fork lift trucks, straddle carriers, etc., depending on the types of cargoes and cargo handling methods. Generally, the performance verification of apron pavements shall be carried out for the maximum surcharges and the ground contact pressures that maximize the pavement thicknesses on the basis of the ground contact areas on which surcharges are applied.

② The characteristic values of the surcharges used for the verification of apron pavements may refer to **Table 9.18.2**.[34)] Outriggers are applied to the cases of movable cranes, where a wheel means a single wheel or dual wheels (two laterally connected wheels). In the cases where the loads of actually used cargo handling equipment can be precisely set, this table may not be used.

③ The values for mobile cranes in **Table 9.18.2** are determined in consideration of the relations between lifting capacity and maximum outrigger loads as well as between maximum outrigger loads and contact areas per outrigger with reference to the actual performance of existing mobile cranes of leading producers, where 95% confidence values and averages are used for the outrigger loads and contact areas, respectively. The ground contact pressure in the table means the values obtained by dividing outrigger loads by contact areas. The values for folk lift trucks are determined in consideration of the relations between loading capacity and maximum loads per wheel as well as between maximum loads per wheel and ground contact pressure, where the maximum loads, contact areas, and ground contact pressure are set in the same ways as is the case for outrigger loads. The values for trucks, tractor trailers, and straddle carriers are determined in accordance with actual performance values of respective machines.

Table 9.18.2 Characteristic Values of the Actions Considered in the Performance Verification of Apron Pavements

Type of action (cargo handling equipment load)		Maximum load of an outrigger or a wheel (kN)	Ground contact area of an outrigger or a wheel (cm^2)	Ground contact pressure (N/cm^2)
Mobile cranes	Type 20	220	1,250	176
Truck crane	Type 25	260	1,300	200
Rough terrain crane	Type 30	310	1,400	221
All terrain crane	Type 40	390	1,650	236
	Type 50	470	1,900	247
	Type 80	690	2,550	271
	Type 100	830	3,000	277
	Type 120	970	3,350	290
	Type 150	1170	3,900	300
Truck	25 ton class	100	1,000	100
Tractor trailer	For 20 ft	50	1,000	50
	For 40 ft	50	1,000	50
Fork lift truck	2 ton	25	350	71
	3.5 ton	45	600	75
	6 ton	75	1,000	75
	10 ton	125	1,550	81
	15 ton	185	2,250	82
	20 ton	245	2,950	83
	25 ton	305	3,600	85
	35 ton	425	4,950	86
Straddle carrier		125	1,550	81

(4) Performance verification for concrete pavements

① **Compositions of concrete pavements**

As shown in **Fig. 9.18.2**, concrete pavements generally have a cross-sectional structure where a base course and a concrete slab are arranged on subgrade. A base course and a concrete slab are collectively called a pavement.

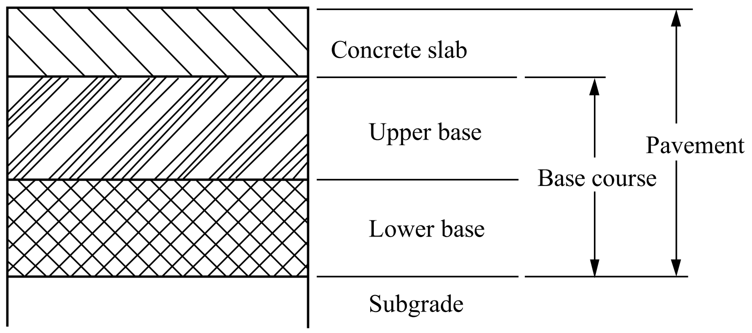


Fig. 9.18.2 Composition of Concrete Pavement

② **Procedures of performance verification**

(a) **Fig. 9.18.3** shows an example of the procedures of the performance verification for concrete pavements.

(b) It is preferable that the performance verification of concrete pavements is carried out for the thicknesses of base courses and concrete slabs in consideration of action conditions, the number of repetitions of actions, and the conditions of bearing capacity of subgrade.

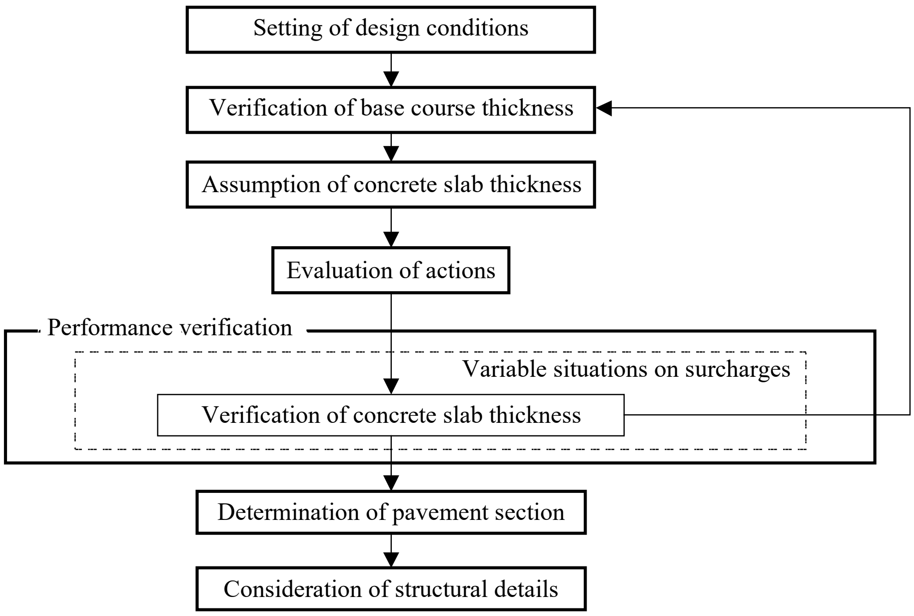


Fig. 9.18.3 Example of the Procedures of Performance Verification for Concrete Pavements

③ **Design conditions**

(a) The general design conditions to be considered in the performance verification are as follows:

1) design working life;

2) action conditions;

3) the number of repetitions of actions;

4) bearing capacity of subgrade; and

5) materials used.

(b) Design working life

The design working life of concrete pavements shall be properly set considering the conditions of use and other related conditions of mooring facilities. The design working life of concrete pavements used for the aprons of quaywalls and other facilities may be generally set at 20 years.

(c) Action conditions

The design action conditions are those requiring the maximum concrete slab thicknesses among the types of actions to be considered. The characteristic values of actions may be set referring to Table 9.18.3. The "Action classification" in Table 9.18.3 is the classification needed when using the empirical method in **Chapter 5, 9.18.3, (4) ④ (d) Empirical method of setting concrete slab thickness**.

Table 9.18.3 Reference Values for the Action Conditions of Concrete Pavements used for the Aprons of Quaywalls and Other Facilities

Action classification	Type of action		Action (kN)	Ground contact radius (cm)
CP_1	Fork lift truck	2 t	25	10.6
	Tractor trailer	for 20 ft, 40 ft	50	17.8
	Fork lift truck	3.5 t	45	13.8
CP_2	Fork lift truck	6 t	75	17.8
CP_3	Truck	25 ton class	100	17.8
	Fork lift truck	10 t	125	22.2
	Straddle carrier		125	22.2
	Fork lift truck	15 t	185	26.8
CP_4	Mobile crane (truck crane, rough terrain crane, all terrain crane)	Type 20	220	19.9
	Fork lift truck	20 t	245	30.7
	Mobile crane (truck crane, rough terrain crane, all terrain crane)	Type 25	260	20.3

(d) Calculation of the number of repetitions of actions

The methods for calculating the number of repetitions of surcharges during design working life is considered to be as follows:

1) the estimation based on the actual performance of other ports with similar scales and

2) the estimation based on the cargo throughput of the port concerned.

One of the references to the method 2) for estimating the number of repetitions of surcharges based on cargo throughput[34)] is the method proposed by Nagao et al. for calculating the number of repetitions of loads for the performance verification of the superstructures of piled piers in the fatigue limit state.[35)]

(e) Bearing capacity of subgrade

In the performance verification of concrete pavements, the bearing capacity of subgrade can be set on the basis of a design bearing capacity coefficient K_{30}.

1) The design bearing capacity coefficient K_{30} of subgrade can be set in accordance with the **Method for Plate Load Test on Soil for Road (JIS A 1215)**. Here, the design bearing capacity coefficient K_{30} is generally set as the value corresponding to a settlement of 0.125 cm. It is preferable to perform the plate loading test at one or two locations per 50 m along the face line directions of quaywalls.

2) When setting the design bearing capacity coefficients K_{30} in areas of subgrade made of identical materials, it is preferable to calculate the values of K_{30} with **equation (9.18.1)** using the measured values of three or more points excluding extreme values.

Bearing capacity coefficient K_{30} of subgrade =

$$\text{The average of bearing capacity coefficients at respective locations.} - \frac{\text{The maximum value of bearing capacity coefficient} - \text{The minimum value of bearing capacity coefficient}}{C} \quad \textbf{(9.18.1)}$$

where

C : a coefficient used for calculating the bearing capacity coefficients. The values in **Table 9.18.4** may be used.

Table 9.18.4 Reference Values for Coefficient *C*

Number of test points (n)	3	4	5	6	7	8	9	10 or more
C	1.91	2.24	2.48	2.67	2.83	2.96	3.08	3.18

3) When the subgrade has already been constructed, the bearing capacity coefficients can be obtained by performing plate load tests on the subgrade when it has the highest moisture content. In the case where plate load tests cannot be performed under such a condition, the test results are subjected to correction, when calculating the bearing capacity coefficients, using **equation (9.18.2)** in which California Bearing Ratio (*CBR*) test results shall be those of undisturbed test pieces.

$$\text{Bearing capacity coefficient (corrected) of subgrade} = \text{Calculated bearing capacity coefficient using actual measured values} \times \frac{CBR \text{ (with immersion in water for 4 days)}}{CBR \text{ (with natural moisture content)}} \quad \textbf{(9.18.2)}$$

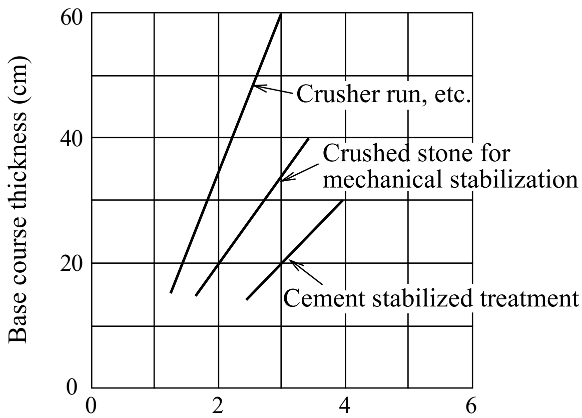
(f) Materials used

The requirements for the quality and grain sizes of respective base course materials can follow the provisions in the **Pavement Design and Construction Guide**.[30)]

④ **Performance verification**

(a) Verification of base course thicknesses

1) It is preferable to obtain the thicknesses of base courses in a manner that prepares test base courses and identifies the thicknesses that achieve the bearing capacity coefficient of 200 N/cm^3. In the cases where the preparation of test base courses is difficult, the base course thicknesses may be directly set using the design curves shown in **Fig. 9.18.4.** The minimum base course thickness is generally set at 15 cm.



$$\frac{\text{Bearing capacity coefficient of base course}}{\text{Design bearing capacity coefficient of subgrade}} = \frac{K_1}{K_2}$$

K_1 is the bearing capacity coefficient of base course K_{30} (200 N/cm^3)

K_2 is the bearing capacity coefficient of subgrade K_{30}

Fig. 9.18.4 Design Curves for Base Course Thicknesses[30)]

2) The base course thicknesses of concrete pavements may be set referring to **Table 9.18.5** that is prepared on the basis of the past records.

Table 9.18.5 Reference Values for Base Course Thicknesses of Concrete Pavements

Design condition	Base course thickness (cm)				
Design bearing capacity coefficient of subgrade K_{30} (N/cm^3)	Upper base		Lower base		Total base course thickness
	Cement stabilized base	Mechanical stabilized material	Mechanical stabilized material	Crusher run, etc.	
50 or more and less than 70	–	40	–	20	60
	20	–	20	–	40
	25	–	-	30	55
70 or more and less than 100	–	20	15	-	35
	–	20	–	20	40
	15	–	15	-	30
	15	–	–	15	30
100 or more	-	20	–	–	20
	15	–	–	–	15

(b) Verification of concrete slab thicknesses

1) Bending strength of concrete slabs

The bending strength of concrete slabs may be generally set at 450 N/cm^2 on the basis of specimens with the age of 28 days. The strength can be set with reference to the **Method of Making and Curing Concrete Specimens (JIS A 1132)** and the **Method of Test for Bending Strength of Concrete (JIS A 1106)**. Using concrete with bending strength enhanced by lowering water–cement ratios is one of the methods to minimize the increase in concrete slab thicknesses. However, it is necessary to pay attention to the fact that lower water–cement ratios may cause the reduction in workability and the increase in the risk of drying shrinkage cracks.

2) **Fig. 9.18.5** shows the relation between concrete slab thicknesses and bending stresses. The bending stresses are calculated using an **equation (9.18.3)** called Picket's formula or Arlington formula. The partial factor in the equation is set at 1.0. The symbols CP_1 to CP_4 in **Fig. 9.18.5** are the classification needed when using the empirical method in **Part II, Chapter 5, 9.18.3, (4) ④ (d) Empirical method of setting concrete slab thickness**.

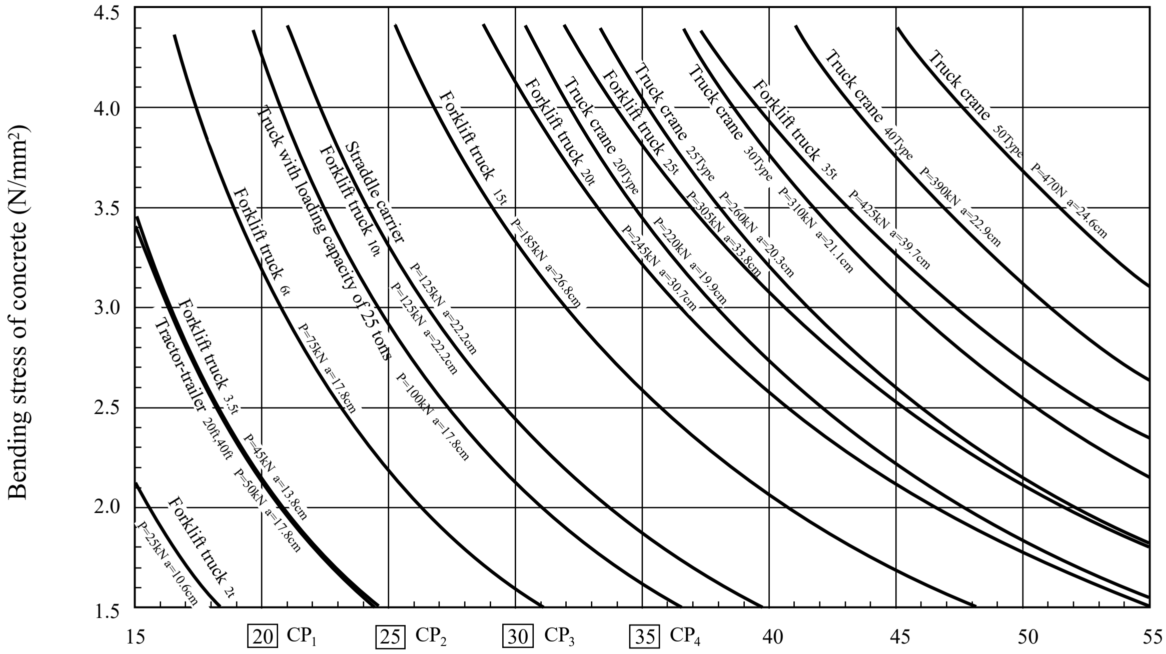


Fig. 9.18.5 Relation between Concrete Slab Thickness and Bending Stress

$$\sigma = \frac{10CP}{h^2}\left(1 - \frac{\sqrt{\dfrac{a}{l}}}{0.925 + 0.22\dfrac{a}{l}}\right) \tag{9.18.3}$$

where

σ : the maximum stress at a right angle corner of a concrete slab (N/mm²);

C : a coefficient that can be set at 3.36 when using dowel bars;

P : a weight of a cargo handling machine (kN);

h : a thickness of a concrete slab (cm);

a : a radius of equivalent contact area of the cargo handling machine (cm);

l : a radius of relative stiffness (cm); $l = \sqrt[4]{\dfrac{Eh^3}{12(1-\nu^2)K_{75}}}$

E : the modulus of elasticity of concrete (N/cm²) that can be normally set at 3,500,000 N/cm²;

ν : the Poisson ratio of concrete that can be normally set at 0.15; and

K_{75} : the design bearing capacity coefficient of a base course (N/cm³) that is normally set at $K_{75} = K_{30}/2.8 = 200/2.8 \approx 70$ N/cm³ on the basis of $K_{30}/K_{75} = 2.8$.

(c) Setting of concrete slab thicknesses

The method of setting the thicknesses of concrete slabs in compliance with the concept of the **Pavement Design and Construction Guide**[30] has been proposed.[34] In this method, the fatigue characteristics of concrete slabs are calculated on the basis of the wheel load stresses imposed on concrete slabs and the number of repetitions of the stresses during design working life. And the relation between the above-mentioned characteristics and the degree of fatigue as a failure criterion is proposed to set the thicknesses of concrete slabs. An outline of the method is shown below:

1) The allowable number of repetitions of wheel load stresses is calculated by the fatigue **equation (9.18.4)**.

$$N_i = 10^{\{(1.000-SL)/0.044\}} \qquad 1.0 \geq SL > 0.9$$
$$N_i = 10^{\{(1.077-SL)/0.077\}} \qquad 0.9 \geq SL > 0.8 \qquad \textbf{(9.18.4)}$$
$$N_i = 10^{\{(1.224-SL)/0.118\}} \qquad 0.8 \geq SL$$

where

N_i : an allowable number of wheel load stress i imposed on concrete slab; and

SL: wheel load stress/design reference bending strength (= 450 N/cm^2) with the value of the wheel load stress calculated from **equation (9.18.3)**.

2) Calculation of the degrees of fatigue

The degree of fatigue of a concrete slab is calculated from **equation (9.18.5)**.

$$FD = \sum \left(\frac{n_i}{N_i} \right) \qquad \textbf{(9.18.5)}$$

where

FD : a degree of fatigue;

n_i : the number of repetition of wheel load i; and

N_i : the allowable number of repetition of a wheel load i imposed on a concrete slab.

3) Setting of concrete slab thicknesses

Using the degree of fatigue as the failure criterion of concrete slabs, concrete slab thicknesses are set so that the degree of fatigue FD is equal to 1.0 or less.

(d) Empirical method of setting concrete slab thicknesses

1) The concrete slab thicknesses set referring to the empirical values given in **Table 9.18.6** may be considered to have the same performance as the one set using the method stipulated in **Part II, Chapter 5, 9.18.3 (4) ④ (c)** Setting Concrete Slab Thickness. However, the thicknesses are preferably set in accordance with the method stipulated in **Part II, Chapter 5, 9.18.3 (4) ④ (c) Setting Concrete Slab Thickness** in the case of the performance verification of the concrete pavements subjected to large cargo handling machines.

Table 9.18.6 Reference Values for Concrete Slab Thickness

Action classification	Concrete slab thickness (cm)
CP_1	20
CP_2	25
CP_3	30
CP_4	35
Applied to piled pier slab	10

2) The "Action classification" in **Table 9.18.6** corresponds to the one given in **Table 9.18.3**. It should be noted, in classifying actions, that there are cases where the maximum loads are not equivalent to the values shown in **Table 9.18.2**. In such cases, the classifications with the values closest to and larger than the ones in **Table 9.18.2** are used. For example, a truck crane with the maximum load per outrigger of 120 kN and a forklift truck with the maximum load per wheel of 64 kN can be considered as the actions of type 20 truck crane and 6 ton fork lift truck, respectively.

3) For the loads plotted at the right side of a curve of type 25 truck crane in **Fig. 9.18.5**, it is preferable to separately carry out the performance verification of the concrete slab thicknesses using, for example, **equation (9.18.3)**.

4) When setting concrete slab thicknesses, for design loads exceeding CP_4, with reference to the values in **Table 9.18.6**, it is preferable to study the possibility of using prestressed concrete pavements or continuous reinforced concrete pavements because such large design loads significantly increase the slab thicknesses of non-reinforced concrete pavements. When using cranes such as truck cranes having larger ground contact pressure than other cargo handling machines on aprons, it is also preferable to lay iron plates or the like under outriggers to reduce the ground contact pressure per unit area.

⑤ **Structural details**

(a) Frost heave prevention layers

In the cold regions where pavements are subjected to freezing and thawing, it is necessary that pavements are provided with frost heave prevention layers in the cases of pavement thicknesses less than frost penetration depths.

(b) Wire meshes

1) It is effective to place wire meshes in concrete slabs in terms of crack prevention. It is preferable that wire meshes are made of deformed steel bars with a diameter of 6 mm and arranged in concrete slabs at a rate of about 3 kg/m^2.

2) It is preferable to overlap wire meshes at their joints and to properly set overlap lengths and the depths of wire meshes from the surfaces of concrete slabs depending on the thicknesses of concrete slabs.

(c) Joints

It is preferable that concrete pavements are provided with joints that allow concrete slabs to expand, shrink, and warp freely to some extent and thereby reduce stresses.

1) Joints are classified into several types depending on directions and purposes. Longitudinal and transverse joints are the joints installed parallel to and perpendicular to construction directions, respectively. Construction joints are installed for construction purposes such as concrete placement and temporary suspension of construction work. Contraction joints are installed to prevent contraction cracks in a manner that reduces tensile stresses in concrete slabs by allowing the joints to absorb the contraction deformation of concrete slabs due to temperature reductions and drying. Expansion joints are installed to prevent blowup (upward movement of concrete slabs) in a manner that reduces compressive stresses in concrete slabs by allowing the joints to absorb the dilation deformation of concrete slabs due to temperature increases.

2) Joints shall be installed at appropriate locations in accordance with the sizes of aprons, structures of mooring facilities, the types of joints, and loading conditions and have appropriate structures suitable for the types of joints. In contrast, it is necessary to minimize the number of joints because they are structural weak points in concrete pavements and require complex construction procedures that cause construction and maintenance costs to be increased. Considering that joints are to be installed for allowing concrete slabs to freely deform, it is preferable that joints of concrete slabs close to each other are aligned to the extent possible. When intricately arranged as is the case with misaligned transverse contraction joints installed on concrete slabs arranged in rows with a longitudinal joint in between, joints may constrain free deformation of concrete slabs.

3) Longitudinal joints

i. Longitudinal construction joints:

Longitudinal construction joints shall generally have a press-type structure with tie bars (refer to **Part II, Chapter 5, 9.18.3 (4) ⑤ (d) Tie bars and dowel bars**). Tie bars are, however, not used for the pavements on piled pier slabs. It is preferable to set longitudinal joints at proper intervals, less than 5 m in many cases, depending on paving machines used, total pavement widths, and traveling crane rails.

ii. Longitudinal expansion joints:

Longitudinal expansion joints generally have a structure comprising a joint-sealing compound (upper) and a joint filler (lower) with dowel bars (refer to **Part II, Chapter 5, 9.18.3 (4) ⑤ (d) Tie bars and dowel bars**). Dowel bars are, however, not used for the pavements close to the superstructures of quaywalls and sheds as well as on piled pier slabs. It is preferable to place longitudinal expansion joints on the shoulder of backfill, the joints of quaywalls, and the position of sheet-pile anchorages to reduce the effects of structural boundaries below base courses, differences in bearing capacity, and the joints of quaywalls on concrete pavements.

4) Transverse joints

i. Transverse contraction joints:

Transverse contraction joints shall generally have a dummy-type structure with dowel bars. On piled pier slabs, however, dowel bars are not used. It is preferable to set transverse contraction joints at proper intervals, less than 5 m in many cases, depending on construction conditions. It is also preferable for shrinkage joints to be installed on the joints of quaywalls.

ii. Transverse construction joints:

Transverse construction joints shall generally have a press-type structure with dowel bars. On piled pier slabs, however, dowel bars are not used. Transverse construction joints are installed at the end of daily construction work or inevitably installed due to rain during construction or the failures of construction machines or other equipment. It is preferable that the positions of transverse construction joints coincide with those of transverse contraction joints.

iii. Transverse expansion joints:

It is preferable that transverse expansion joints generally have a structure comprising a joint-sealing compound (upper) and a joint filler (lower) with dowel bars. For the joints close to the superstructures of quaywalls and sheds as well as on piled pier slabs, however, dowel bars are not used. It is also preferable to set transverse expansion joints at proper intervals, normally 100 to 200 m when constructed in summer and 50 to 100 m in winter, depending on construction conditions. Because expansion joints are the weakest points in pavements, consideration is needed for minimizing their installation locations.

5) Joint structures

Figs. 9.18.6 to **9.18.9** show standard joint structures.

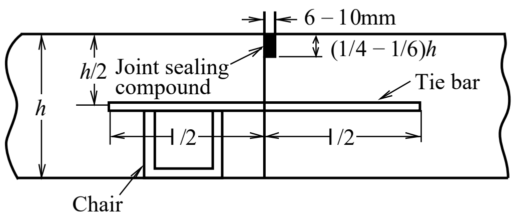


Fig. 9.18.6 Longitudinal Construction Joint

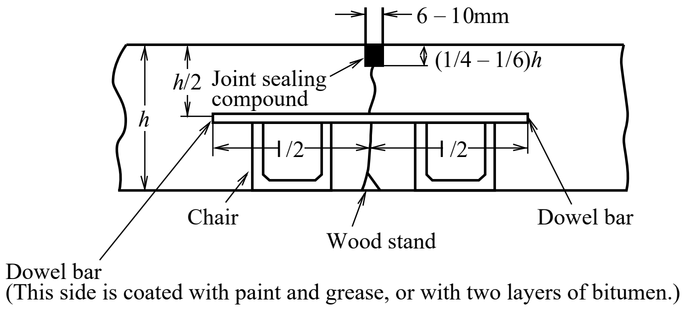


Fig. 9.18.7 Transverse Contraction Joint

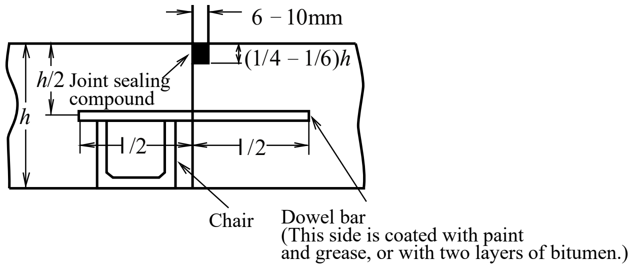


Fig. 9.18.8 Transverse Construction Joint

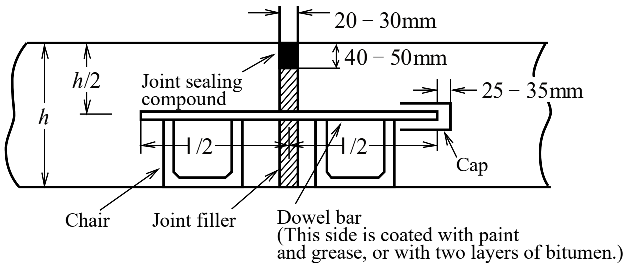


Fig. 9.18.9 Transverse Expansion Joint

(d) Tie bars and dowel bars

1) Tie bars are installed to prevent slabs next to each other from being separated or having differences in level. They are also capable of transmitting loads between slabs. Apron pavements having relatively narrow widths with both ends confined by structures such as quaywalls and sheds have rarely caused concrete slabs to be separated, but tie bars are still used for longitudinal construction joints to enable concrete slabs to cope with uneven settlement of the layers below base courses and traffic loads acting on them in diverse directions unlike in the case of traffic loads on general roads.

2) Dowel bars have function to transfer loads without restricting relative movement of concrete slabs next to each other in the direction perpendicular to joints and to prevent concrete slabs from having differences in level. Generally, dowel bars are used for transverse contraction, construction, expansion joints as well as longitudinal expansion joint to enable loads to be sufficiently transferred.

3) Tie bars and dowel bars shall be properly selected considering the traveling loads imposed on apron pavements in all directions.

4) The specifications and placement intervals of tie bars and dowel bars may refer to the values shown in **Table 9.18.7**.

Table 9.18.7 Reference Values for the Specifications and Placement Intervals of Tie Bars and Dowel Bars

Action classification	Slab thickness (cm)	Tie bar			Dowel bar		
		Diameter (cm)	Length (cm)	Interval (cm)	Diameter (cm)	Length (cm)	Interval (cm)
CP_1	20	25	80	45	25	50	45
CP_2	25	25	100	45	25	50	45
CP_3	30	32	100	40	32	60	40
CP_4	35	32	100	40	32	60	40

Note: The values of tie bars and dowel bars are those of SD295A (deformed steel bar) specified in JIS G 3112 and of SS400 (round steel bar) specified in JIS G 3101, respectively.

(e) End protection

1) Pavements are preferably provided with end protection work at locations with risks of destruction of base courses due to infiltration of rain water or destruction of the concrete slabs and base courses due to heavy loading.

2) When apron pavements are located next to open storage yards or empty land to be paved later, the landward pavement edges are normally brought into contact with soil and, therefore, have risks of destruction of base courses due to the infiltration of rain water and the destruction of base courses and concrete slabs due to transverse traffic loads in the cases of aprons next to open storage yards. Thus, the portions of pavements with these risks shall be provided with end protection to obviate them.

3) Examples of end protection are shown in **Fig. 9.18.10**.

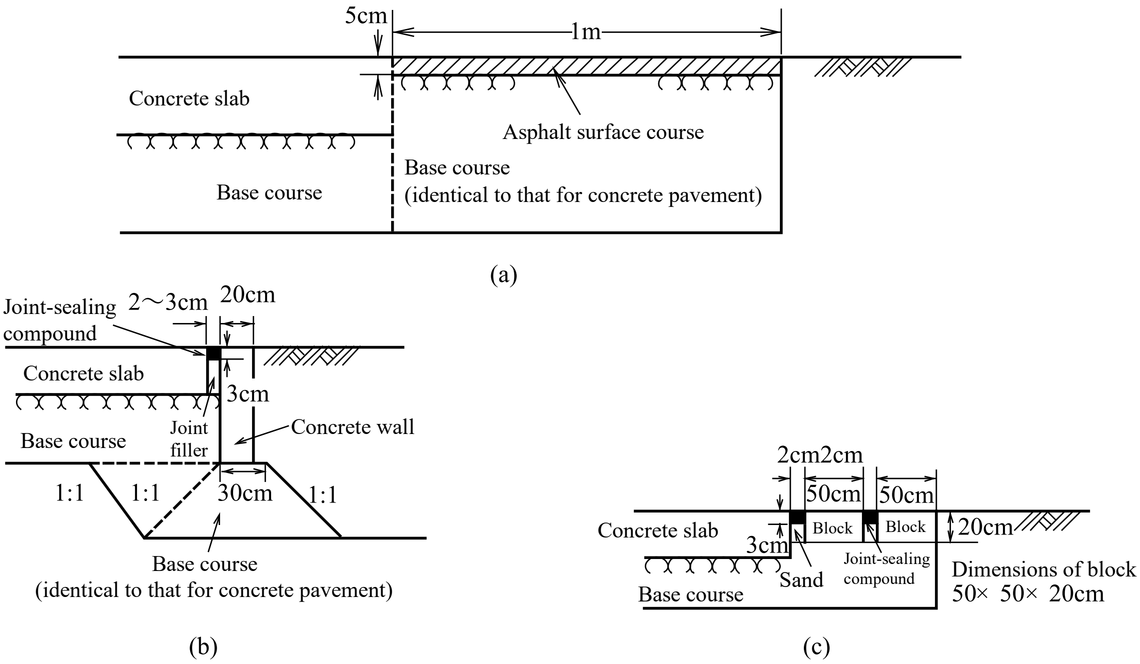


Fig. 9.18.10 Examples of End Protection

(f) Junctions with asphalt pavements

1) When the boundaries between concrete and asphalt pavements are subjected to traffic loads, it is preferable to install transition boards below asphalt pavements in order to prevent the occurrence of the differences in levels between the two types of pavements or damage to the boundaries due to the difference in their bearing capacity.

2) For the dimensions of transition boards, reference can be made to the **Manual for Pavement Design** and the **Manual for Airport Pavement Design**.

(5) Performance verification of asphalt pavements

① Compositions of asphalt pavements

As shown in **Fig. 9.18.11**, an asphalt pavement generally has a cross-sectional structure where a base course, an asphalt mixture layer (surface and binder courses), and base course (upper and lower bases) are arranged on subgrade.

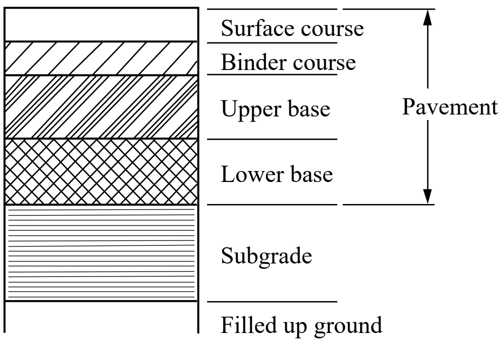


Fig. 9.18.11 Composition of Asphalt Pavements

② Procedures of performance verification

(a) **Fig. 9.18.12** shows an example of the procedures of the performance verification for asphalt pavements.

(b) The performance verification of asphalt pavements can be based on the T_A method (refer to **Part II, Chapter 5, 9.18.3 (4) ④ (a) Verification of asphalt pavement**) in which the compositions of pavements are determined so that pavement thicknesses do not fall below the layer equivalent thicknesses calculated from the bearing capacity of subgrade and the number of repetitions of actions.

(c) In the performance verification of asphalt pavements subjected to the traveling loads of large cargo handling machines, it is preferable to use a theoretical design method in which the compositions of pavements are determined by the strain to be generated in pavements and the fatigue failure frequency calculated on the basis of the multilayer elastic theory.

③ **Design conditions**

(a) The design conditions to be considered in the performance verification are generally as follows:

1) design working life;

2) action conditions;

3) the number of repetitions of actions;

4) bearing capacity of subgrade; and

5) materials used.

(b) Design working life

The design working life of asphalt pavements shall be properly set considering the usage conditions of mooring facilities. The design working life of asphalt pavements used for the aprons of quaywalls may be generally set at 10 years.

(c) Action conditions

Action conditions shall be those, among the kinds of subject actions, which maximize asphalt pavement thicknesses.

(d) Calculation of the number of repetitions of actions

For the calculation of the numbers of repetitions of actions, reference can be made to **Part II, Chapter 5, 9.18.3 (4) ③ (d) Calculation of the number of repetitions of actions**.

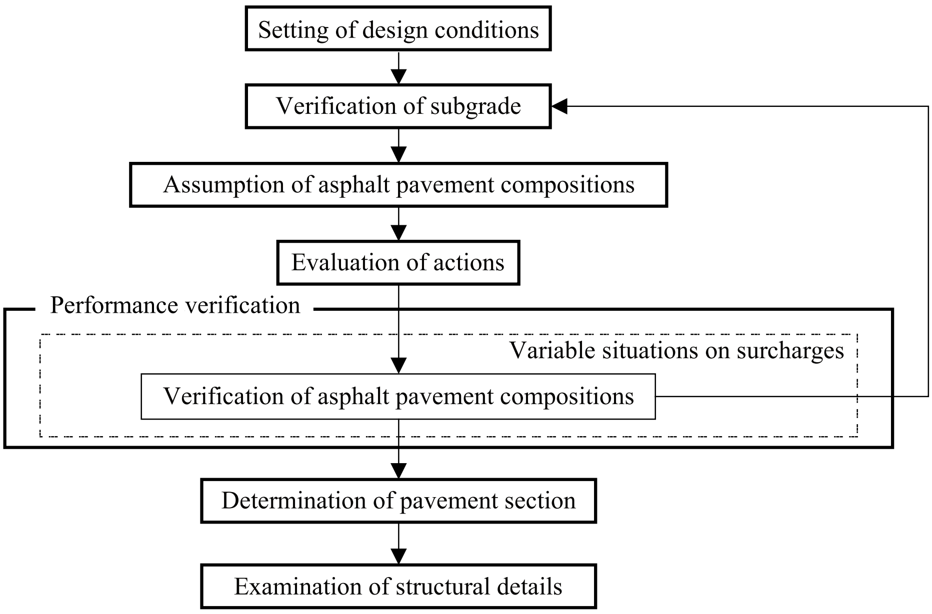


Fig. 9.18.12 Example of Procedures of Performance Verification for Asphalt Pavements

(e) Bearing capacity of subgrade

When obtaining design *CBR*s for the subgrade of pavement sections subjected to the performance verification, CBR tests shall be carried out in a manner that measures CBRs after tamping down subgrade soil containing natural moisture and immersing it in water for 4 days in compliance with the **Test Methods for the CBR of Soils in Laboratory (JIS A 1211)**. In the CBR tests, sampled subgrade soil from which aggregates not less than 40 mm are separated shall be put in molds in three layers with each layer subject to tamping 67 times. In the cases where subgrade is already constructed, design *CBR*s shall be obtained generally through on-site CBR tests with subgrade soil in the wettest condition. If on-site CBR tests cannot be carried out in the ideal wet conditions, design *CBR*s can be obtained by correcting the on-site CBR test results using **equation (9.18.6)**. Here, it should be noted that *CBR*s are applicable to undisturbed soil samples. It is preferable that soil is sampled from one to two locations in every 50 m along the face lines of quaywalls at the depths of 50 cm or more from the surfaces of completed subgrade or exposed surfaces of borrow pits.

$$CBR\ \text{(corrected)} = \text{In-situ}\ CBR \cdot \frac{CBR\ \text{(with immersion in water for 4 days)}}{CBR\ \text{(with natural moisture content)}} \qquad \textbf{(9.18.6)}$$

Design *CBR*s can be obtained by **equation (9.18.7)** using the above-defined *CBR*s excluding extreme values.

$$\text{Design}\ CBR = \begin{array}{l}\text{The average of CBRs}\\ \text{at respective locations}\end{array} - \frac{\begin{array}{l}\text{The maximum}\\ \text{value of CBR}\end{array} - \begin{array}{l}\text{The minimum}\\ \text{value of CBR}\end{array}}{C} \qquad \textbf{(9.18.7)}$$

where C is given in **Table 9.18.4**.

(f) Material used

1) Base course

The requirements for the items related to the quality and grain sizes of base course materials can follow the provisions in the **Pavement Design and Construction Guide**.[30)]

2) Surface and binder courses

i. It is preferable that polymer-modified asphalt (Modification Type II, Type III or special asphalt with dynamic stability equivalent to semi-flexible mixtures) is used for the surface and binder courses in the areas with possible actions of heavy or static loads; a risk of early development of rutting on the basis of previous experience; or the possibility that excessively large rutting pose problems with apron operation.

ii. There may be the cases where pavements of sorting facilities have deep cracks or dents due to locally applied impact loads during container handling and aged deterioration. Because these deep cracks and dents have a risk of being fertile breeding grounds for alien species transported together with containers and the like, it is preferable that these cracks and dents are properly repaired and that pavement design considers local reinforcement measures in accordance with use conditions of sorting facilities, for example, concrete slabs or steel plates laid on the places where containers are planned to be stored.

④ **Performance verification**

(a) Verification of asphalt pavement compositions

1) Setting of pavement sections

Pavement compositions shall be determined so that the layer equivalent thicknesses of set pavement cross sections do not fall below the required layer equivalent thicknesses.

2) Required layer equivalent thicknesses

The required layer equivalent thicknesses T_A can be calculated by **equation (9.18.8)**.

$$T_A = \frac{3.84 N^{0.16}}{CBR^{0.3}} \qquad \textbf{(9.18.8)}$$

where

T_A : a required layer equivalent thickness (cm); and

N : the value obtained by converting the number of repetition of action during design working life n_i to 49 kN wheel load using the following equation.

$$N = \sum_{i=1}^{m}\left[\left(\frac{P_i}{49}\right)^4 n_i\right] \tag{9.18.9}$$

where

P_i : a wheel load (kN);

n_i : the number of repetitions of wheel load P_i; and

m : the number of loaded states.

3) Layer equivalent thicknesses of assumed sections

The layer equivalent thicknesses T_A' of assumed cross sections can be calculated by **equation (9.18.10)**.

$$T_A' = \sum_{i=1}^{n}\left[a_i h_i\right] \tag{9.18.10}$$

where

T_A' : a layer equivalent thickness of an assumed section (cm);

h_i : a thickness of layer i (cm);

a_i : a layer equivalent value set for material and work method used for each pavement layer (refer to **Table 9.18.8**); and

n : the number of layers.

Table 9.18.8 Equivalent Conversion Factors of Asphalt Pavements

Layer	Construction method/material	Requirement	Layer equivalent value	Remark
Surface and binder courses	Hot asphalt mixture for surface and binder courses	–	1.00	AC I–AC IV
Upper base	Bituminous stabilization	Marshall stability 3.43 kN or greater	0.80	A-treated material II
		Marshall stability 2.45 to 3.43 kN	0.55	A-treated material I
	Hydraulic mechanically stabilizing steel slag	Modified CBR 80 or greater Unconfined compressive strength (14 days) 1.2 MPa	0.55	
	Mechanical stabilized material	Modified CBR 80 or greater	0.35	Mechanical stabilized material
Lower base	Crusher run, slag, sand, etc.	Modified CBR 30 or greater	0.25	
		Modified CBR 20 to 30	0.20	Grain material

(b) Example of empirical verification of asphalt pavement compositions

Table 9.18.10 shows an example of the empirical verification of asphalt pavement compositions. The table is prepared referring to the action conditions shown in **Table 9.18.9**. The symbols H and T_A' in **Table 9.18.10** express total pavement thickness and the equivalent conversion asphalt pavement thickness of the assumed section, respectively. If the design CBR of a subgrade is 2 or more and less than 3, it is preferable to replace it with one using good quality materials or to add a water-sealing layer. If it is less than 2, it is preferable to replace it with good quality materials and set the pavement thickness once again.

(c) The type and material quality of asphalt mixtures can be set as listed in **Table 9.18.11**.

Table 9.18.9 Reference Values for Action Conditions for Asphalt Pavements on Quaywall Aprons

Action classification	Cargo handling machine	
AP_1	Tractor trailer	20 ft, 40 ft
AP_2	Fork lift truck	2 t
	Fork lift truck	3.5 t
	Fork lift truck	6 t
AP_3	Fork lift truck	10 t
	Fork lift truck	15 t
	Truck	25 ton class
	Straddle carrier	
	Mobile crane (truck crane, rough terrain crane, all terrain crane)	Type 20
AP_4	Mobile crane (truck crane, rough terrain crane, all terrain crane)	Type 25

Table 9.18.10 Examples of Compositions of Asphalt Pavement

Condition of actions		Pavement composition								
Classification of actions	Design *CBR* of subgrade (%)	Surface course		Binder course		Upper base		Lower base	Total thickness	
		Type	h_1 (cm)	Type	h_2 (cm)	Type	h_3 (cm)	h_4 (cm)	H (cm)	T_A' (cm)
AP$_1$	Equal to or above 3 and less than 5	AC I	5	AC III	5	Mechanical stabilized material	25	35	70	25.8
		AC I	5	–	–	A-treated material I	25	35	65	25.8
	Equal to or above 5 and less than 8	AC I	5	AC III	5	Mechanical stabilized material	20	25	55	22.0
		AC I	5	–	–	A-treated material I	20	30	55	22.0
	Equal to or above 8 and less than 12	AC I	5	AC III	5	Mechanical stabilized material	15	20	45	19.3
		AC I	5	–	–	A-treated material I	15	30	50	19.3
	Equal to or above 12 and less than 20	AC I	5	AC III	5	Mechanical stabilized material	15	15	40	18.3
		AC I	5	–	–	A-treated material I	15	20	40	17.3
	Equal to or above 20	AC I	5	AC III	5	Mechanical stabilized material	15	15	40	18.3
		AC I	5	–	–	A-treated material I	15	15	35	16.3
	On the deck slab of piled pier	AC I	5	AC III	4 or greater	–	–	–	9 or greater	–
AP$_2$	Equal to or above 3 and less than 5	AC II	5	AC IV	5	Mechanical stabilized material	25	35	70	25.8
		AC II	5	–	–	A-treated material I	25	35	65	25.8
	Equal to or above 5 and less than 8	AC II	5	AC IV	5	Mechanical stabilized material	20	25	55	22.0
		AC II	5	–	–	A-treated material I	20	30	55	22.0
	Equal to or above 8 and less than 12	AC II	5	AC IV	5	Mechanical stabilized material	15	20	45	19.3
		AC II	5	–	–	A-treated material I	15	30	50	19.3
	Equal to or above 12 and less than 20	AC II	5	AC IV	5	Mechanical stabilized material	15	15	40	18.3
		AC II	5	–	–	A-treated material I	15	20	40	17.3
	Equal to or above 20	AC II	5	AC IV	5	Mechanical stabilized material	15	15	40	18.3
		AC II	5	–	–	A-treated material I	15	15	35	16.3
	On the deck slab of piled pier	AC II	5	AC IV	4 or greater	–	–	–	9 or greater	–
AP$_3$	Equal to or above 3 and less than 5	AC II	5	AC IV	15	Mechanical stabilized material	30	45	95	40.0
		AC II	5	AC IV	10	A-treated material II	20	45	80	40.0
	Equal to or above 5 and less than 8	AC II	5	AC IV	15	Mechanical stabilized material	25	30	75	34.8
		AC II	5	AC IV	10	A-treated material II	20	20	55	35.0
	Equal to or above 8 and less than 12	AC II	5	AC IV	15	Mechanical stabilized material	15	20	55	29.3
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	Equal to or above 12 and less than 20	AC II	5	AC IV	15	Mechanical stabilized material	15	15	50	28.3
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	Equal to or above 20	AC II	5	AC IV	15	Mechanical stabilized material	15	15	50	28.3
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	On the deck slab of piled pier	AC II	5	AC IV	4 or greater	–	–	–	9 or greater	–

Condition of actions		Pavement composition								
Classification of actions	Design *CBR* of subgrade (%)	Surface course		Binder course		Upper base		Lower base	Total thickness	
		Type	h_1 (cm)	Type	h_2 (cm)	Type	h_3 (cm)	h_4 (cm)	H (cm)	T_A' (cm)
AP_4	Equal to or above 3 and less than 5	AC II	5	AC IV	15	Mechanical stabilized material	40	60	120	46.0
		AC II	5	AC IV	10	A-treated material II	20	70	105	45.0
	Equal to or above 5 and less than 8	AC II	5	AC IV	15	Mechanical stabilized material	30	45	95	39.5
		AC II	5	AC IV	10	A-treated material II	20	40	75	39.0
	Equal to or above 8 and less than 12	AC II	5	AC IV	15	Mechanical stabilized material	25	30	75	34.8
		AC II	5	AC IV	10	A-treated material II	15	35	65	34.0
	Equal to or above 12 and less than 20	AC II	5	AC IV	15	Mechanical stabilized material	15	25	60	303
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	Equal to or above 20	AC II	5	AC IV	15	Mechanical stabilized material	15	15	50	28.3
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	On the deck slab of piled pier	AC II	5	AC IV	4 or greater	–	–	–	9 or greater	–

Note: In the case of the deck slab of piled pier, the boxes for the binder course in Table 9.18.10 refer to the values for binder courses including filling materials that are not limited to asphalt mixtures.

Table 9.18.11 Types and Quality of Asphalt Mixtures

Type	AC I	AC II	AC III	AC IV
Use	For surface course		For binder course	
Number of blows for Marshall stability test	50 times	75 times	50 times	75 times
Marshall stability (kN)	4.9 or greater	8.8 or greater	4.9 or greater	8.8 or greater
Flow value (1/100 cm)	20–40	20–40	15–40	15–40
Air void (%)	3–5	2–5	3–6	3–6
Degree of saturation (%)	75–85	75–85	65–80	65–85

Note: The number of blows of 75 times for Marshall stability test is applied to the cases where the ground contact pressure is not less than 70 N/cm^2 or rutting is expected because of particularly heavy traffic of large vehicles.

⑤ **Structural details**

(a) In the cold regions where pavements are subjected to freezing and thawing, it is necessary that pavements are provided with frost heave prevention layers in the cases of pavement thicknesses less than frost penetration depths.

(b) For end protection, reference can be made to **Part II, Chapter 5, 9.18.3 (4) ⑤ (e) End protection**.

⑥ **Semi-flexible pavements**

(a) Semi-flexible pavements mostly have the same pavement compositions as asphalt pavements with semi-flexible mixtures used for the surface course. When increasing the thickness of semi-flexible mixture layers, it is necessary to ensure that cement milk fully infiltrates the layers from their surfaces to their bottoms.

(b) The design working life of semi-flexible pavements shall be properly set considering the usage conditions of mooring facilities. The design working life of semi-flexible pavements used for the aprons of quaywalls may be generally set at 10 years.

(c) Semi-flexible pavements are subjected to fine cracks on their surfaces due to the drying shrinkage when cement milk hardens and the shrinkage associated with the fluctuation of external temperature.[36)] These cracks do not pose structural problems unless they penetrate semi-flexible mixture layers. Thus, application of sealing material shall be implemented with attention to the prevention of rainwater from infiltrating the layers through cracks. Also, the boundaries of semi-flexible pavements having different bearing capacity may have early development of cracks due to the difference in the levels of deformation when subjected to traveling loads. Thus, it is preferable that joints are installed along such boundaries in a manner that cuts pavements after they harden.[37)]

9.19 Foundations for Cargo Handling Equipment

[Public Notice] (Perform–ance Criteria of the Foundations for Cargo Handling Equipment)

Article 74

1 The performance criteria of the foundations for cargo handling equipment shall be as prescribed respectively in the following items in consideration of the types of cargo handling equipment and the structural types of foundations:

(1) The foundations shall have the dimensions necessary for enabling the safe and smooth operation of cargo handling tasks, transportation of cargo handling equipment.

(2) The foundations shall satisfy the following criteria under the variable situation, in which the dominating actions are Level 1 earthquake ground motion and surcharge load:

(a) For pile-type structures, the risk that the axial force acting on a pile may exceed the resistance force with which underground failure occurs shall be equal to or less than the threshold level.

(b) For pile-type structures, the risk that the stress in a pile may exceed its yield stress shall be equal to or less than the threshold level.

(c) The risk of impairing the integrity of beam members shall be equal to or less than the threshold level.

(d) For pile-less structures, the risk of beam sliding shall be equal to or less than the threshold level.

(3) The beam deflection shall be equal to or less than the threshold level under the variable situation in which the dominating action is surcharge load.

2 In addition to the provisions of the preceding paragraph, the performance criteria for the foundations for the cargo handling equipment to be installed in high earthquake-resistance facilities shall be such that the degree of damage under the accidental situation, in which the dominating action is Level 2 earthquake ground motion, is equal to or less than the threshold level corresponding to the performance requirements.

[Interpretation]

11. Mooring Facilities

(18) The performance criteria of the foundations for cargo handling equipment (Article 33 of the Ministerial Ordinance and the interpretation related to Article 74 paragraph 1 of the Public Notice)

① **General provisions for the foundations for cargo handling equipment**

(a) The performance requirements for the foundations for cargo handling equipment shall focus on serviceability. Serviceability is defined as the appropriate setting of the dimensions of cargo handling equipment in accordance with the types of cargo handling equipment and the structural types of foundations so that the safe and smooth operation of cargo handling tasks, transportation of cargo handling equipment, etc., are ensured.

(b) In addition to the above provision, the performance requirements for the foundations for cargo handling equipment under the variable situation in which the dominating actions are Level 1 earthquake ground motion and surcharge shall focus on serviceability. Furthermore, the performance verification items and the standard indexes of the determination of the limit values against the actions are shown in **Attached Table 11-35**.

Attached Table 11-35 Performance Verification Items and Standard Indexes of the Determination of the Limit Values in Each Design State for the Structure of the Foundations for Cargo Handling Equipment (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index of the determination of the limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
33	1	2	74	1	2 a	Serviceability	Variable situation	L1 earthquake ground motion (Surcharge*3))	Self weight, earth pressure	Axial force acting on pile*1)	Ratio of bearing capacity–related action on pile-to-resistance capacity (pushing, pulling)
					2 b					Yield of pile*1)	Design yield stress
					2 c					Section failure of the beam	Design resistance value of section
					2 d					Sliding of the beam*2)	Ratio of sliding related action-to-resistance capacity
					3			Surcharge*3)	Self weight, earth pressure	Deflection of the beam	Deflection

*1): This is only applicable to structures with piles for the foundations for cargo handling equipment.
*2): This is only applicable to structures without a pile for the foundations for cargo handling equipment.
*3): This is the action exerted by cargo handling equipment on the foundations and should be properly set according to the design state.

② **The foundations for cargo handling equipment in high earthquake-resistance facilities** (the interpretation of the context of Article 33 of the Ministerial Ordinance and Article 74, Paragraph 2 of the Public Notice)

The performance requirements for the foundations for cargo handling equipment under the accidental situation in which the dominating action is Level 2 earthquake ground motion shall focus on restorability. Furthermore, the performance verification item and the standard index of the determination of the limit value against the action are shown in **Attached Table 11-36**. It should be noted that the verification item in **Attached Table 11-36** shows “Damage,” which is meant to be a comprehensive item, because these items differ in terms of the structure and structural types of the facilities concerned. The indexes to determine the limit values shall be designated appropriately for every performance verification item.

Attached Table 11-36 Performance Verification Item and Standard Index of the Determination of the Limit Value of the Foundations for Cargo Handling Equipment in High Earthquake-Resistance Facilities under an Accidental Situation

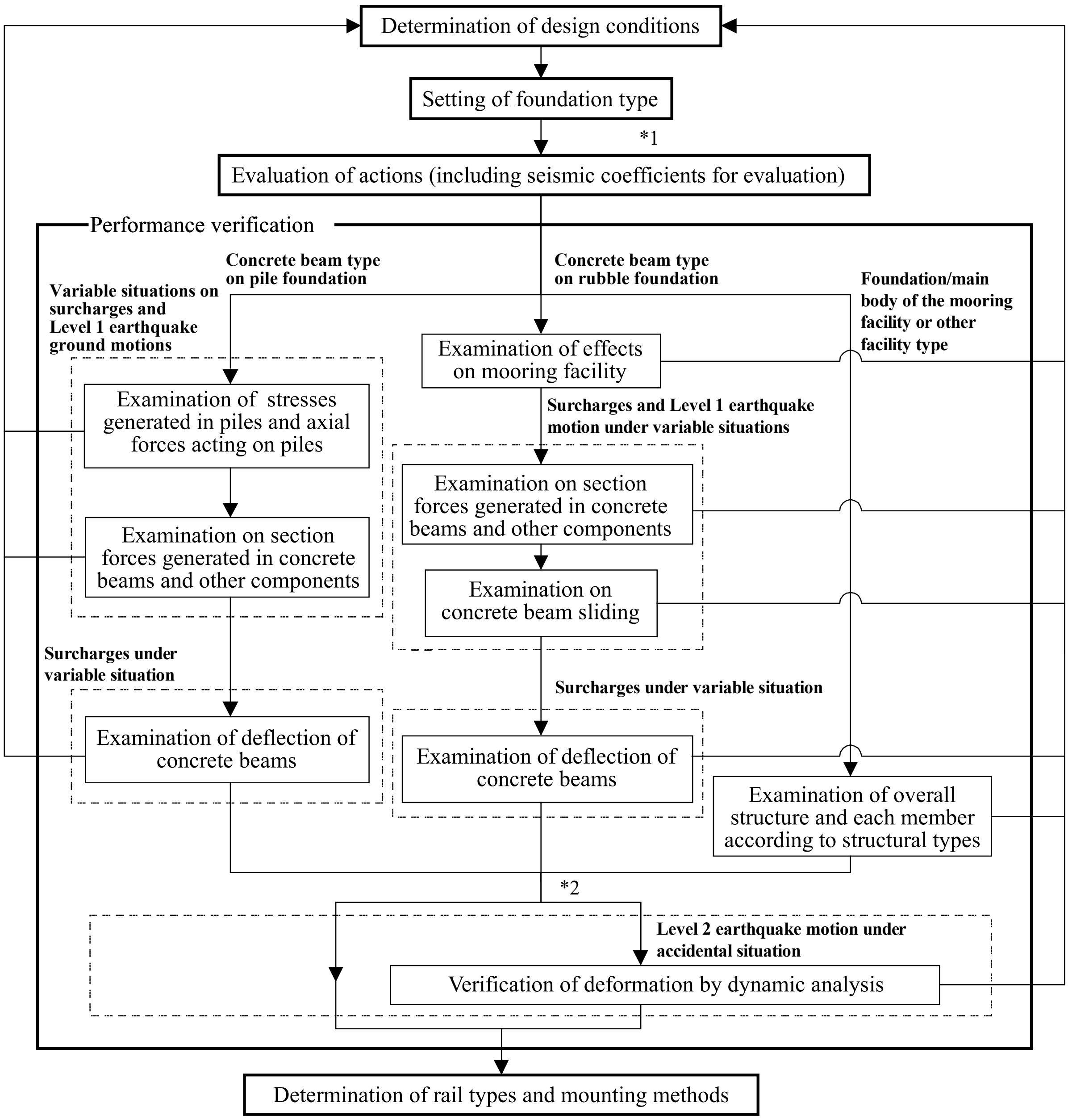
Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index of the determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
33	1	2	74	2	–	Restorability	Accidental situation	L2 earthquake ground motion	Self weight, surcharge, earth pressure	Damage	–

9.19.1 General

(1) The dimensions of the foundations for cargo handling equipment need to be properly set according to the types of cargo handling equipment and the structural types of foundations to allow the safe and smooth operation of cargo handling tasks, transportation of cargo handling equipment, etc.

(2) The foundations for rail-mounted cargo handling equipment need to be determined appropriately in consideration of the action that is exerted on them, their displacement limit value, the difficulty of their maintenance, their effect on the main body of mooring facilities, their economic value, etc.

(3) **Fig. 9.19.1** shows the procedure for the performance verification of the foundations for cargo handling equipment as an example. However, the evaluation of the effect of liquefaction that may be caused by earthquake ground motion is not shown in this figure. Therefore, the possibility of occurrence of liquefaction and the countermeasures need to be evaluated appropriately by referring to **[Part II: Actions and Material Strength Requirements], Chapter 7 Ground Liquefaction**. For the variable situation in Level 1 earthquake ground motion, the seismic coefficient method may be applied to the verification. However, in the case of high earthquake-resistance facilities, the displacement should be evaluated by using nonlinear seismic response analysis in which the dynamic interaction between the ground and the structure is taken into consideration. In addition to the foundations shown in the figure, sleepers on ballasted track bed foundation type similar to railway ballast, which is used in sites where the ground settlement is large or the wheel load of a light type crane is small, and other types are included.



*1: The evaluation of the effects of liquefaction is not included in this chart; therefore, it should be considered separately.
*2: The foundations for cargo handling equipment installed in high earthquake-resistance facilities shall be verified on Level 2 earthquake ground motion.

Fig. 9.19.1. Example of the Procedure for the Performance Verification of the Foundations for Cargo Handling Equipment

(4) Types of Rail-Mounted Foundations

① **Foundation type in which the piles are connected by reinforced concrete beams on a piled foundation**

This type of foundation is used on soft ground where differential settlement is expected. It is also used for foundations for large cargo handling equipment built on Sandy Ground in good conditions.

② **Foundation type in which the main body of mooring facilities or other facilities are utilized**

This type of foundation utilizes the main body of mooring facilities such as the reinforced concrete beams of a piled pier and the superstructure of a caisson type quaywall or the anchorage wall of a sheet pile quaywall as a foundation for cargo handling equipment; the performance verification of these facilities shall be conducted in advance by considering the action to be exerted by the cargo handling equipment. In this case, it becomes economical in terms of the facilities as a whole. When one leg is on the main body of a mooring facility and the other leg is on an independent foundation, caution is needed to avoid differential settlement. It should be noted that the action of earthquake ground motion may cause the displacement of a crane foundation, thus resulting in the displacement or derailing of the crane leg. Furthermore, a column fixed to a beam structure of a portal crane is not normally mounted on a piled pier. Considering that the end of a jetty-type piled pier is a weak spot to the actions of ship berthing force or tractive force or action by earthquake ground motion, reinforcement should be considered.

③ **Foundation type in which concrete beams are built on a rubble foundation**

This type of foundation is applicable to the ground relatively in good conditions with a slight possibility of settlement.

(5) Limit Value of Displacement of Rails

① The limit value of displacement of rails mounted on the foundations of cargo handling equipment needs to be set in with consideration to the relation between the critical displacement values of the cargo handling equipment concerned and its production costs, the precision of building the foundations, the stability and efficiency, etc., when the cargo handling equipment is in operation.

② When a relative displacement is assumed at the leg part of the equipment concerned on a gravity-type or a sheet-pile-type quaywall, it is necessary to examine the deformation performance.

③ In the performance verification and the execution of foundations, it is necessary that the critical displacement values designated for foundations by the manufacturer of the equipment concerned should be examined, a structure that ensures the lowest differential settlement should be selected, and an execution method that ensures the highest precision should be adopted.

④ The maintenance inspection of foundations for cargo handling equipment must be performed. When displacement exceeding the standard value specified for the maintenance is found, the foundation concerned must be adjusted by liner adjustment or a filling.

⑤ It should be noted that the displacement of rails increases with time; therefore, it is desirable that the execution error be made as small as possible. Although allowable displacement varies depending on manufacturer, the average laying standards and criteria are in **Table 9.19.1**.[38)]

Table 9.19.1. Example of Traveling Rails Laying Standards

Item	Installation standards	Criteria (critical values for usage)
Span	±10 mm or less for the entire rail length	±10 mm or less for the entire rail length
Lateral and vertical warps of the rail	5 mm or less per 10 m of rails	10 mm or less per 10 m of rails
Elevation difference between seaward and landward rails	1/1000 of rail span or less	1/500 of rail span or less
Gradient in the traveling direction	1/500 or less	1/250 or less
Straightness	±50 mm or less for the entire rail length	±80 mm or less for the entire rail length
Rail joints	Vertical and lateral differences: ±0.5 mm or less	Vertical and lateral differences: ±1 mm or less
	Gap: 5 mm or less	Gap: 5 mm or less
Wear of the head of the rail	–	10% or less of the original dimension

(6) The rails are fastened to the foundation structure with rail clips: Normally, 37 or 50 kg rails are used for the track structure of cranes handling sundry goods and for small cranes, whereas 73 kg crane rails are used for large unloaders handling containers and minerals. A shock absorber such as a rubber pad should be applied to avoid crack generation in concrete, which can be caused by the movement of cranes, under usage conditions because the wheel load of a crane is large and because direct contact occurs between concrete and rails or plates.

(7) When the action of earthquake ground motion is large, rocking motion is generated on a traveling-type crane, thus resulting in damage to the crane leg; therefore, consideration should be given to a seismicity.

(8) There are two types of structures for crane legs: a column fixed to beam type and a hinged column (or a rear leg) type. A hinged column is often applied to the sea side where a large horizontal force cannot be exerted on the sea side foundation (in piled pier structure, in sheet pile structure, etc.). The span of rails is very broad, thus making it difficult to meet the level of rail setting criteria. By contrast, a column fixed to a beam type is applied to the land side in the aforementioned cases, as well as to both the sea and land sides as far as there is no such restriction (all columns fixed to beam). Furthermore, the horizontal force acting on rails in a hinged column structure on the sea side is smaller than that in a column fixed to a beam structure because of the effect of a hinge system, although the horizontal force acting on rails generally becomes large as a column fixed to a beam type on the land side exerts a larger horizontal force accordingly on rails than all columns fixed to a beam type. Furthermore, these crane leg structures do not make a difference in regard to vertical force.

(9) The crane locking devices comprise buffer stops, turnover prevention apparatus, and rail clamps. The buffer stop is a device that prevents the crane from running off to its traveling direction owing to a wind load at the time of a storm, and the turnover prevention apparatus is a device that prevents the crane from overturning from a wind load during a storm. The rail clamps prevent the crane in operation from running off to its traveling direction owing to a wind load being exerted by a gust.

(10) The examples of the arrangement of crane locking devices for a container crane are shown in **Fig. 9.19.2**. Crane locking devices can be an integrated type, which has both functions of buffer stops and turnover prevention apparatus, or a separated type, which has the functions of the aforementioned two independent units. **Fig. 9.19.3** shows the examples of metal fittings to the foundations of buffer stops and those to the foundations of a turnover prevention apparatus of a separated type.

(11) In renewing a worn-out wheel, they must be jacked up. However, it may not be possible to jack them up where the ground is soft; therefore, jacks should be prevented from sinking by installing the metal fittings to jack-up both sides of the rails (**Fig. 9.19.4**).

(12) The metal fittings of end stoppers are commonly fixed to foundations at the ends of traveling rails (**Fig. 9.19.5**) to mechanically stop the accidental running of the crane.

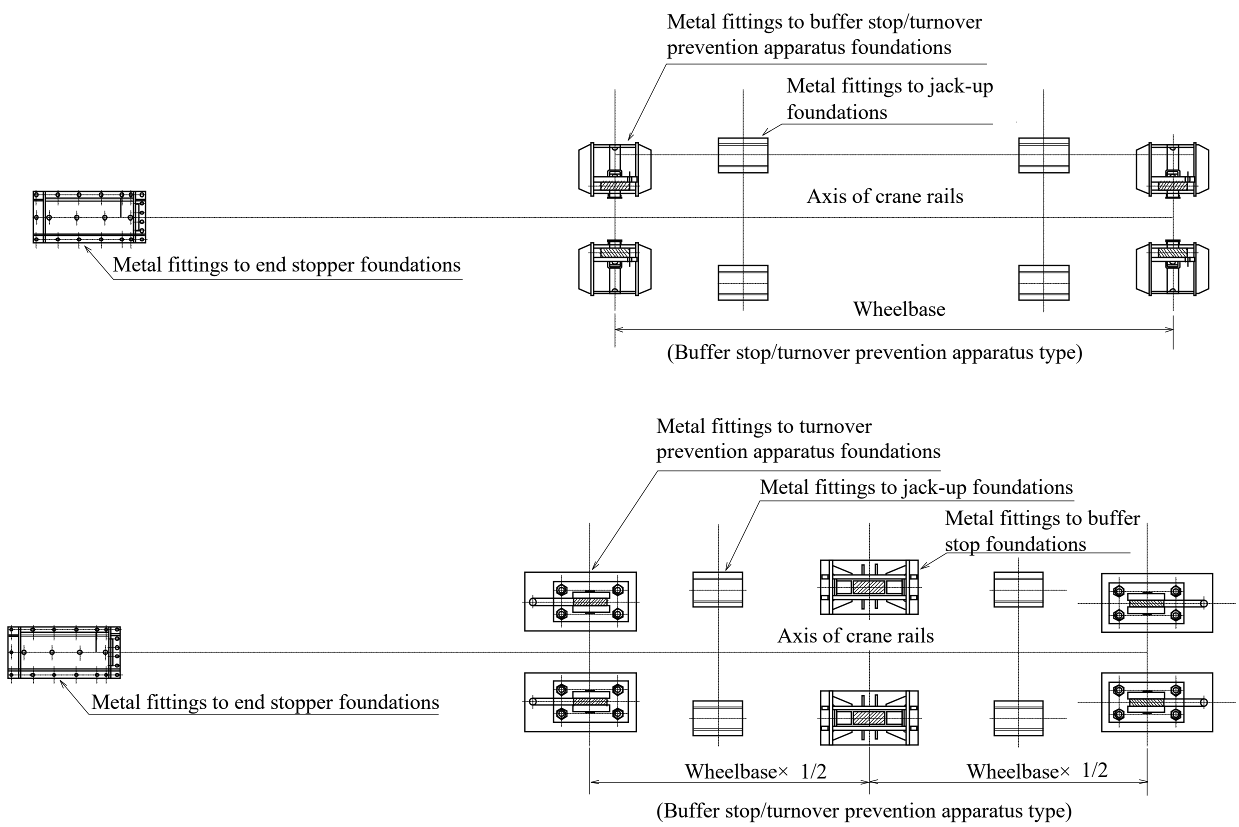
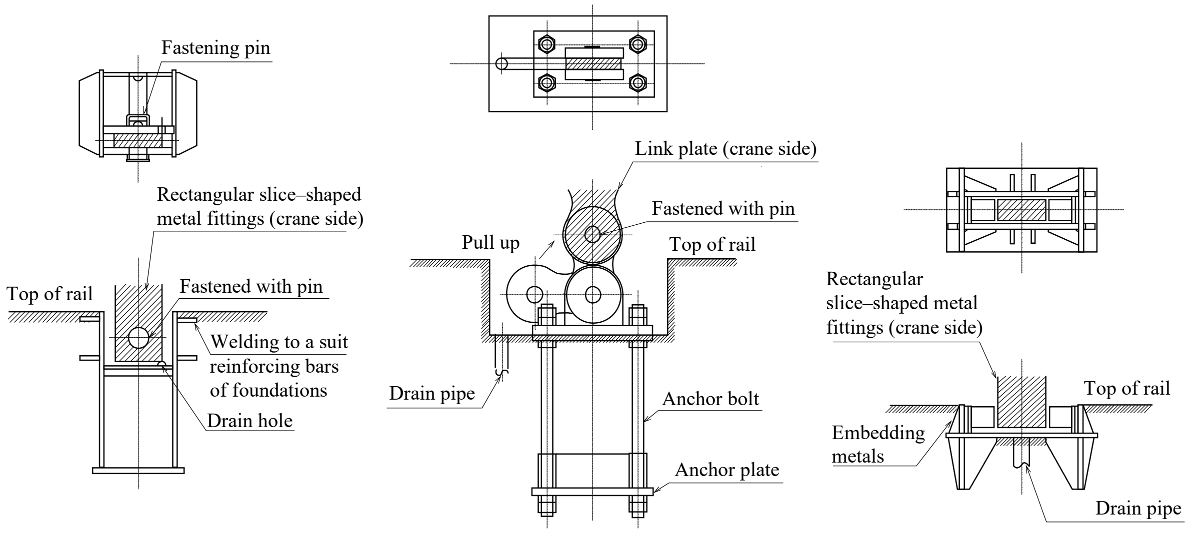


Fig. 9.19.2. Examples of the Arrangement of Locking Devices for Container Cranes[39)]



Metal fittings to buffer stop/turnover prevention apparatus foundations

Metal fittings to turnover prevention apparatus foundations

Metal fittings to buffer stop foundations

Fig. 9.19.3 Examples of the Metal Fittings of Locking Devices to the Foundations for a Container Crane[39)]

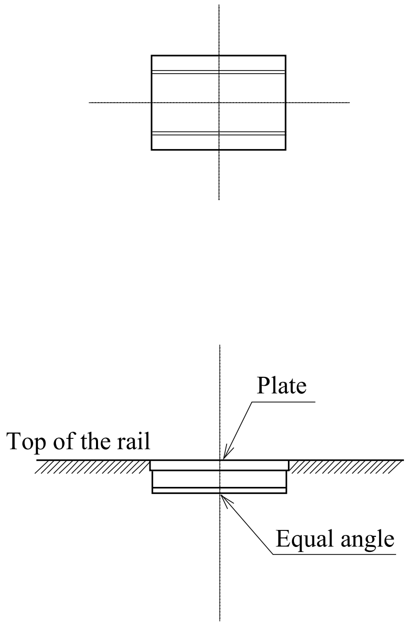


Fig. 9.19.4. Metal Fittings to Foundations for Jack-Up[39)]

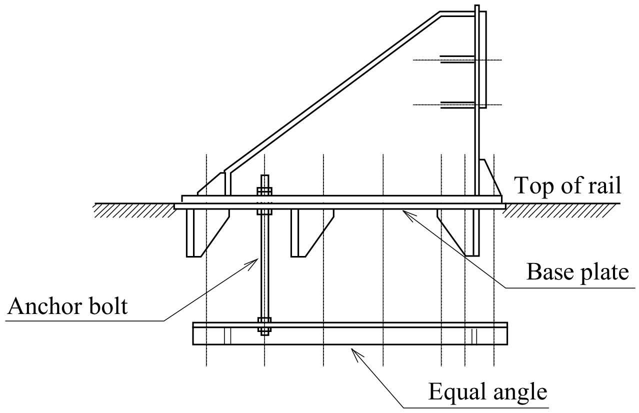


Fig. 9.19.5 Metal Fittings to the Foundations for End Stoppers[39)]

9.19.2 Actions

(1) The action exerted on the foundations for cargo handling equipment shall be determined appropriately in due consideration of the crane type, operation conditions, and other factors.

(2) For the action on the foundations for cargo handling equipment, the values calculated on the basis of **the Calculation Standards for Steel Structures of Cranes (JIS B8821)** or **Structure Standards for Cranes (the Public Notice of the Ministry of Labour)** may be used. For the wheel loads of container cranes being installed in high earthquake-resistance facilities and being under the conditions of an earthquake, refer to **[Part III: Facilities] Chapter 7, 2.2 Container Cranes**. Furthermore, the maximum wheel loads can be tabulated as shown in **Table 9.19.2**. The simultaneous generation of the maximum wheel loads at both sea and land sides must be taken into consideration in the performance verification.

Table 9.19.2 Maximum Wheel Load Items[40] (Unit: kN/wheel)

<table>
<tr><th colspan="3">Columns
State of the crane</th><th>Sea side</th><th>Land side</th><th>Notes</th></tr>
<tr><td rowspan="3">During operation</td><td colspan="2">Vertical</td><td>○○○</td><td>○○○</td><td rowspan="9">Capacity of the crane (t/h)
Self weight of the crane (kN)
Wheelbase (m)
Rail span (m)
Number of wheels: sea side (wheels)
land side (wheels)
Distance between wheels (m)
Traveling speed of crane (m/min)
Column fixed to a beam, hinged column</td></tr>
<tr><td rowspan="2">Horizontal</td><td>Square to rail</td><td>○○</td><td>○○</td></tr>
<tr><td>Parallel to rail</td><td>○○</td><td>○○</td></tr>
<tr><td rowspan="3">During storm</td><td colspan="2">Vertical</td><td>○○○</td><td>○○○</td></tr>
<tr><td rowspan="2">Horizontal</td><td>Square to rail</td><td>○○</td><td>○○</td></tr>
<tr><td>Parallel to rail</td><td>○○</td><td>○○</td></tr>
<tr><td rowspan="3">During earthquake</td><td colspan="2">Vertical</td><td>○○○</td><td>○○○</td></tr>
<tr><td rowspan="2">Horizontal</td><td>Square to rail</td><td>○○</td><td>○○</td></tr>
<tr><td>Parallel to rail</td><td>○○</td><td>○○</td></tr>
</table>

(3) The action shall be considered acting on the entire length of the rails during the operation and during an earthquake, whereas the action shall be considered acting only at the place where the crane is locked during a storm.

(4) For the wheel load during operation, a 20% increase in the maximum wheel pressure of a crane is generally considered a traveling load, whereas a 10% increase is considered a traveling load when the traveling speed of the crane is less than 60 m/min.

9.19.3 Performance Verification of Pile-Type Foundations

(1) Concrete Beams

① The performance verification of concrete beams constructed on pile foundations may be conducted assuming that they are continuous beams supported by pile heads. In this case, the effects of beams contacting the ground are ignored.

② The concrete beams constructed on pile foundations need to be stable against the contact pressure between the rails and concrete, against the stress transmitted from the rails, etc.

③ The underside of the supporting points of beams is subject to tension when elastic settlement occurs on piles by axial force. The beams are generally calculated by assuming that no settling occurs on the supporting points and that the effects of such elastic settlement is taken into account in the arrangement of reinforcement; however, the effects of settling on the supporting points need to be considered when a large wheel load is present,[41] and **[Part III: Facilities], Chapter 2, 3.4.5 Pile Head Displacement by Axial Force** may be used as a reference for performance verification.

④ To avoid elastic settlement or to reduce it, it is advisable either to fill the inside of steel piles with concrete or to employ large-sized steel piles in diameter.

⑤ To calculate the stress on tracks, a solution that assumes beams as infinite beams on elastic foundations is used commonly. This method is particularly applied to the calculation of cases in which elastic compressive materials, such as rubber pads, are inserted between the rails and concrete to distribute the action and prevent the compression failure of the foundation concrete.

⑥ Calculation Method of the Infinite Continuous Beam Supported by Elastic Foundations

The rail stress and the contact pressure between the rails and concrete can be calculated by referring to **9.19.4 Performance Verification in the Cases of Pile-less Foundation, (2) Concrete Beams** in this chapter, described later. In this case, the symbols in **equation (9.19.1)** should be read as follows:

E_c : modulus of elasticity of the rail

I_c : moment of inertia of the rail

K : modulus of elasticity of the material placed under the rail (when tie pads are used, use their modulus of elasticity)

When the bearing stress is excessively large, it should be reduced by inserting elastic plates under the rails.

⑦ The fastening force between the rail and the foundation can be calculated by using the beam theory on elastic foundation[42], but it is necessary to have a sufficient allowance to avoid the effect of the impact. For the calculation of the fastening force for the cases where the double elastic fastening method is employed, refer to Minemura[43]. In many cases, bolts with a diameter of approximately 22 mm are used at intervals of approximately 50 cm.

(2) Maximum Static Resistance Forces of Piles

① The piles shall be stable against the actions caused by cargo handling equipment and foundations.

② The action that is exerted on the piles shall be the reaction force at each supporting point calculated in accordance with above **(1) Concrete Beams**.

③ The maximum static resistance forces of piles may be calculated by referring to **[Part III: Facilities], Chapter 2, 3.4 Pile Foundations**.

④ In cases wherein piles are affected by the failure surface of active earth pressures, the performance verification of the relieving platform piles described in **2.8 Quaywalls with Relieving Platforms** in this chapter may be used as a reference.

⑤ When piles are under the influence of the active earth pressure failure surface, the required embedment length differs between seaward piles and landward piles; however, it is common practice to use foundation piles of the same length for both sides to avoid a differential settlement of the foundation. Nevertheless, the same embedment length is not required when the piles at each side are driven into the bearing stratum.

9.19.4 Performance Verification in Cases of Pile-Less Foundations

(1) Analysis of the Effect on the Quaywall[44]

① When the foundation for cargo handling equipment is not a piled structure, the effect of the actions by the cargo handling equipment and its foundations on the main body of mooring facilities shall be examined.

② A surcharge on the area behind a gravity-type structure increases the earth pressure and may cause the forward sliding of the quaywall. The influence of a concentrated load on the earth pressure is significant at its loading point around the ground surface and becomes distributed depending on the depth. This has significant influence particularly on structures with a short wall height and length (of the direction of alignment); therefore, this type of load should be considered. Furthermore, if the facility is loaded directly on its top, the subgrade reaction increases. In particular, when the load is applied on a quaywall at its front end, the subgrade reaction at the front toe becomes significantly large. In quaywalls with a small width and short length, this tendency is significant and should be noted.

③ In ordinary sheet pile quaywalls, the maximum stress occurs between the tie rod installation point and the sea bottom; however, when a concentrated load acts on the area behind the sheet pile wall, the maximum stress may occur at the level of the tie rod installation point. However, this rarely causes an adverse effect on the embedded part of the sheet pile. It is preferable to provide a sufficient earth covering thickness for the tie rods to avoid adverse effects.

(2) Concrete Beams

① The reinforced concrete beams placed on rubble foundations laid on the ground shall ensure stability against bending moments, shear forces, and deflection, and their settlement shall be less than the limit value of the settlement.

② The characteristic values of the bending moments, shear forces, and deflection of the reinforced concrete beams placed on rubble foundations can be obtained from **equations (9.19.1)** to **(9.19.6)**. Here the variables with subscript k denote the characteristic values.

(a) In cases where loads act near the middle of a beam,

$$M_k = \sqrt[4]{\frac{E_c I_c}{64K}} \sum W_i e^{-\beta x_i} (\cos\beta x_i - \sin\beta x_i) \qquad \textbf{(9.19.1)}$$

$$S_k = \frac{1}{2} \sum W_i e^{-\beta x_i} \cos\beta x_i \qquad \textbf{(9.19.2)}$$

$$y = \sqrt[4]{\frac{1}{64E_cI_cK^3}}\sum W_i e^{-\beta x_i}(\cos\beta x_i + \sin\beta x_i) \tag{9.19.3}$$

(b) In cases where loads act on beams ends or junctions,

$$M = \sum \frac{W_i}{\beta} e^{-\beta x_i} \sin\beta x_i \tag{9.19.4}$$

$$S = \sum W_i e^{-\beta x_i}(\sin\beta x_i - \cos\beta x_i) \tag{9.19.5}$$

$$y = \sum \frac{2W_i\beta}{K} e^{-\beta x_i} \cos\beta x_i \tag{9.19.6}$$

where

M : bending moment on subject cross section (N･mm)

S : shearing force on subject cross section (N)

y : deflection on subject cross section (mm)

$$\beta = \sqrt[4]{\frac{K}{4E_cI_c}}$$

E_c : modulus of elasticity of concrete (N/mm^2)

W_i : wheel load (N)

I_c : moment of inertia of concrete foundation (mm^4)

K : modulus of elasticity of ground $K = Cb$ (N/mm^2)

C : pressure needed for a unit area of ground to settle by unit depth (N/mm^3)

b : bottom width of concrete beam (mm)

x_i : distance from wheel load point to subject section (mm)

③ The reinforced concrete beams placed on rubble foundations are assumed to be supported by the continuous elastic foundations of a uniform section over the entire length. In other words, it is assumed that the reaction forces of loaded beams are continuously distributed, and their strengths are directly proportional to the deflection at each point. By defining the bending moment generated at a point of a distance x from the traveling wheel as M and the deflection as y, M and y are expressed by **equations (9.19.7)** and **(9.19.8)** by elastic theory, respectively.[45), 46)]

$$M_k = W\sqrt[4]{\frac{E_cI_c}{64K}}e^{-\beta x}(\cos\beta x - \sin\beta x) = W\sqrt[4]{\frac{E_cI_c}{64K}}\phi_1 \tag{9.19.7}$$

$$y = \frac{W}{\sqrt[4]{64E_cI_cK^3}}e^{-\beta x}(\cos\beta x + \sin\beta x) = \frac{W}{\sqrt[4]{64E_cI_cK^3}}\phi_2 \tag{9.19.8}$$

When two or more wheels are close to each other, the bending moment directly under any one wheel is obtained from **equation (9.19.9)**.

$$M_{1k} = W_1\sqrt[4]{\frac{EI}{64K}} \tag{9.19.9}$$

By expressing the distance to another wheel as x_2 and ϕ_1 for βx_2 as ϕ_{12}, the bending moment is calculated from **equation (9.19.10)**.

$$M_{2k} = W_2\sqrt[4]{\frac{EI}{64K}}\phi_{12} \tag{9.19.10}$$

The resultant moment directly under the first wheel can be determined from $M = M_1 + M_2$. **equation (9.19.1)** can be derived from this expression. The deflection can be obtained in the same way. The values given by the following expression may be used for the values of C.[45), 47)]

$C = 5.0 \times 10^{-2}$ to 0.15 (N/mm^3)

④ **Verification of the Sliding of Concrete Beams**

In the verification of sliding of concrete beams, **equation (9.19.11)** may generally be used. In the following equations, γ represents a partial factor related to the subscript concerned, and subscripts k and d indicate the characteristic value and the design value, respectively.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$R_k = LR_P + f\left(LW_c + \sum W_i\right) \qquad \textbf{(9.19.11)}$$
$$S_k = P + LE_A$$

where

P : horizontal force (action by earthquake ground motion, wind pressure) (kN)

E_P : passive earth pressure (kN/m)

E_A : active earth pressure (kN/m)

W_c : weight of concrete beam (kN/m)

ΣW_i : sum of wheel loads acting on subject calculation section (kN)

f : friction coefficient between rubbles and concrete $f_k = 0.6$

L : length of one block or 10 m, whichever is shorter (m)

In examining the sliding of a concrete beam, an appropriate value of 1.2 or more may be used as the modification coefficient m, and 1.0 may be used for all other partial factors.

⑤ In examining the bearing stresses on concrete beams exerted from between the rails and concrete, the stresses of rails, etc., refer to **9.19.3 Performance Verification of Pile-type Foundations, (1) Concrete Beams** in this chapter.

⑥ The subgrade reaction may be examined by assuming that the loads are equally distributed to a beam of one block concerned or 10 m in length, whichever it is shorter, and by referring to **[Part III: Facilities] Chapter 2, 3.2.2 Bearing Capacity of foundations on Sandy Ground**.

9.20 Transitional Parts[48)]

9.20.1 General

(1) Generally, transitional parts can be divided into the four types listed below.

① Part in which the front water depth changes
② Part in which different type structures are connected
③ Corner part
④ Transitional part to a breakwater, etc.

(2) In designing a transitional part, the items listed below shall be considered.

① **Natural conditions**

The ground near a transitional part can often change in complicated ways, so the ground conditions must be thoroughly understood. In addition, waves often concentrate in corners, so careful attention is required.

② **Differential settlement**

The structural types near transitional parts often vary. These parts may cause breakage due to differential settlements, particularly when making an installation on an existing facility on soft ground.

③ **Outflow of backfilled earth and sand**

When the structural types vary, backfilled earth and sand could flow out from the transitional part, so careful attention is required.

④ **Differences in rigidity**

When the structural types vary at a transitional part, deformation will vary due to differences in rigidity, which often becomes a cause of breakage.

⑤ **Relationship with existing facilities**

Attention shall be paid to transitional parts so that they do not affect the existing facilities. In addition, when a transitional part is expected to be extended in the future, it is desirable for attention to be paid to make future extensions easier.

(3) Attention shall be paid to construction procedures for transitional parts to prevent reworking and make it possible for the construction to follow the procedures. In addition, construction machines to be used are preferably the same as those used for the main body part. Construction methods that require completely different machines should be avoided.

9.20.2 Notes on Sections in Which the Front Water Depth Changes

(1) For sections in which the front water depth changes, the design conditions of the facility for which the depth of the connected water is deeper shall be used.

(2) For performance verification of facilities in sections in which the front water depth changes, the applicable structural types can be referred to.

(3) For sections in which the front water depth changes, there will be issues with determining the design conditions and the stability of the slope. Ideally, the seabed slope in the transitional section is preferably steep, especially from the viewpoint of utilization and cost. However, the seabed slope is preferably determined in consideration of a stable gradient based on the ground, the influence of waves, slope protection, slope gradient of dredging, and other factors. Usually, the seabed slope for sandy soil can be approximately 1:3.

(4) The design water depth can gradually be changed based on the stiffness of the facilities, deformation at the time of seismic vibrations (actions), cost, and other factors.

① When blocks are used, the design depth should ideally is preferably changed in stages as shown in **Fig. 9.20.1** using the height of the blocks as units.

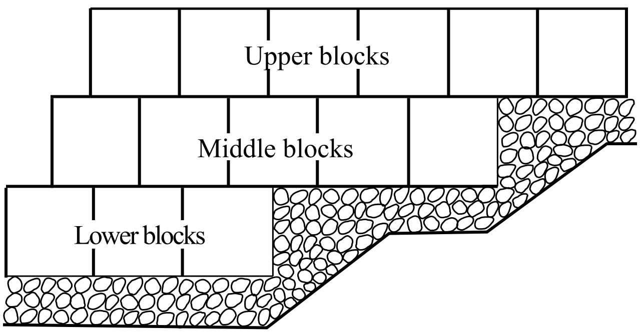


Fig. 9.20.1 Installation by Piling Blocks When the Front Water Depth Changes

② For a sheet pile structure, the design depth is often changed in stages by approximately 2 to 3 m as shown in **Fig. 9.20.2**.

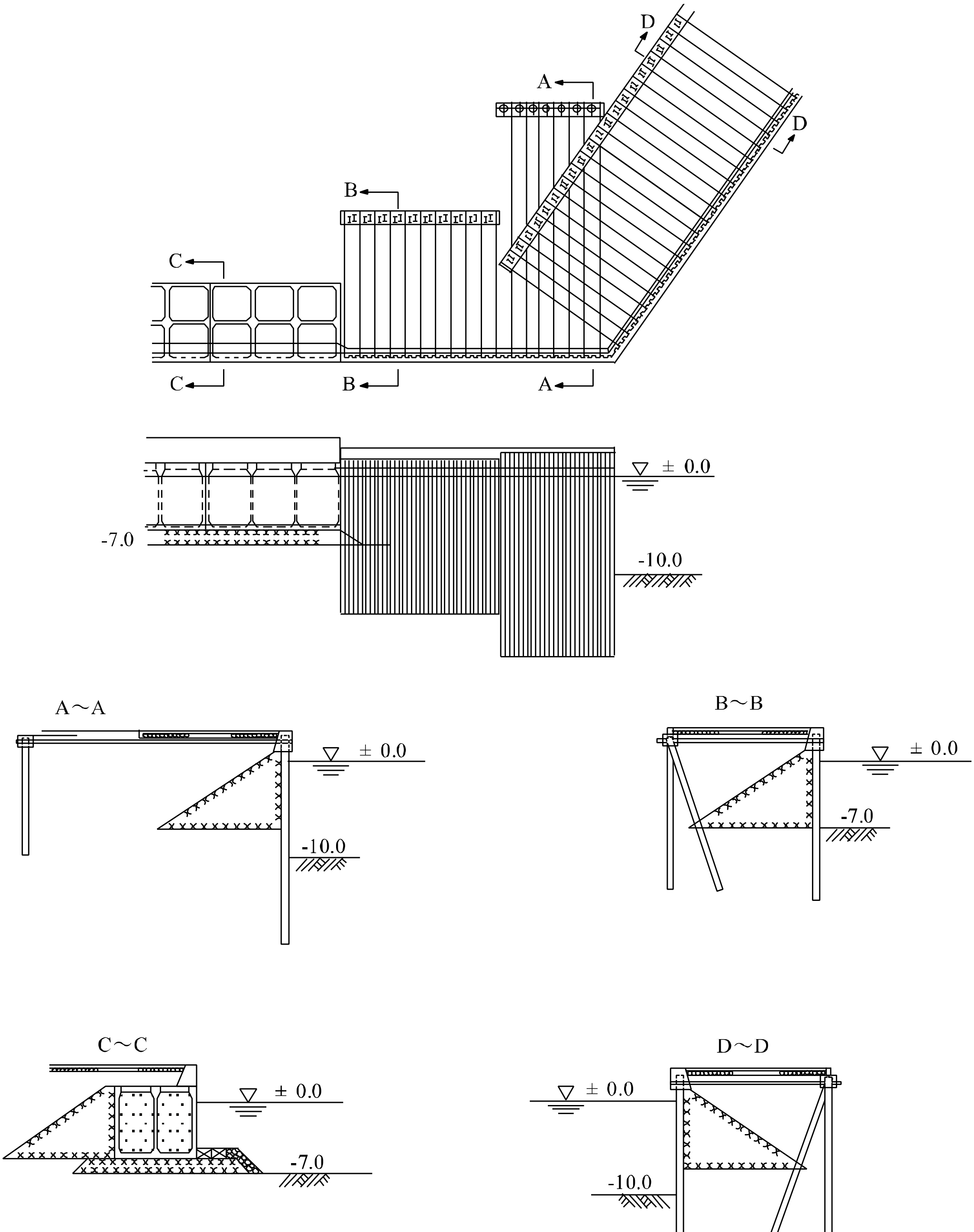


Fig. 9.20.2 Installation Using a Sheet Pile Structure When the Front Water Depth Changes

9.20.3 Notes on Sections in Which Different Facilities Are Connected

For sections in which different facilities are connected, there are cases where facilities having different structural types are directly connected, and cases where a connection facility is provided between facilities of different structural types. When an in-between connection facility is provided, the strictest design conditions from the two facilities in terms of stability should be applied.

9.20.4 Notes on Corner Sections

(1) For the facility design conditions for corner areas, the strictest design conditions from the two facilities in terms of stability should be applied.

(2) Sharp corners make configuration and construction difficult, so it is desirable to avoid making corners sharp as much as possible.

(3) When sheet pile structures are connected to each other

There are various design examples, but one issue is the type of shoring used. When the anchor slab type is used, an anchor slab may get into an active earth pressure area, or an area may be formed where passive earth pressures work against each other, so there is an issue with determining the earth pressure that acts as resistance. Therefore, it is desirable to avoid using the anchor slab type.

(4) Example cross sections of transitional parts that can be regarded as acceptable for the structures shown below.

In straight-pile shoring, the lateral resistance of the straight pile receives tension from the tie rod. In this case, it is desirable that the angle between the tie rod and sheet pile wall be a right angle. For performance verification, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**. **Fig. 9.20.3** illustrates example cross sections when straight-pile shoring was installed at a corner.

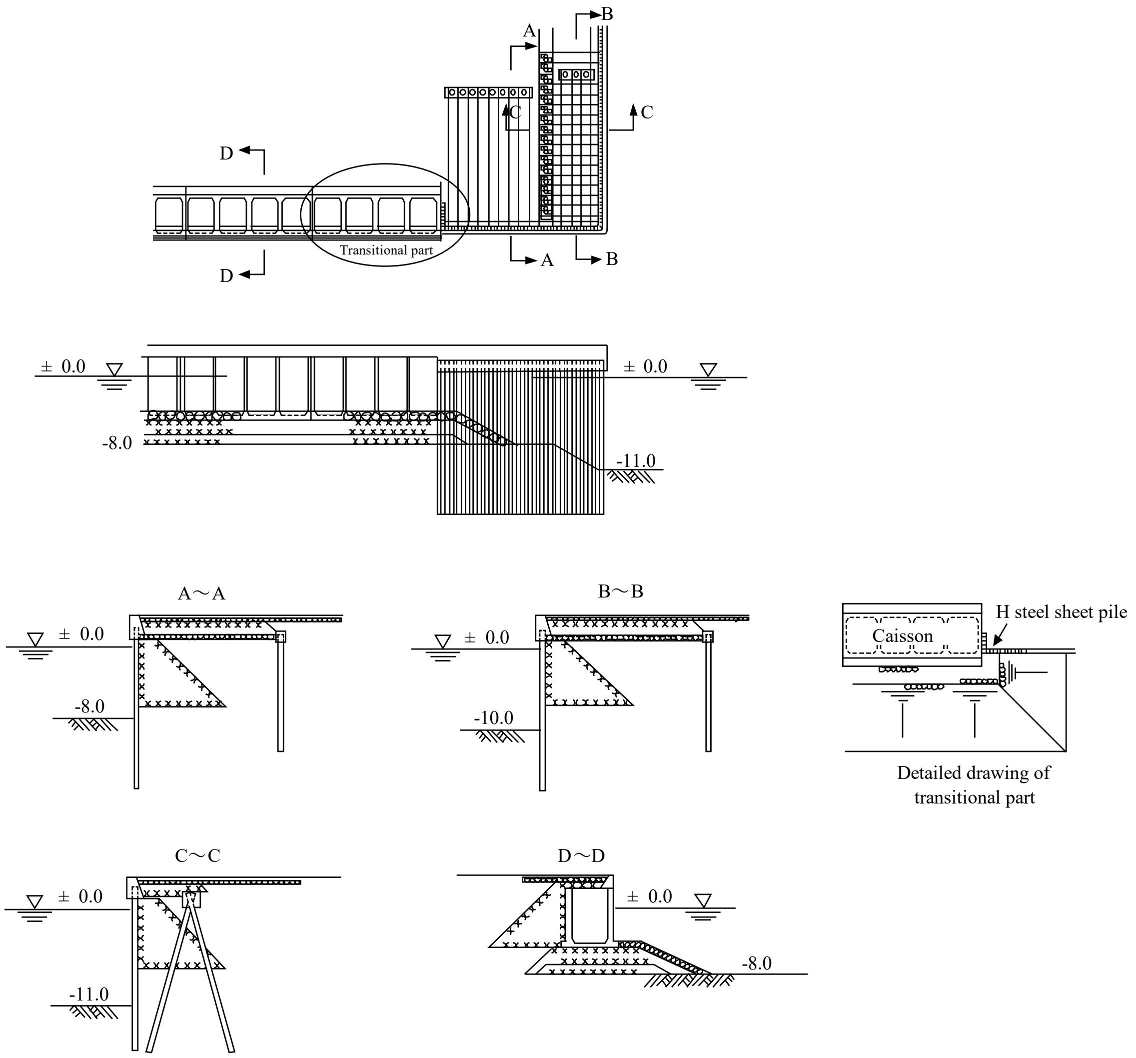


Fig. 9.20.3 Example Cross Sections When Straight-Pile Shoring is Installed at a Corner

② **When a structure with a relieving platform is installed at a corner**

(a) Structures with relieving platforms do not require complicated shoring, so they are used relatively frequently. For performance verification, the items listed below generally shall be considered.

1) For bending moments and axial forces, those in the x and y directions shown in **Fig. 9.20.4** and those in the resultant force direction shall be considered. Those in the resultant force direction can be calculated as a vector sum of the values calculated for the x and y directions.
2) For the embedment of piles, the embedment length of a pile that will be the most dangerous is preferably used for the others.
3) For performance verification of structures with relieving platforms, refer to **Part III, Chapter 5, 2.8 Quay Walls with Relieving Platforms**.

(b) **Fig. 9.20.5** illustrates example cross sections when a structure with a relieving platform was installed at a corner.

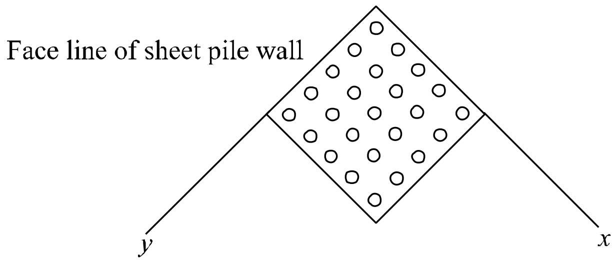


Fig. 9.20.4 When a Structure with a Relieving Platform is Installed at a Corner

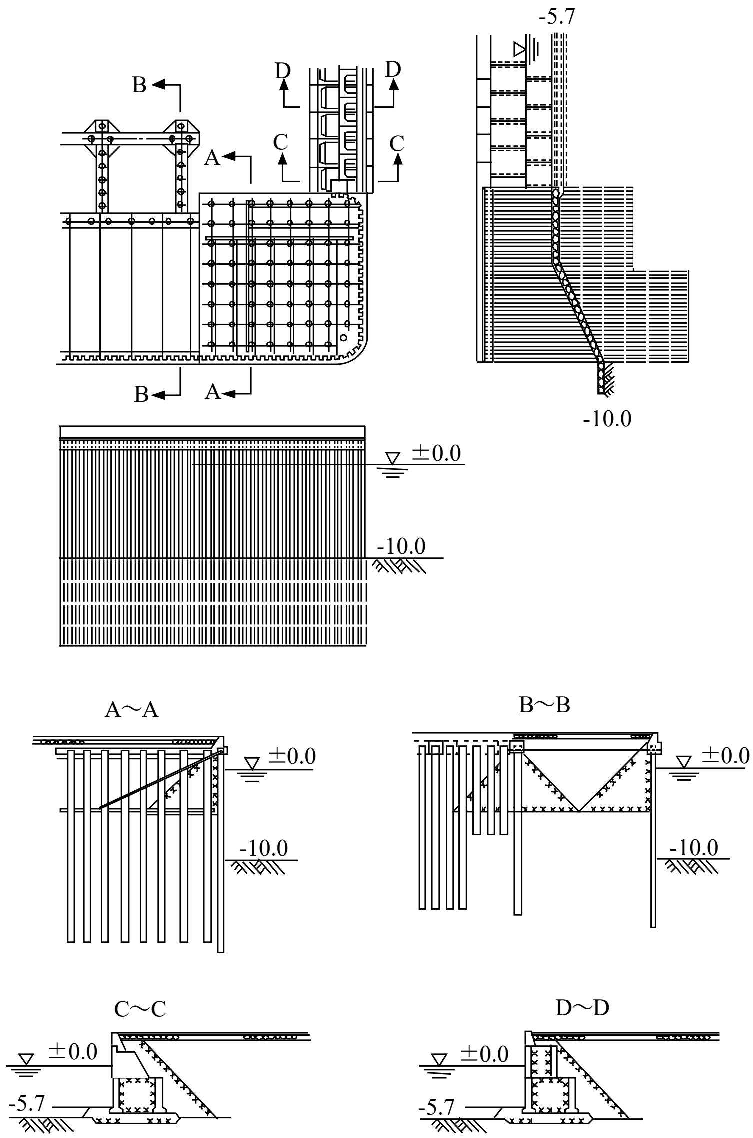


Fig. 9.20.5 Example Cross Sections When a Structure with a Relieving Platform is Installed at a Corner

③ **When a double sheet pile structure is installed at a corner**

Double sheet pile structures are often used particularly when the water depth is shallow. For performance verification of double sheet pile structures, refer to **Part III, Chapter 5, 2.7 Double Sheet Pile Quay Walls**.

④ **When a sheet pile structure is connected to a cantilevered sheet pile structure**

Cantilevered sheet pile structures do not require shoring, so they are sometimes used when the ground is fine or for wharves with shallow water depth (shallow draft wharves). In this combination, usually the displacement of the cantilevered sheet pile structure is larger than that of the sheet pile structure, and thereby a large force may be applied to the tie rod of the transitional part. Therefore, consideration is desirable, for example, when the rigidity of the cantilevered sheet pile near the transitional part is made higher, or when the tie rod is made thicker. For performance verification of cantilevered sheet pile structures, refer to **Part III, Chapter 5, 2.4 Cantilevered Sheet Pile Quaywalls**. **Fig. 9.20.6** illustrates example cross sections when a sheet pile structure and a cantilevered sheet pile structure are connected.

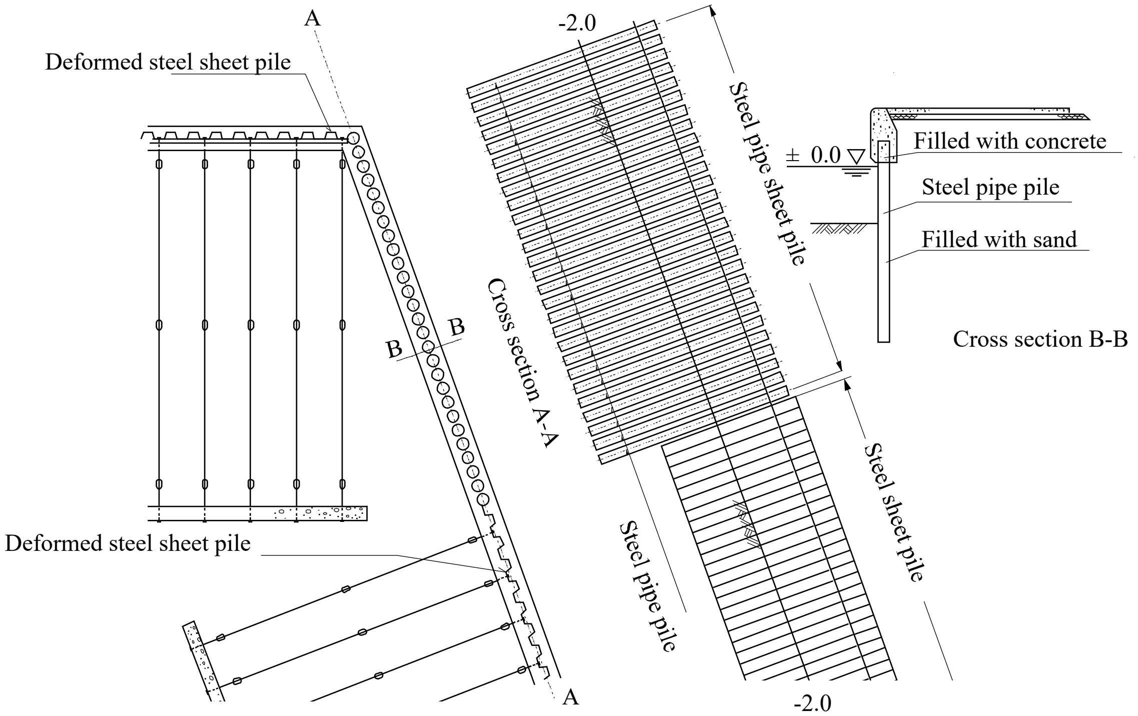


Fig. 9.20.6 Example Cross Sections When a Sheet Pile Structure and a Cantilevered Sheet Pile Structure are Connected

⑤ **When a caisson is installed at a corner**

When a caisson is installed at a corner, the force in the x and y directions shown in **Fig. 9.20.7** shall be considered, and it is desirable that the force in the resultant force direction (direction z) be considered. For performance verification of caissons, refer to **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls**.

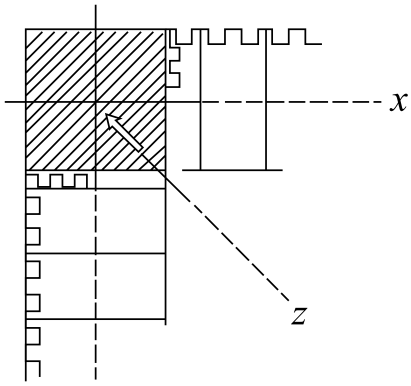


Fig. 9.20.7 When a Caisson is Installed at a Corner

⑥ **When a cellular-bulkhead structure is installed at a corner**

Methods to install a cellular-bulkhead structure or well at a corner are also available, but generally the construction costs tend to be high. For performance verification of cellular-bulkhead structures, refer to **Part III, Chapter 5, 2.9 Cellular-Bulkhead Quaywalls with Embedded Sections**. **Fig. 9.20.8** illustrates example cross sections when a well was installed at a corner.

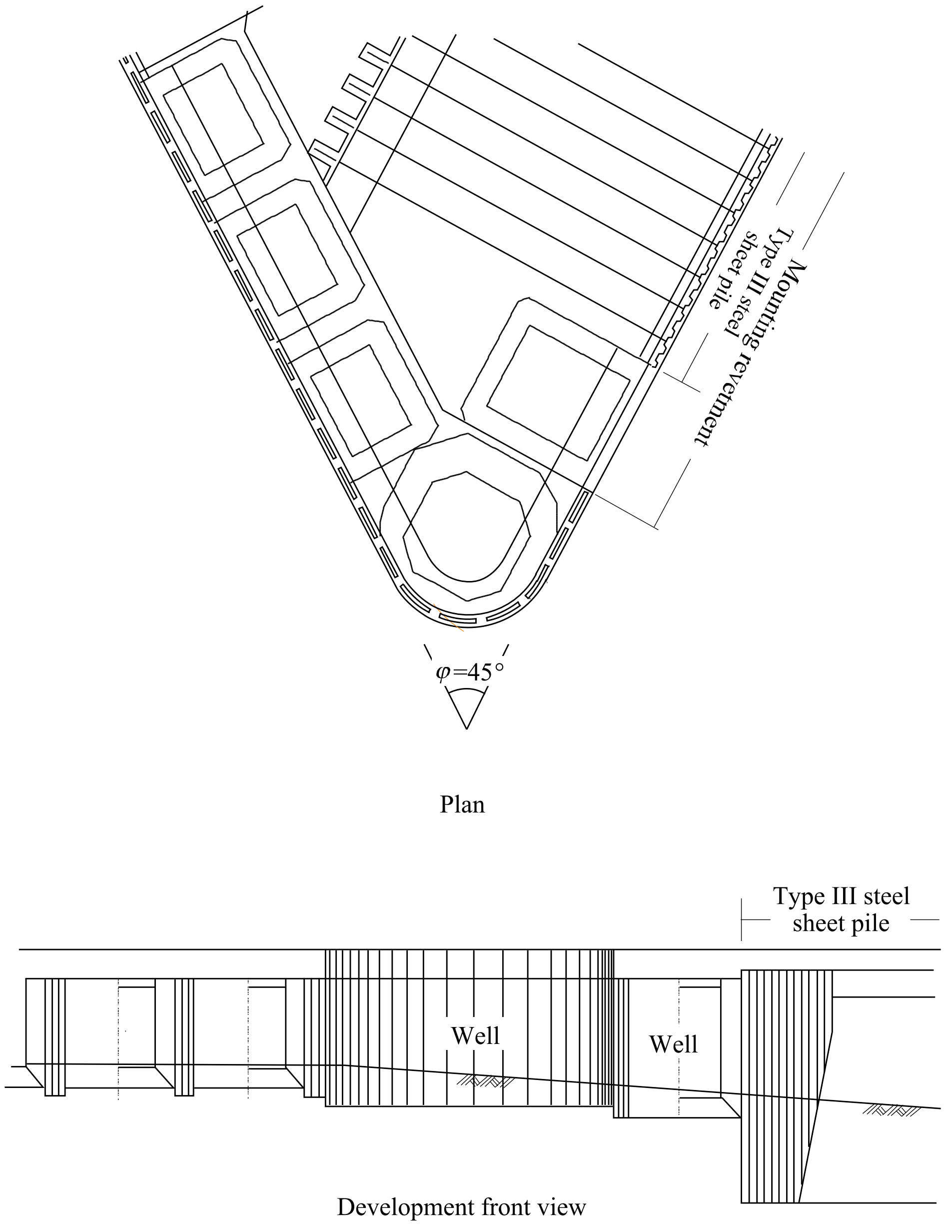


Fig. 9.20.8 Example Cross Sections When a Well is Installed at a Corner

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10. Mooring Facilities for Electric Power Generation Systems using Renewable Energy

(English translation of this section from Japanese version is currently being prepared.)

10.1 Fundamentals of Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

Chapter 6 Port Transportation Facilities

(English translation of this section from Japanese version is currently being prepared.)

1 General

(English translation of this section from Japanese version is currently being prepared.)

1.1 Traffic Signs and Markings

(English translation of this section from Japanese version is currently being prepared.)

2 Port Roads

(English translation of this section from Japanese version is currently being prepared.)

2.1 Fundamentals of Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

2.2 Carriageways and Lanes

(English translation of this section from Japanese version is currently being prepared.)

2.3 Clearance Limits

(English translation of this section from Japanese version is currently being prepared.)

2.4 Widening of the Curved Sections of Roads

(English translation of this section from Japanese version is currently being prepared.)

2.5 Longitudinal Slopes

(English translation of this section from Japanese version is currently being prepared.)

2.6 Level Crossings

(English translation of this section from Japanese version is currently being prepared.)

2.7 Performance Verification of Pavements

(English translation of this section from Japanese version is currently being prepared.)

3 Underwater Tunnels

[Ministerial Ordinance] (Performance Requirements for Roads)

Article 36

1 The performance requirements for roads shall be as prescribed in the following items:

(1) Roads shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to ensure the safe and smooth flow of traffic within ports,and between ports, and their hinterlands in consideration of the traffic characteristics in the ports.

(2) Damage due to surcharge loads, etc. shall not adversely affect the continous use of the roads without impairing their functions.

2 In addition to the provisions of the preceding paragraph, the performance requirements for roads with tunnel sections shall be as prescribed in the following items:

(1) Damage due to self-weight, earth pressure, water pressure, Level 1 earthquake ground motions, etc shall not adversely affect the continous use of the roads without impairing their functions.

(2) Damage due to Level 2 earthquake ground motions, flames and heat due to fires, etc. shall not adversely affect the restoration of their functions through minor repair works.

[Public Notice] (Performance Criteria of Underwater Tunnels)

Article 78

1 The performance criteria of underwater tunnels shall be as prescribed in the following items:

(1) Underwater tunnels shall be covered with appropriate materials of required thicknesses to secure the integrity of the structural members and the stability of their structures against the dropping and dragging of ship anchors, scouring of seabed by waves and currents, etc.

(2) Underwater tunnels shall be equipped with the management facilities necessary for their safe and smooth use.

(3) The degree of damage under accidental situation, in which the dominating actions are Level 2 earthquake ground motions, flames, and heat due to fires shall be equal to or less than the threshold level.

2 In addition to the provisions of the preceding paragraph, the performance criteria of underwater tunnels shall be as prescribed in the following items:

(1) The risk of failure due to the insufficient bearing capacity of the foundation ground under the permanent state, in which the dominating action is self-weight, shall be equal to or less than the threshold level.

(2) The risk of impairing the integrity of structural members under the permanent state, in which the dominating action is earth pressure, shall be equal to or less than the threshold level.

(3) The risk of the floating-up of immersed tunnel elements, ventilation facilities and shafts ,under the variable situation in which the dominating action is water pressure, shall be equal to or less than the threshold level.

(4) The risk of impairing the integrity of structural members and losing the stability of immersed tunnel elements, ventilation facilities, shafts, joint sections, etc. under the variable situation in which the dominating action is Level 1 earthquake ground motion, shall be equal to or less than the threshold level.

[Interpretation]

12 Port Transportation Facilities

(2) Performance Criteria of Underwater Tunnels (Article 36 paragraph 2 of the Ministerial Ordinance and the interpretation related to Article 78 of the Public Notice)

① The required performance of underwater tunnels shall be serviceability. Serviceability is defined as follows:

a) The quality and thickness of materials covering underwater tunnels shall be appropriately set with careful consideration to the stability against the uplift of underwater tunnels, the effects of the penetration of cast and dragged anchors by ships navigating over underwater tunnels, and the scouring of covering sections owing to waves and currents.

b) Underwater tunnels shall be provided with appropriate management facilities and equipment necessary for their safe and smooth use.

② The required performance of underwater tunnels under the accidental situation in which the dominating actions are Level 2 earthquake ground motions, flames, and heat due to fires shall be restorability. **Attached Table 12-3** shows the performance verification items and standard indexes for determining limit values with respect to the actions. In this table, the term "damage" is used for the performance verification item as a comprehensive expression of verification items that differ depending on the types of underwater tunnels.

Attached Table 12-3 Performance Verification Items and Standard Indexes for Determining the Limit Values of Underwater Tunnels under an Accidental Situation

Ministerial ordinance			Public notice			Performance requirements	Design situation			Verification item	Standard index for determining the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
35	2	–	78	1	3	Restorability	Accidental	Flames and heat due to fires	–	Damage	–
36	2	2						Level 2 earthquake ground motions	Self-weight, earth pressure, water pressure, surcharge		

③ In the performance verification of underwater tunnels under the accidental situation of flames and heat due to fires, the actions of the flames and heat due to fires shall be appropriately set in accordance with the types of vehicles passing through the underwater tunnels. Furthermore, the members constituting underwater tunnels shall be coated with fire-resistant materials when necessary.

④ The performance criteria of immersed and shield tunnels shall be applied mutatis mutandis to those of underwater tunnels. In addition to the provision above, **Attached Table 12-4** shows the performance verification items and standard indexes for determining the limit values with respect to the actions on immersed and shield tunnels under the permanent action situation in which the dominating action are self-weight and earth pressure and under the variable situation in which the dominating actions are water pressure and Level 1 earthquake ground motions.

Attached Table 12-4 Performance Verification Items and Standard Indexes for Determining the Limit Values of Immersed and Shield Tunnels under Respective Design Situations

<table>
<tr><th colspan="3">Ministerial ordinance</th><th colspan="3">Public notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index for determining the limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="7">35
36</td><td rowspan="7">2
2</td><td rowspan="7">–
1</td><td rowspan="7">78</td><td rowspan="7">2</td><td>1</td><td rowspan="7">Usability</td><td rowspan="2">Permanent</td><td>Self-weight</td><td>Water pressure, earth pressure, and surcharge</td><td>Bearing capacity of the foundation ground</td><td>Action–resistance ratio with respect to bearing capacity</td></tr>
<tr><td>2</td><td>Earth pressure</td><td>Self-weight, water pressure, and surcharge</td><td>Soundness of members</td><td>–</td></tr>
<tr><td>3</td><td rowspan="5">Variable</td><td>Water pressure</td><td>Self-weight, earth pressure, and surcharge</td><td>Uplifting of immersed tunnel elements, ventilation facilities, and shafts</td><td>–</td></tr>
<tr><td rowspan="4">4</td><td rowspan="4">Level 1 earthquake ground motion</td><td rowspan="4">Self-weight, earth pressure, water pressure, and surcharge</td><td>Stability of immersed tunnel elements, ventilation facilities, and shafts</td><td>–</td></tr>
<tr><td>Soundness of members</td><td>–</td></tr>
<tr><td>Stability of joint sections</td><td>–</td></tr>
</table>

⑤ In the performance verification of the soundness of members constituting tunnels (**Attached Table 12-4**), the standard indexes for determining the limit values shall be appropriately set in accordance with the structures of tunnels and the materials of the members.

⑥ In the performance verification of the stability of the immersed tunnel elements, ventilation facilities, and shafts (**Attached Table 12-4**), the standard indexes for determining the limit values shall be appropriately set in accordance with the structures of tunnels.

⑦ In the performance verification of the stability of the joint sections of tunnels (**Attached Table 12-4**), the standard indexes for determining the limit values shall be appropriately set in accordance with the structures of tunnels, materials, and joint sections. The stability of joint sections shall ensure the strength, deformation performance, and waterproof property of the joint sections.

3.1 General

(1) Tunnels as port transportation facilities are classified into road, railroad, and other tunnels in terms of intended use. They are also classified into mountain, open-cut, shield, and immersed tunnels in terms of construction method.

(2) The explanations in this section can be applied to the performance verification of underwater tunnels as port road facilities constructed using the submerged tunneling method (hereinafter immersed tunnels) and those constructed using the shield tunneling method (hereinafter shield tunnels). For the performance verification of tunnels for other uses or with different structural types, the standards relevant to the respective tunnels should be followed.

(3) In the performance verification of underwater tunnels, refer to the **Road Structure Ordinance** (Cabinet Order No. 424 on December 26, 2011) for items that are not specified in the following sections.

(4) **Fig. 3.1.1** shows the definitions of terms related to the immersed tunnels used in this section.

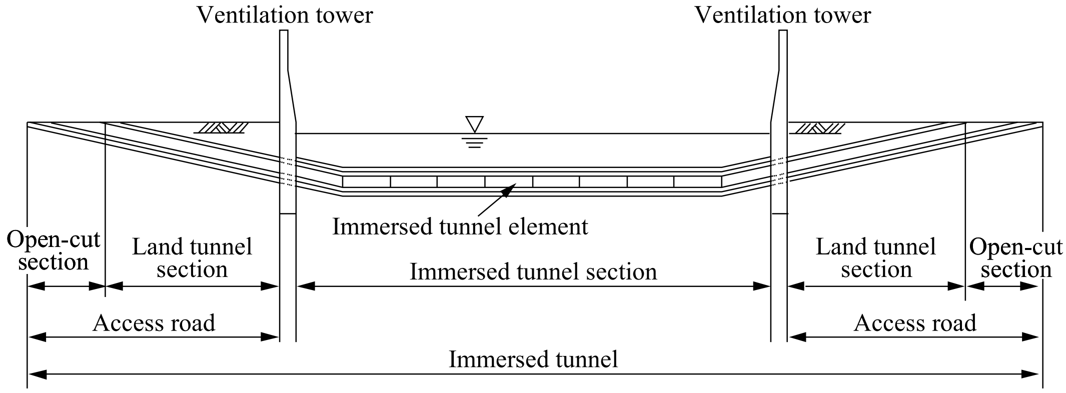

Fig. 3.1.1 Definitions of terms on Immersed Tunnel

(5) The standards related to underwater and other types of tunnels are as follows.

① The **Technical Standard for Road Tunnels** (Official Notice by the Director-General, City Bureau and the Director-General, Road Bureau, Ministry of Construction, May 19, 1988)

② The **Technical Standard and Commentaries for Road Tunnels (Ventilation)** (The Japan Road Association, June 2015)

③ The **Technical Standard and Commentaries for Road Tunnels (Structure)** (The Japan Road Association, December 2003)

④ The **Installation Standard for Road Tunnel Emergency Facilities** (Official Notice by the Director-General, City Bureau and the Director-General, Road Bureau, Ministry of Construction, April 21, 1981)

⑤ The **Installation Standard and Commentaries for Road Tunnel Emergency Facilities** (The Japan Road Association, October 2001)

⑥ The **Installation Standard and Commentaries for Road Lighting Facilities** (The Japan Road Association, October 2007)

⑦ The **Standard Specifications for Tunneling-2016: Mountain Tunnels** (The Japan Society of Civil Engineers, August 2016)

⑧ The **Standard Specifications for Tunneling-2016: Cut-and-Cover Tunnels** (The Japan Society of Civil Engineers, August 2016)

⑨ The **Standard Specifications for Tunneling-2016: Shield Tunnels** (The Japan Society of Civil Engineers, August 2016)

(6) In the performance verification of immersed tunnels as port road facilities, the **Technical Manual for Immersed Tunnels**[1)] can be used as a reference. **References 2)** to **7)** can be used as a general reference for the design and construction of immersed tunnels. Furthermore, **References 8)** and **9)** can be used as references for the examination of the earthquake resistance of immersed tunnels.

3.2 Fundamentals of Performance Verification

(1) The locations, alignments, and cross-sectional shapes of underwater tunnels shall be appropriately set in accordance with the use conditions, plans of related facilities, such as future deepening plans of the navigation channels, and the natural conditions of water areas where tunnels are constructed. Port districts have particularly high composition rates of heavy vehicles; therefore, it is necessary to give appropriate consideration to ventilation methods and road alignments.

(2) Past examples of construction can be used as references in selecting the locations and deciding the alignments and the cross sections of underwater tunnels.

(3) One of the general methods for reducing the construction costs of underwater tunnels is to shorten the tunnel lengths by increasing the longitudinal gradients in consideration of the design speeds of roads. However, the soot

concentration in the exhaust gas of vehicles sharply increases when the gradients increase, thereby increasing the costs for ventilation facilities (installation and maintenance costs). Furthermore, the longitudinal gradients affect smoke control on the occurrence of fires inside tunnels. Therefore, the longitudinal gradients of underwater tunnels shall be determined by taking these factors into consideration.

(4) The cross sections of underwater tunnels shall be determined with consideration to the traffic volumes of vehicles, the composition ratios of large vehicles, the needs for pedestrian and bicycle tracks, the evacuation passages, the types of cables and pipes in utility ducts, with or without restriction on traffic carrying hazardous materials, the presence or absence of toll gates, and the connections with other roads.

The future utilization plans of underwater tunnels should be adequately studied because it is difficult to add additional functions, such as the widening of widths, after their completion. Furthermore, in the case of the necessity to provide immersed tunnel sections with curved geometry in planar view, the inner cross sections of the immersed tunnel sections shall be appropriately set to avoid interference with clearance limits.[3)]

(5) If pedestrian and bicycle tracks and evacuation passages will be installed, due consideration shall be given to their usability for elderly and physically handicapped persons.

(6) Underwater tunnel elements shall have fire-resistant structures, fire-safety facilities, and evacuation passages. Furthermore, underwater tunnel elements shall have other safety facilities as required such as evacuation passages and emergency telephones in case of accidents and disasters. For the fire-resistant design of tunnels with concrete structures, steel members, and joint sections, **References 10)** and **11)** can be used as references.

(7) On the basis of the provisions in Article 46 of the **Road Act**, underwater tunnels can be planned and designed with a ban or restriction on vehicles loaded with volatile or combustible articles and other dangerous goods.

(8) Management facilities and equipment

Management facilities and equipment include the facilities and equipment for ventilation, emergency, lighting, electric power, security and measurement, monitoring and control, and drainage.

In cases wherein ventilation towers are constructed as ventilation facilities, it is necessary to functionally allocate ventilation, electrical, control, and ancillary equipment. It is also necessary to install inlet ports, exhaust ports, and connection ducts that connect the ventilation towers with the elements of tunnels to achieve efficient ventilation.

(9) Immersed tunnel elements (Immersed tunnel sections)

① The structural types of immersed tunnel elements are classified into steel shell type, concrete types (reinforced concrete and prestressed concrete types), and composite (hybrid) types. The most appropriate structures shall be selected with consideration to the characteristics of the respective types.

② Composite-type immersed tunnels are further classified into an open-sandwich type with a structural member comprising steel plates and reinforced concrete and a full-sandwich type with steel plates on both surfaces of the structural members.

③ For the design and construction of composite-type immersed tunnels, particularly open-sandwich type, refer to **References 12)** to **14)**.

④ Concrete types are generally constructed in dry docks, and steel and composite types are generally constructed in shipyards.

(10) The maintenance and management plans of underwater tunnels shall be established as a standard practice to efficiently and reliably implement the management of underwater tunnels. The maintenance and management plans shall also be established in accordance with the installation conditions of underwater tunnels and by referring to **Part I, Chapter 2, 4 Maintenance of Facilities Subjected to Technical Standards** and Past Cases (for example, **References 15)** and **16)**).

3.3 Determination of Basic Cross Sections

(1) Underwater sections of tunnel elements

① The top surfaces of immersed tunnel elements shall be covered with appropriate materials of the required thickness to ensure the structural safety of tunnel elements by taking into consideration the penetration depths of the anchors cast and dragged by navigating ships, the frequencies of casting and dragging of anchors, the buoyancy of tunnels, and the scouring due to waves and water flows. In principle, it is preferable that the thickness of the cover layers, including the thickness of the concrete layers to protect the upper slabs, should be 1.5 m or greater. The elevations of the top surfaces of the cover layers shall be set in consideration of the construction errors associated with dredging or backfilling work.[4)]

② The immersion depths of tunnel elements shall be appropriately set by taking into consideration any future plans to deepen the water areas above and around the tunnel elements.

③ The structural types and lengths of immersed tunnel elements shall be determined giving due consideration to sectional force, joint structures, size of fabrication yards, tunnel element installation and joint construction methods, and economical efficiency of the immersed tunnel elements, including joints. In general, the lengths of immersed tunnel elements are set at approximately 100 m, but there are cases of longer lengths so that the number of installation operations and joint locations can be reduced.[5)]

The dimensions of the inner cross section of immersed tunnel elements shall be set by taking into consideration the margins for the installation of fire-resistant materials, allowance rooms, and construction errors when fabricating and installing immersed tunnel elements.[3)]

(2) Ventilation towers

① The structures of the ventilation towers for immersed tunnels need to be studied with appropriate methods corresponding to the characteristics of the facilities and grounds on the basis of the evaluated actions.

② Ventilation machines, electrical facilities and their equipment, and control facilities and their equipment should be functionally arranged in a ventilation tower. Furthermore, the ventilation towers shall have structures with inlet ports, exhaust ports, and connection ducts that connect the ventilation towers with the tunnel elements to achieve efficient ventilation.

③ Sufficient spaces should be provided inside ventilation towers so that the monitoring, inspection, and minor repair of the installed equipment can be performed smoothly. In particular, large machines, such as ventilation machines, shall have structures that enable them to be easily carried in and out of ventilation towers.

④ The locations and structures of inlet ports shall enable the exhausting of the intake volume of the air from the exhaust ports or shall minimize the entrances of the tunnels.

⑤ The locations of exhaust ports shall ensure that the concentration of exhaust gas at ground levels remains within a allowable level.

⑥ Generally, shafts double as ventilation towers, but they can be separated.

⑦ Ventilation towers shall have a ventilation function and shall be designed giving scenery consideration to the surrounding landscapes.

⑧ The locations of ventilation towers shall be set by taking into consideration the relative displacement between the ventilation towers and tunnels owing to consolidation settlement and earthquake ground motions.

(3) Access roads

① The structures of access roads shall be generally designed with due consideration to the usage plans, natural conditions, social conditions, construction methods, and construction costs.

② The road surface elevations of the entry and exit sections of access roads shall be determined by taking into consideration the connection with other roads, the elevations of the neighboring grounds, the inflow of seawater or river water during storm surges, the water depths on the occurrence of tsunamis, and the longitudinal gradients of underwater tunnels.

③ Access roads generally comprise open-cut sections and land tunnel sections. The structures of the open-cut sections are classified into concrete and earth slope types, and concrete type is generally used. The land tunnel sections are generally constructed with the open-cut method.

3.4 Performance Verification

(1) Stability of underwater tunnels

① Considering that tunnels are considerably long in longitudinal directions, the examination of the structural stability of underwater tunnels shall be performed for both the longitudinal and transverse directions of the tunnels.

② The structural stability of underwater tunnels can be generally examined in a manner that analyzes the stability in the transverse directions on the basis of rigid frames (in the case of immersed tunnels) and the stability in the longitudinal directions on the basis of beams on elastic ground with ground reaction modeled as springs or in a manner that analyzes the entire stability, including the ground surrounding the tunnels, by using the finite element method.

③ The foundations of underwater tunnels shall be examined in terms of their bearing capacity against the weights of the tunnels and covering soil, consolidated settlement, elastic settlement, and liquefaction.[6)]

④ Although earthquake ground motions may act on underwater tunnels in all directions, the performance verification can be generally performed in two directions: the transverse directions, in which tunnels are subjected to maximum flexural moment and shear force, and the longitudinal directions, in which tunnels are subjected to maximum axial force.

⑤ It is preferable to select the appropriate types and quality of backfill materials to ensure the safety against settlement and uplift of tunnel elements, mitigate the possibility of liquefaction due to the actions of earthquake ground motions, and facilitate maintenance dredging to maintain navigation channel depths.

⑥ Given that underwater tunnels are bottom-seabed facilities, many cases of construction of underwater tunnels on soft ground have been reported. Therefore, the influence of soft ground on the cracks in concrete and water leaks through joints, which may impair the functions of underwater tunnels, need to be fully studied.

⑦ In the earthquake-resistant design, verification shall be performed on the stability against earthquake ground motions in harbors (**Part II, Chapter 6 Earthquakes**), the possibility of liquefaction, and lateral flows when ventilation towers are installed close to revetments.

⑧ In cases wherein underwater tunnels are constructed on soft ground, it is necessary to confirm that underwater tunnels do not cause the slide failures of surrounding ground owing to the actions of earthquake ground motions.

(2) Stability of the structural members of underwater tunnels

① The structural members of underwater tunnels shall have safe structures in terms of the following factors:

(a) Watertightness

(b) Cracks in concrete

(c) Uplift of structure elements due to buoyancy after installation

(d) Ventilation and disaster prevention functions

(e) Other additional functions.

② Additional functions may include waterworks, power cables, and gas conduit pipes.[7)]

③ Immersed tunnel elements

It is preferable to install waterproof-coated layers on the outside surface of immersed tunnel elements to ensure watertightness. The typical materials for waterproof-coated layers are steel plates, synthetic rubber (butyl system), and asphalt. According to the recent domestic construction of concrete immersed tunnels, steel plates are frequently used for the lower faces of floor slabs and outer faces of side walls, and synthetic rubber or steel plates are frequently used for the top faces of upper slabs. Moreover, sufficient waterproof treatment is necessary around anchor bolts that penetrate waterproof-coated layers.

(3) Underwater tunnel joints

① The joints of underwater tunnels shall have safe structures against stresses and displacement due to the actions of earthquake ground motions, settlement of surrounding ground, and temperature fluctuations.

② The locations and structural types of joints on underwater tunnels are normally determined giving due consideration to the size of fabrication yards, shifting of waterways, capacity of construction machines during construction, uneven settlement of ground and foundations, and influence of temperature variations after construction. In the assessment of the earthquake resistance of underwater tunnels, the locations and structural types of joints are also important factors. Therefore, the earthquake resistance of joints needs to be adequately examined when determining the locations and structural types of joints.

③ The joints between tunnel elements and ventilation towers shall also have safe structures against stresses and displacement due to the actions of earthquake ground motions.

④ The joint structures are largely classified into two types: a continuous structure (rigid joint) that has the same stiffness and strength as those of the cross sections of the immersed tunnel elements so that it can endure deformation, strain during permanent actions, earthquake ground motions, and other actions; a flexible structure (flexible joint) that has sufficient flexibility to absorb deformations during permanent actions, earthquake ground motions, and other actions.

⑤ Immersed tunnel elements

The popular methods for executing underwater joining and primary water sealing between immersed tunnel elements are water pressure connection and underwater concrete methods. In recent years, the water pressure connection method has become more popular than the underwater concrete method.

The flexible joint structures combining rubber gaskets and PC cables have been used in many cases, and they are classified into bellows and crown seal joints.[17] It is preferable to select the appropriate types of joints by taking into consideration the effects of uneven settlement, behavior of tunnel elements during earthquakes, locations, structures, workability, and economic efficiency of the joints. Conventionally, joints have been used only at the junctions between immersed tunnel elements. However, in recent years, they have been installed in immersed tunnel elements (other than junctions) to enable them to cope with uneven settlement and earthquake ground motions. For the mechanical property and durability of rubber gaskets, refer to **References 18)** to **20)**.

The methods for installing joints at the locations to be immersed at the very end are classified into dry work, water sealing panel, terminal block, V-block, and key element methods. It is preferable to select appropriate methods by taking into consideration their locations, structures, construction methods, and workability.

3.5 Structural Details

(1) Underwater tunnels shall be provided with the following facilities as necessary:

① Ventilation facilities

② Emergency facilities

③ Lighting facilities

④ Electric power related facilities

⑤ Security and measurement facilities

⑥ Monitoring and control facilities

⑦ Drainage facilities

(2) Ventilation is essential for underwater tunnels to prevent the adverse effects of exhaust gas from motor vehicles from accumulating inside the tunnels. Although natural ventilation may be sufficient for short tunnels, ventilation facilities shall be installed for the immersed tunnels of roads in a port.

[References]

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4 Parking Lots

(English translation of this section from Japanese version is currently being prepared.)

4.1 General

(English translation of this section from Japanese version is currently being prepared.)

4.2 Examination of Sizes and Locations

(English translation of this section from Japanese version is currently being prepared.)

4.3 Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

5 Bridges

(English translation of this section from Japanese version is currently being prepared.)

5.1 General

(English translation of this section from Japanese version is currently being prepared.)

5.2 Fundamentals of Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

5.3 Ensuring Durability

(English translation of this section from Japanese version is currently being prepared.)

5.4 Performance Verification of Fenders

(English translation of this section from Japanese version is currently being prepared.)

6 Canals

(English translation of this section from Japanese version is currently being prepared.)

6.1 General

(English translation of this section from Japanese version is currently being prepared.)

6.2 Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

7 Railroads

(English translation of this section from Japanese version is currently being prepared.)

7.1 General

(English translation of this section from Japanese version is currently being prepared.)

7.2 Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

8 Heliports

(English translation of this section from Japanese version is currently being prepared.)

8.1 General

(English translation of this section from Japanese version is currently being prepared.)

Chapter 7 Cargo Sorting Facilities

1 General

[Ministerial Ordinance] (General Provisions)

Article 41

1 The performance requirements for cargo sorting facilities shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied in light of geotechnical characteristics, meteorological characteristics, sea states and other environmental conditions, as well as the conditions of cargo handling.

2 The performance requirements for cargo sorting facilities shall be such that the facilities have stability against self-weight, waves, earthquake ground motions, surcharges, winds, etc.

[Ministerial Ordinance] (Necessary Items concerning Cargo Sorting Facilities)

Article 44

The items necessary for the performance requirements for cargo sorting facilities as specified in this Chapter by the Minister of Land, Infrastructure, Transport and Tourism and other requirements shall be provided by the Public Notice.

[Public Notice] (Cargo Sorting Facilities)

Article 81

The items to be specified by the Public Notice under Article 44 of the Ministerial Ordinance concerning the performance requirements for cargo sorting facilities shall be as provided in the following Article through Article 84.

1.1 General

(1) This chapter can be applied to the performance verification of cargo sorting facilities.

(2) For the performance verification of cargo sorting areas, refer to **References 1)** to **3)**.

[References]

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3) Civil Engineering Handbook, Part 37, Port and harbours, Giho-do Publishing, pp.1620-1621, 1989 (in Japanese)

2 Stationary Cargo Handling Equipment and Rail-Mounted Cargo Handling Equipment

[Ministerial Ordinance] (Performance Requirements for Cargo Handling Equipment)

Article 42

1 The performance requirements for stationary cargo handling equipment and rail-mounted cargo handling equipment (hereinafter referred to as "cargo handling equipment") shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied so as to ensure the safe and smooth sorting of cargo and to prevent interference with the mooring or berthing and unberthing of ships.

2 In addition to the provisions of the preceding paragraph, the performance requirements of the cargo handling equipment cited in the following items shall be as prescribed respectively in those items:

(1) "The performance requirements for cargo handling equipment (excluding petroleum cargo handling equipment, liquefied petroleum gas cargo handling equipment and liquefied natural gas cargo handling equipment, which is referred to as "petroleum cargo handling equipment, etc." in the following item)" shall be such that damage due to self-weight, Level 1 earthquake ground motions, surcharges, winds and other actions shall not adversely affect the continuous use of the cargo handling equipment or impair its function.

(2) "The performance requirements for petroleum cargo handling equipment, etc." shall be such that damage due to the self-weight of equipment, Level 1 earthquake ground motions, winds, the self-weight and pressure of petroleum, liquefied petroleum gas and liquefied natural gas and other actions shall not adversely affect the continuous use of the petroleum cargo handling equipment, etc. or impair its function.

(3) "The performance requirements for cargo handling equipment installed at high earthquake-resistance facilities" shall be such that damage due to Level 2 earthquake ground motions and other actions shall not affect restoration through minor repair works of the functions of the equipment.

[Public Notice] (Performance Criteria for Cargo Handling Equipment)

Article 82

1 The performance criteria of cargo handling equipment shall be as prescribed respectively in the following items in consideration of the type of cargo handling equipment:

(1) Cargo handling equipment shall be arranged appropriately and provided with the necessary dimensions in consideration of the design ships, types and volumes of cargo, structure of the mooring facilities, and condition of cargo handling.

(2) In order to protect the environment surrounding the facilities, cargo handling equipment shall be provided with functions appropriate for the prevention of dust, noise and other hazards as necessary.

2 In addition to the provisions specified in the preceding paragraph, the performance criteria for rail-mounted cargo handling equipment for the use of loading and unloading ships shall be such that the rail-mounted cargo handling equipment is provided with the appropriate functions to prevent runaway due to winds.

3 In addition to the provisions in the paragraph (1), the performance criteria for petroleum cargo handling equipment, liquefied petroleum gas cargo handling equipment and liquefied natural gas cargo handling equipment shall be as prescribed respectively in the following items:

(1) Under a permanent situation in which the dominating action is self-weight, the risk of impairing the integrity of structural members shall be equal to or less than the threshold level.

(2) Under a variable situation in which the dominating actions are Level 1 earthquake ground motions, winds or the weight and pressure of petroleum, liquefied petroleum gas or liquefied natural gas, the risk of impairing the integrity of structural members and losing structural stability shall be equal to or less than the threshold level.

(3) Appropriate measures shall be taken to enable petroleum cargo handling equipment, liquefied petroleum gas cargo handling equipment and liquefied natural gas cargo handling equipment to be transferred from the mooring facilities of the ships in the event of an emergency situation without issue.

4 In addition to the provision in the paragraph (1), the performance criteria for cargo handling equipment to be installed at high earthquake-resistance facilities shall be such that the degree of damage owing to the action, under accidental situation in which the dominating action is Level 2 earthquake ground motions, is equal to or

less than the threshold level.

[Interpretation]

13 Cargo Sorting Facilities

(1) Performance criteria for cargo handling equipment

① **Petroleum cargo handling equipment** (Article 42 paragraph 2 item 1 of the Ministerial Ordinance and the interpretation related to Article 82 paragraph 3 of the Public Notice)

a) Here, petroleum cargo handling equipment, liquefied petroleum gas cargo handling equipment and liquefied natural gas cargo handling equipment (only those using loading arms) are called "petroleum cargo handling equipment."

b) The performance requirements of petroleum cargo handling equipment in a permanent state in which the dominating action is the self-weight of equipment, and in a variable situation in which the dominating actions are Level 1 earthquake ground motions, winds or the weight or pressure of petroleum and other materials (such as liquefied petroleum gas and liquefied natural gas) shall be serviceability. The performance verification items and standard indexes to determine limit values with respect to the actions shall be as shown in **Attached Table 13-1**. In the performance verification of petroleum cargo handling equipment, it is necessary to appropriately set the standard indexes to determine the limit values.

Attached Table 13-1 Performance Verification Items and Standard Indexes to Determine the Limit Values of Petroleum Cargo Handling Equipment under the Respective Design States (Excluding Accidental Situations)

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design situation</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index for determining the limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>Situation</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="2">42</td><td rowspan="2">2</td><td rowspan="2">1</td><td rowspan="2">82</td><td rowspan="2">3</td><td>1</td><td rowspan="2">Serviceability</td><td>Permanent</td><td>Self-weight</td><td>Wind, earth pressure and surcharge</td><td>Soundness of members</td><td>–</td></tr>
<tr><td>2</td><td>Variable</td><td>Level 1 earthquake ground motions [Wind] [Self-weight and pressure of petroleum and other materials*1]</td><td>Self-weight, earth pressure and surcharge</td><td>Soundness of members, stability of the structure</td><td>–</td></tr>
</table>

* [ ] means alternative dominating actions to be studied as design states.

*1: The term "other materials" means liquefied petroleum gas and liquefied natural gas.

c) In **Attached Table 13-1**, the performance verification of the members of petroleum cargo handling equipment shall be carried out by appropriately setting the limit value with respect to their soundness. In addition, the performance verification of the structures of petroleum cargo handling equipment shall be carried out by appropriately setting the performance criteria with respect to stability depending on the structural types.

② **Cargo handling equipment installed at high earthquake-resistance facilities** (Item 1, Paragraph 2, Article 42 of the Ministerial Ordinance and Interpretation related to Paragraph 4, Article 82 of the Public Notice)

The performance requirements of cargo handling equipment installed at high earthquake-resistance facilities in an accidental situation in which the dominating action is Level 2 earthquake ground motions shall be restorability. The performance verification items and standard indexes to determine the limit values with respect to the actions shall be as shown in **Attached Table 13-2**. In **Attached Table 13-2**, the term "damage" is used for performance verification items as a comprehensive expression for the verification items which differ depending on the types, structures and structural types of cargo handling equipment. In the performance verification of cargo handling equipment, it is necessary to appropriately set the standard indexes to determine the limit values.

Attached Table 13-2 Performance Verification Items and Standard Indexes to Determine the Limit Values of Cargo Handling Equipment Installed at High Earthquake-Resistance Facilities in Accidental Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
42	2	1	82	4	–	Restorability	Accidental	Level 2 earthquake ground motion	Self-weight and earth pressure	Damage	–

2.1 General

(1) In the Ports and Harbor Act, cargo handling equipment as port facilities is classified into stationary cargo handling equipment and rail-mounted cargo handling equipment, which are cargo sorting facilities, and movable cargo handling equipment, which are movable facilities. In this Chapter, stationary, rail-mounted, and movable cargo handling equipment are collectively called "cargo handling equipment."

(2) Cargo handling equipment as port facilities means equipment which is used for cargo handling work and which includes stationary, rail-mounted and mobile cranes, as well as unloaders, conveyors, forklifts, bulldozers, tractors, straddle carriers, reach stackers, log loaders and loading arms, but excludes on-board equipment and vehicle boarding facilities for roll-on/roll-off (RORO) ships. For movable cargo handling equipment, refer to **Part III, Chapter 10, 1 Movable Cargo Handling Equipment**.

(3) The purpose of introducing cargo handling equipment in ports is to achieve laborsaving, fast and safe cargo handling operation. The selection of the types, structures and capabilities of cargo handling equipment is preferably made by sufficiently considering the design ships and types, shapes, volumes and particulars of the cargo, as well as the relationship with the yard facilities located behind them and the modes of secondary transportation.

(4) The cargo handling equipment to be installed in cargo sorting areas or mooring facilities shall have the optimal structures, capabilities and placements for the usage patterns of those areas and facilities, and will ensure structural safety, as well as the functions adequate to prevent public hazards such as dust and noise, and smooth and safe cargo handling operation.

(5) When installing cargo handling equipment, it is necessary to ensure that the areas where cargo handling equipment travels, turns or moves up and down do not obstruct buildings and electric wires, and that the cargo handling equipment is prevented from making accidental contact with ships traveling alongside or leaving the facilities, or ships being moored at berths.

(6) The cargo handling equipment and the ancillary facilities for cargo handling equipment (such as electric equipment, rails and ditches) are preferably installed so as not to reduce the safety of ships traveling alongside the facilities or leaving berths.[1)]

(7) In the case of possible cargo handling operation without using cargo handling equipment, it is preferable to examine the locations and structures of the cargo sorting facilities so as not to prevent smooth and safe execution of cargo sorting operation.

(8) Because bulk cargo operation is likely to produce noise and dust, cargo handling equipment for bulk cargo shall be equipped with dust and noise prevention measures as standard practice. In particular, as a general rule, explosion-proof measures are required for equipment handling inflammable dust.

(9) Related laws, regulations and guidelines

① The International Standards, regulations and guidelines related to cargo handling equipment are as follows:

(a) Cranes-Design principles for loads and load combinations-Part1: General (ISO 8681-1, 2012)

(b) **Cranes-principles for seismic ally resistant design** (ISO 11031, 2016)

(c) The **Safety Ordinance for Cranes** (Ordinance of the Ministry of Labour No. 34 of 1972)

(d) The **Structural Standards for Cranes** (Public Notice of Ministry of Labour No. 134 of 1995)

(e) The **Structural Standards for Mobile Cranes** (Public Notice of Ministry of Labour No. 135 of 1995)

(f) The **Structural Standards for Derricks** (Public Notice of Ministry of Labour No. 55 of 1962)

(g) **Cranes-Design Principles for Loads and Load Combinations** (JIS B 8831, 2004)

(h) **Calculation Standards for Steel Structures of Cranes** (JIS B8821, 2013)

(i) **Cranes-Anchoring devices for in-service and out-of-service conditions. Part1: General** (JIS B 8828-1, 2013)

(10) For basic information on cargo handling equipment in ports, refer to the **Directory of Port Cargo Handling Machinery**.[2)]

2.2 Container Cranes

2.2.1 General

(1) Container cranes are a type of rail-mounted cargo handling equipment installed at mooring facilities to directly transfer containers between the container ships and mooring facilities.

(2) Container cranes shall have structures complying with the items to be considered in accordance with the characteristics of ports and the safety standards for cranes as based on the **Industrial Safety and Health Act**.

2.2.2 Fundamentals of Performance Verification

(1) In the performance verification of container cranes, the **Structural Standards for Cranes** can be used as a reference except for the items to be specifically required in accordance with the characteristics of ports.

(2) The items to be specifically required in accordance with the characteristics of ports are shown in **Part III, Chapter 7, 2.2.3 Performance Verification of Earthquake Resistance** and **Part III, Chapter 7, 2.2.4 Appropriate Functions to Prevent Runaway Due to Winds**.

(3) The terms regarding the loads applied to structures such as the actions of self-weight, Level 1 earthquake ground motions, surcharges and winds shall be appropriately replaced by the terms specified in **Structural Standards for Cranes**, **Article 8 (Types of Loads to Be Used for Calculations)**. (Refer to **Commentaries for Structural Standards for Cranes and Other Equipment**[3)].)

(4) Although the design of container cranes is normally carried out after or parallel to the design of the mooring facilities, the process of ensuring the performance requirements of the container cranes is subjected to the ensuring of the performance requirements of the mooring facilities on which the container cranes are to be installed. Thus, in the performance verification of the container cranes, it is necessary to ensure that the mooring facilities are designed based on the design conditions including the actions of the container cranes on the mooring facilities.

2.2.3 Performance Verification of Earthquake Resistance

(1) Points of caution when carrying out the verification of earthquake resistance of container cranes

① Earthquake-resistant performance of container cranes in consideration of mooring facilities

Because container cranes for the use of loading and unloading container ships function in tandem with the mooring facilities, the earthquake resistance of the container cranes shall also be verified integrally with the mooring facilities. During the occurrence of an earthquake, the container cranes and mooring facilities interact with each other. For example, the deformation of mooring facilities due to an earthquake may expand the rail spans and cause damage to the container cranes.[4)] Therefore, the verification of earthquake resistance of both container cranes and mooring facilities shall be carried out with due consideration to such interactions. In cases where the rail spans are expected to be expanded to a length larger than the allowable elastic deformation ranges for leg sections, the container cranes need to be provided with the adequate mechanisms, if necessary, to keep any possible damage to the crane bodies within the levels that satisfy the predetermined performance requirements.

② Prevention of uplift of the leg sections of container cranes and earthquake-resistant performance of members

The members of container cranes shall be prevented from damage due to Level 1 earthquake ground motions, which may adversely affect the continued use of the container cranes. The members of container cranes installed at earthquake-resistance facilities shall be prevented from damage due to Level 2 earthquake ground motions, which may adversely affect the restoration of functions through minor repair work.

Rail-mounted container cranes have a high risk of receiving damage to their leg sections when subjected to uplift of the leg sections due to earthquakes. Thus, in order to fulfill the required serviceability, rail-mounted container cranes shall be basically prevented from uplift of the leg sections. Furthermore, in order to fulfill the required restorability, in the case of seismically isolated container cranes, the leg sections shall be basically prevented from complete uplift, though uplift in a state where some wheel flanges contact with rails is allowed.

The performance verification of the uplift of leg sections and members of the container cranes shall be carried out based on the predetermined natural periods and damping constants of the container cranes. For example, although the natural periods in cross-shore directions of container cranes that do not have seismic isolation mechanisms vary depending on the sizes and types of the container cranes, they approximately range from 1.5 to 3 seconds.[5)] In contrast, the natural periods of the container cranes with seismic isolation mechanisms are about 4 seconds. Based on these natural periods, the performance of the seismic isolation mechanisms for container cranes is often set so as to enlarge the damping constants.

When the predominant periods of earthquake ground motions at the container crane installation locations are close to the natural periods of the container cranes and the damping constants of the container cranes are small, the container cranes are likely to undergo uplift of the leg sections and receive damage to the members as a result of the generation of large responses. In such cases, modifying the container cranes so that they have natural periods that are not close to the dominant periods of earthquake ground motions is considered to be an effective measure to improve earthquake resistance. In addition, the application of seismic isolation or vibration control mechanisms to container cranes is an effective measure to reduce response acceleration and thereby prevent the uplift of leg sections and damage to members. However, it shall be noted that large earthquakes are likely to have longer dominant periods than small or medium-sized earthquakes due to the effect of the non-linear behavior of surface layers.

For details regarding the performance verification of container cranes, refer to the descriptions about performance verification of the respective facilities in this book, **Guideline for the Earthquake Resistant Design of Container Cranes**,[6)] and **Reference 7)**.

(2) Fundamentals of performance verification of container cranes in respect to Level 1 earthquake ground motions

The performance verification of container cranes in respect to Level 1 earthquake ground motions can be carried out according to the procedure shown in **Fig. 2.2.1**.

① Setting of Level 1 earthquake ground motions

Level 1 earthquake ground motions can be set based on **Part II, Chapter 6, 1.2 Level 1 Earthquake Ground Motions Used in the Performance Verification of Facilities**, which can be downloaded from the homepage of the National Institute for Land and Infrastructure Management (http://www.ysk.nilim.go.jp/kakubu/kouwan/sisetu/sisetu.html).

② **Calculation of the maximum response acceleration of container cranes**

(a) In the cases of mooring facilities other than piled piers, the earthquake ground motions at rail installation positions can be obtained by calculating the time history of ground surface acceleration according to **Part II, Chapter 6, 1.2.3 Earthquake Response Calculation of Surface Ground**. The earthquake response calculation may be one-dimensional earthquake response calculation with respect to the ground at the rear of the mooring facilities. Then, the maximum response acceleration of container cranes needs to be calculated with the calculated earthquake ground motions input into container cranes modeled as lumped mass systems.

(b) In the case of piled piers, because the dynamic interaction between the piled piers and container cranes needs to be considered, piled piers and container cranes are modeled as dual lumped mass systems. Next, the time history of ground acceleration at levels $1/\beta$ below the virtual ground surfaces of the piled piers can be calculated according to **Part II, Chapter 6, 1.2.3 Earthquake Response Calculation of Surface Ground**. Then, the maximum response acceleration of container cranes needs to be calculated with the calculated time history of ground acceleration input into the dual lumped mass systems. For the virtual ground surfaces and $1/\beta$, refer to **Part III, Chapter 5, 5.2.2 Setting of Basic Cross Sections**. The concept of the calculation model of a dual lumped mass system is shown in **Fig. 2.2.2**, where k is the equivalent stiffness of piles on 1 block of a piled pier; m is the mass of the 1 block of the superstructure; c is the damping constant; k_c is the equivalent stiffness of the container crane; m_c is the mass of the container crane; and c_c is the damping constant of the container crane. The equivalent stiffness is obtained by converting the stiffness of all the piles on 1 block of the piled pier into springs of a lumped mass system and the stiffness of all the legs of the container crane into additional springs of a lumped mass system as shown in **Fig. 2.2.2**.

(c) In both cases of (a) and (b) above, for the vibratory characteristics, the natural periods shall be adjusted to those of the actual container cranes. The damping constants of the container cranes may be set at 1 to 3% if individual data is not available. In cases where seismic isolation mechanisms are expected to be introduced, lumped mass systems need to have the appropriate stiffness and damping constants corresponding to the mechanisms. When the specific dimensions of the container cranes are unknown, the vibratory characteristics can be set at levels acceptable for the manufacturers of the container cranes.

③ **Examination of whether the leg sections of the container cranes undergo uplift**

(a) Whether or not the leg sections of the container cranes undergo uplift can be examined through the seismic coefficient method in a manner that calculates values (hereinafter referred to as the "design value of seismic coefficient") by dividing the maximum response acceleration of the container cranes by gravitational acceleration, and applies the loads obtained by multiplying the vertical static loads of the container cranes by the design value of seismic coefficient to the container cranes in a horizontal direction.

(b) When it is determined that the leg sections of the container cranes are subjected to uplift, it is necessary to change the dimensions of the container cranes and go back to procedure ② above, or review the determination results through dynamic response analyses. In the latter case, analyses can be made according to (3) below with the introduction of a seismic isolation mechanism as needed. When it is determined that the leg sections are not subjected to uplift, the performance verification can proceed to ④ below.

④ **Verification of stresses on members**

(a) When the design value of seismic coefficient is not more than 0.2, container cranes conforming to the **Structural Standards for Cranes** are considered to satisfy the performance requirements in respect to Level 1 earthquake ground motions at ports.

(b) When the design value of seismic coefficient is larger than 0.2, the performance verification shall be carried out with respect to the stresses on members through the methods specified in the **Structural Standards for Cranes** with the loads obtained by multiplying the vertical static loads of the container cranes by the design value of seismic coefficient applied to the container cranes in a horizontal direction. If any members fail the performance verification, it is necessary to change the members concerned and go back to procedure ② above. If no members fail, the performance verification can be completed.

(c) When designing container cranes, the following analysis results of the values obtained by dividing the least horizontal loads causing members to have stresses larger than those allowable by the vertical static loads of container cranes (hereinafter referred to as "crane limit seismic coefficient") can be used as a reference. The analysis was conducted in a manner that applies static horizontal loads to actual container cranes designed in accordance with the **Structural Standards for Cranes**. According to the analysis results, in many cases, the crane limit seismic coefficient ranges from 0.20 to 0.29 depending on the characteristics of the container cranes (based on the most critical among the various situations such as a suspension or operation situation). The analysis result above indicates that when the design value of seismic coefficient is larger than 0.20 to 0.29, the container cranes designed only based on the **Structural Standards for Cranes** may not satisfy the performance requirements in respect to Level 1 earthquake ground motions. In additional analyses of crane limit seismic coefficient using the actual container cranes having a crane limit seismic coefficient of 0.20 to 0.25, it was found that partial reinforcement, such as the changes in cross sectional dimensions of diagonal or horizontal members between crane legs, can increase the crane limit seismic coefficient to a level larger than 0.25. Thus, even container cranes having a design value of seismic coefficient of 0.20 to 0.25 can be reinforced through the modification of members to a level which satisfies the performance requirements in respect to Level 1 earthquake ground motions.

However, when the design value of seismic coefficient is larger than 0.25, there may be cases where partial reinforcement does not help the container cranes satisfy the predetermined performance requirements. In such cases, dynamic response analyses can be employed with reference to (3) below with the introduction of seismic isolation mechanisms as needed.

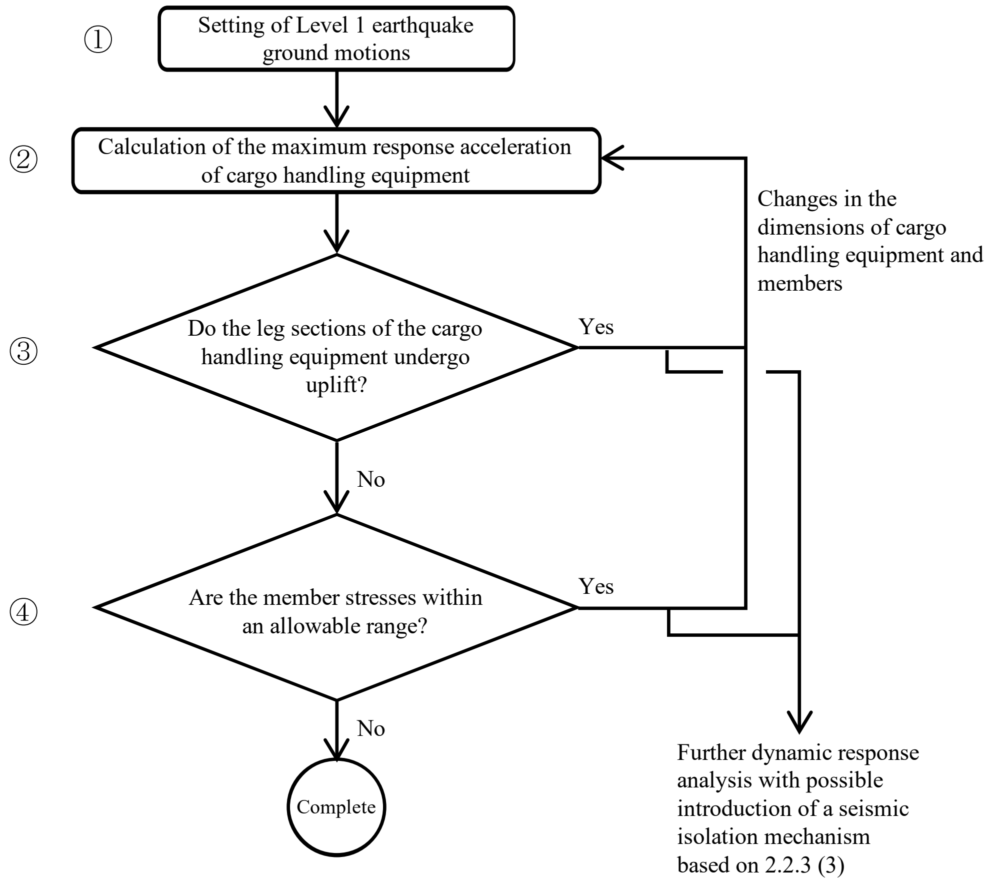

Fig. 2.2.1 Flowchart of Performance Verification of Container Cranes in Respect to Level 1 Earthquake Ground Motions

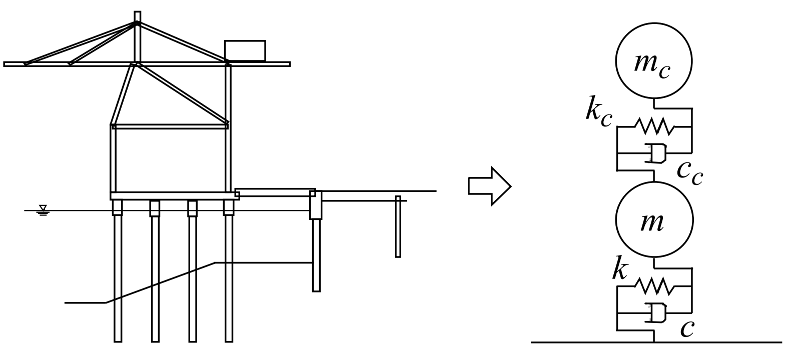

Fig. 2.2.2 Concept of the Calculation Model of the Dual Lumped Mass System of Piled Piers and Container Cranes

(3) Fundamentals of performance verification of container cranes in respect to Level 2 earthquake ground motions

The performance verification of container cranes in respect to Level 2 earthquake ground motions can be carried out by the following procedures.

① **Setting of Level 2 earthquake ground motions**

Level 2 earthquake ground motions can be set with reference to **Part II, Chapter 6, 1.3 Level 2 Earthquake Ground Motions for the Performance Verification of Facilities**.

② **Input earthquake ground motions for container cranes**

The input earthquake ground motions for container cranes can be obtained by calculating the time history of ground surface acceleration at rail installation positions through seismic response analyses of the ground and structures. Examination of the dynamic interactions between container cranes and mooring facilities, if necessary, shall be carried out by using lumped mass models capable of reproducing the natural periods with proper combinations of mass and stiffness, the damping constants, and the gravity center positions of the container cranes. In the case of seismic isolated container cranes, the calculation models shall have characteristics acceptable for the manufacturers of the container cranes concerned.

③ **Examination of uplift of the leg sections, member stresses and seismic isolation mechanisms of container cranes**

The performance verification of container cranes in respect to Level 2 earthquake ground motions shall be carried out for the uplift of leg sections, member stresses and the necessity of seismic isolation mechanisms through dynamic response analyses or other means. For the performance verification of seismic isolation mechanisms, it is necessary to confirm that the response values of loads and displacement are equal to or less than those allowable for planned seismic isolation mechanisms.

In addition, depending on the types of mooring facilities and rail foundations, Level 2 earthquake ground motions may cause rail spans to fluctuate. In such cases, seismic isolation mechanisms can effectively absorb the fluctuation of rail spans in accordance with elastic deformation of the leg sections of the container cranes. For example, when the elastic deformation range of the leg sections of a container crane having a rail span of 30.5 m is about 200 mm, as shown in **Fig. 2.2.3** (which is used here only as reference since different container cranes have different elastic deformation ranges of leg sections), the seismic isolation mechanism can increase the allowable displacement of the rail span to about 500 mm with the displacement stroke of a of about 300 mm (which is also used here only as reference since different seismic isolation mechanisms have different strokes). However, it is necessary to confirm that the increased displacement due to seismic isolation mechanisms does not cause the container cranes to collide with any ships.

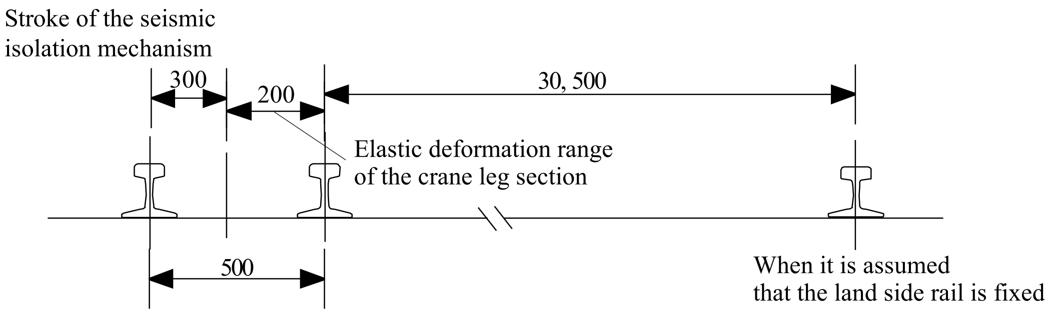

Fig. 2.2.3 Example of the Relationship between the Deformation of the Leg Sections of the Container Cranes and the Displacement of the Rail Span

Note: The seismic isolation mechanisms mentioned in this section are those which have been commercialized at this time. When applying newly developed technologies to the container cranes concerned, the performance verification shall be carried out in compliance with this section.

2.2.4 Appropriate Functions to Prevent the Runaway of Container Cranes Due to Winds

(1) Container cranes shall have buffer stops and power engines having output enabling the container cranes to be moved even against winds of predetermined intensity as appropriate functions to prevent them from running away due to winds.

It is also preferable that container cranes be provided with buffer stop-related devices, which can be used as runaway prevention measures, and vane anemometers to appropriately monitor wind conditions.

For more information, refer to the **Operational Regulations to Prevent the Runaway of Container Cranes**.[8)]

(2) Buffer stops

The **Structural Standards for Cranes, Article 41 (Buffer Stops)** stipulates that buffer stops shall have the capability to resist wind loads equivalent to a wind speed of 60 m/s. Buffer stops include anchors and other related devices, such as rail cramps. In **Cranes-Anchoring devices for in-service and out-of-service conditions, Part 1: General (JIS B 8828-1: 2013)**, rail cramps are required to have the capability to resist wind loads equivalent to a wind speed of 35 m/s.

(3) Buffer stop-related devices

Buffer stop-related devices are devices to be used as measures to prevent the runaway of container cranes, and include brakes, rail brakes and crane stoppers. Examples of rail brakes and crane stoppers are shown in **Figs. 2.2.4** and **2.2.5**, respectively.

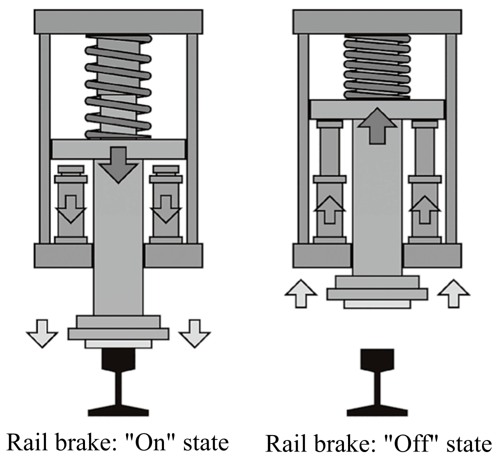

Fig. 2.2.4 Rail Brakes

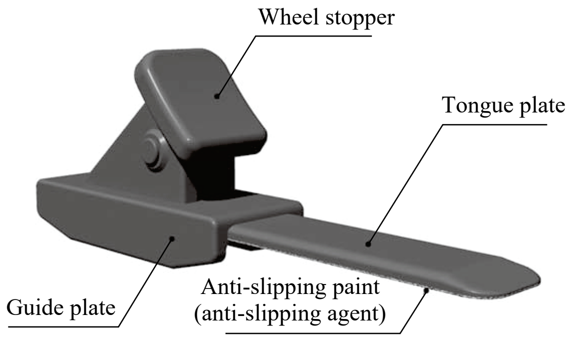

Fig. 2.2.5 Crane Stopper

(4) Power engines

The **Structural Standards for Cranes, Article 42 (Power Engines)** stipulates that container cranes shall be provided with power engines having output enabling the container cranes to be moved against winds of 16 m/s.

(5) Vane anemometers

Vane anemometers are preferably installed at locations not affected by container cranes in order to be used for determining the suspension of cargo handling operation, the implementation of measures to prevent the runaway of rail-mounted cargo handling equipment, and the resumption of cargo handling operation.

(6) Prevention of the overturning of container cranes due to winds

Measures to prevent the overturning of container cranes due to winds shall conform to the **Structural Standards for Cranes, Article 15 (Stability)**.

2.3 Unloaders

(English translation of this section from Japanese version is currently being prepared.)

2.3.1 General

(English translation of this section from Japanese version is currently being prepared.)

2.3.2 Fundamentals of Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

2.3.3 Performance Verification of Earthquake Resistance

(English translation of this section from Japanese version is currently being prepared.)

2.3.4 Appropriate Functions to Prevent the Runaway of Unloaders Due to Winds

(English translation of this section from Japanese version is currently being prepared.)

2.4 Loading Arms (Stationary Cargo Handling Equipment)

(English translation of this section from Japanese version is currently being prepared.)

2.4.1 General

(English translation of this section from Japanese version is currently being prepared.)

2.4.2 Fundamentals of Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

2.4.3 Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

2.5 Rubber Hoses for Petroleum Transportation (Stationary Cargo Handling Equipment)

(English translation of this section from Japanese version is currently being prepared.)

2.5.1 General

(English translation of this section from Japanese version is currently being prepared.)

2.5.2 Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

2.6 Petroleum, LPG and LNG Conduit Pipes (Stationary Cargo Handling Equipment)

(English translation of this section from Japanese version is currently being prepared.)

2.6.1 General

(English translation of this section from Japanese version is currently being prepared.)

2.6.2 Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

2.7 Maintenance of Stationary Cargo Handling Equipment and Rail-Mounted Cargo Handling Equipment[9), 10), 11)]

(English translation of this section from Japanese version is currently being prepared.)

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3 Cargo Sorting Areas

(English translation of this section from Japanese version is currently being prepared.)

3.1 General

(English translation of this section from Japanese version is currently being prepared.)

3.2 Cargo Sorting Areas for Timber

(English translation of this section from Japanese version is currently being prepared.)

3.3 Sorting Facilities for Marine Products

(English translation of this section from Japanese version is currently being prepared.)

3.4 Sorting Facilities for Hazardous Cargoes

(English translation of this section from Japanese version is currently being prepared.)

3.5 Container Terminal Areas

(English translation of this section from Japanese version is currently being prepared.)

4 Sheds

(English translation of this section from Japanese version is currently being prepared.)

4.1 General

(English translation of this section from Japanese version is currently being prepared.)

Chapter 8 Storage Facilities

(English translation of this section from Japanese version is currently being prepared.)

1 General

(English translation of this section from Japanese version is currently being prepared.)

2 Warehouses

(English translation of this section from Japanese version is currently being prepared.)

3 Open Storage Yards

(English translation of this section from Japanese version is currently being prepared.)

4 Timber Storage Yards and Ponds

(English translation of this section from Japanese version is currently being prepared.)

5 Coal Storage Yards

(English translation of this section from Japanese version is currently being prepared.)

6 Yards for Hazardous Cargoes

(English translation of this section from Japanese version is currently being prepared.)

7 Oil Storage Facilities

(English translation of this section from Japanese version is currently being prepared.)

Chapter 9 Facilities for Ship Service

1 General

[Ministerial Ordinance] (Performance Requirements for Facilities for Ship Service)

Article 47

1 The performance requirements for facilities for ship service shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied for the provision of safe and smooth services to ships in light of the geotechnical characteristics, meteorological characteristics, sea states and other environmental conditions, as well as the conditions of ship entry.

2 The performance requirements for water supply facilities for ships shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied for the sanitary supply of water to ships.

3 The performance requirements for ship storage facilities shall be as prescribed respectively in the following items:

(1) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable ships to be safely brought in and out of the facilities.

(2) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable ships to be properly secured to the facilities.

[Ministerial Ordinance] (Necessary Items concerning Facilities for Ship Service)

Article 48

The items necessary for enforcement of the performance requirements for ship service facilities as specified in this Chapter by the Minister of Land, Infrastructure, Transport and Tourism and other requirements shall be provided by Public Notice.

[Public Notice] (Facilities for Ship Service)

Article 89

The items to be specified by Public Notice under Article 48 of the Ministerial Ordinance concerning the performance requirements of ship service facilities shall be as specified in the following Article.

1.1 Water Supply Facilities for Ships

[Public Notice] (Performance Criteria of Water Supply Facilities for Ships)

Article 90

The performance criteria of water supply facilities for ships shall be as prescribed respectively in the following items:

(1) The water supply facilities shall be installed at appropriate locations corresponding to the conditions of use by ships.

(2) The water supply facilities shall have the appropriate water supply capacity corresponding to the dimensions of the design ships.

(3) The water supply facilities shall have structures which are capable of preventing water pollution, and water hydrants shall be maintained in a sanitary condition.

(1) The layout and capacity of the hydrants shall be determined appropriately according to the types of ships.

(2) Water supply facilities shall meet the following sanitation requirements:

① The hydrants of water supply facilities shall have structures that can prevent water pollution.

② Periodic and ad-hoc water quality tests shall be conducted in accordance with Article 15 of **the Ordinance for Enforcement of the Water Supply Act** (the final revision of the Ordinance of the Ministry of Health, Labour and Welfare of No. 133 of August 31, 2015), and the hygiene of the hydrants of the water supply facilities shall be appropriately maintained.

(3) The hydrant outlets shall be positioned so as to enable hoses to be easily attached and shall have structures that prevent water pollution. In particular, the hydrant outlets shall be provided with drain systems when they are buried below apron floors and shall be provided with caps.

(4) Water Supply Volumes

For the water supply volumes to ships, refer to the values shown in **Table 1.1.1**. In the case of large ships, the capacities of water tanks are in many cases around 800 m^3 because such ships have their own fresh water production equipment.

(5) Hydrants, Oil Feed Plugs and Water Supply Pipes

Hydrants are preferably positioned as close to the face lines of mooring facilities as possible. Hydrants are normally buried below the pavement surface with manholes which are closed when not in use. Furthermore, water supply pipes are preferably buried below the pavement surface or suspended from superstructures in the case of piled piers so as not to interfere with vehicle traffic and cargo handling operation. However, hydrants, oil feed plugs and water supply pipes are preferably provided with the appropriate corrosion prevention measures to prevent damage due to salt injury. In addition, oil feed pipes shall be carefully connected to prevent leakage due to the vibration and settlement of the mooring facilities. The water volumes in **Table 1.1.1** correspond to 80% of the capacity of the water tanks, and in many cases, ships only receive about half that amount at ports of call.

Table 1.1.1 Hydrants and Water Supply Volumes

Tonnage of ship (gross tonnage)	Required water supply volume (m^3)	Time required to supply water (h)	Hydrant spacing (m)	Number of hydrants per berth (number of points)	Water supply capacity of each hydrant (m^3/h)
500	40	5	30	2	4
1,000	80	5	30–40	2	8
3,000	250–300	5	40–50	3–4	16
5,000	500	5	40–50	4	18
10,000	800	5	40–50	4	28

1.2 Other Facilities for Ship Service

(1) For the performance verification of other facilities for ship service, refer to **Part III, Chapter 7, Cargo Sorting Facilities and other equivalent provisions**.

(2) Port management bodies shall take the necessary measures to establish systems obliging ocean shipping companies, shipping agents and waste disposers to receive waste generated by ships under the MARPOL Convention, which was revised in January 2013, so as to prohibit in principle the ocean disposal of waste generated by ships.

(3) Facilities for ship service shall be provided with passages enabling ships to be safely brought in and out of the facilities and equipment to secure the ships to the facilities.

Chapter 10 Mobile Facilities

[Ministerial Ordinance] (General Rule)

Article 49

The performance requirements for mobile facilities shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied in light of the geological, meteorological, hydrographic and other natural conditions, cargo handling conditions, and usage conditions of passengers.

[Public Notice] (Mobile Facilities)

Article 91

The items to be specified by Public Notice under Article 52 of the Ministerial Ordinance concerning the performance requirements for mobile facilities shall be as prescribed in the following Article and Article 93.

1 Mobile Cargo Handling Equipment

[Ministerial Ordinance] (Required Performance of Mobile Cargo Handling Equipment)

Article 50

The performance requirements for mobile cargo handling equipment shall be as prescribed respectively in the following items so as to facilitate safe and smooth cargo handling operations in consideration of the structural type.

(1) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable the safe and smooth cargo handling operations.

(2) Damage due to the actions of self-weight, Level 1 earthquake ground motions, surcharge loads and winds, etc. shall not impair the function of the mobile cargo handling equipment and shall not adversely affect the continuous use of the mobile cargo handling equipment.

[Public Notice] (Performance Criteria of Mobile Cargo Handling Equipment)

Article 92

The performance criteria of mobile cargo handling equipment shall be as prescribed respectively in the following items in consideration of the type of cargo handling equipment:

(1) The mobile cargo handling equipment shall be arranged appropriately and shall be provided with the necessary dimensions in consideration of the design ships, the types and volumes of cargo, the structural type of the mooring facilities and the conditions of cargo handling operations.

(2) The mobile cargo handling equipment shall be provided with functions appropriate for the prevention of dust, noise, etc. as necessary so as to contribute to conservation of the environment surrounding the facilities.

(3) The mobile cargo handling equipment shall be provided with the measure appropriate for collision prevention as necessary so as to enable the safe and smooth cargo handling operations.

[Interpretation]

14 Mobile Facilities

(1) Performance Criteria of Mobile Cargo Handling Equipment (Article 50 of the Ministerial Ordinance and the interpretation related to Article 92 of the Public Notice)

① Mobile cargo handling equipment as specified in the Ministerial Ordinance, Public Notice and this transmittal means mobile cargo handling equipment which can handle cargo automatically or by remote control as specified in Article 19 of the Enforcement Order of the Port and Harbour Act.

② The appropriate collision prevention measures mean the measures, such as sensors and emergency stop systems installed on mobile cargo handling equipment, which are to be selected through a comprehensive study of their use conditions including their operation systems.

1.1 General

(1) Mobile cargo handling equipment in the Port and Harbour Act means cargo handling equipment which has rubber tires or other means of mobility so as to be mobile without being constrained by rails, and which is different from mobile cranes (defined as those cranes which can be moved to any location and are mounted with power engines) as specified in Item 8, Paragraph 1, Article 1 of the Enforcement Order of the Industrial Safety and Health Act. For example, a rubber tired gantry crane (RTG) is classified as a crane based on the Industrial Safety and Health Act.

The provisions in this Chapter are intended for mobile cargo handling equipment which can be operated automatically or by remote control (hereinafter referred to as "automated or remotely controlled mobile cargo handling equipment").

(2) Related Laws, Regulations and Guidelines

① The laws, regulations and guidelines related to mobile cargo handling equipment are as follows:

(a) **The Safety Ordinance for Cranes** (Ordinance of the Ministry of Labour No. 34 of 1972)

(b) **The Structural Standards for Cranes** (Public Notice of Ministry of Labour No. 134 of 1995)

(c) **The Structural Standards for Mobile Cranes** (Public Notice of Ministry of Labour No. 135 of 1995)

(d) **The Structural Standards for Derricks** (Public Notice of Ministry of Labour No. 55 of 1962)

(e) **The Structural Standards for Forklifts** (Public Notice of Ministry of Labour No. 89 of 1972)

(f) **The Structural Standards for Shovel Loaders and Other Equipment** (Public Notice of Ministry of Labour No. 136 of 1978)

(g) **The Structural Standards for Straddle Carriers** (Public Notice of Ministry of Labour No. 137 of 1978)

(h) **Cranes – Design Principles for Loads and Load Combinations** (JIS B 8831, 2004)

(i) **Calculation Standards for Steel Structures of Cranes** (JIS B 8821, 2013)

(j) **The Technical Standards for Electric Facilities** (Ordinance of Ministry of International Trade and Industry No. 61 of 1965)

② **The Safety Ordinance for Cranes**, established based on **the Industrial Safety and Health Act** (Act No. 57 of 1972) to ensure the safety of cranes, consistently specifies the requirements during the life cycle of cranes including their production, installation, use and disposal.

③ **The Structural Standards for Cranes, Mobile Cranes, Derricks, Forklifts, Shovel Loaders and Straddle Carriers**, established based on **the Industrial Safety and Health Act**, specify the safety standards for the structural and mechanical sections of cranes and wire ropes.

④ **Cranes – Design Principles for Loads and Load Combinations** and **Calculation Standards for Steel Structures of Cranes** specify the requirements for the performance verification, production, transportation, installation and testing of cranes.

⑤ **The Technical Standards for Electric Facilities**, established based on **the Electricity Business Act** (Act No. 170 of 1964), specify the technical standards for electric facilities.

(3) For items concerning cargo handling equipment as port facilities, refer to **Part III, Chapter 7, 2.1 (1) to (8)**.

1.2 Mobile Cargo Handling Equipment for the Use of Cargo Handling in Container Yards

1.2.1 General

(1) Mobile cargo handling equipment for the use of cargo handling in container yards means mobile cargo handling equipment for transferring, loading and unloading cargo such as containers in container terminals and on wharves.

(2) The structures of mobile cargo handling equipment for the use of cargo handling in container yards shall conform to the items considered in accordance with the characteristics of the cargo sorting areas and safety standards for the respective cargo handling equipment based on **the Industrial Safety and Health Act**.

1.2.2 Fundamentals of Performance Verification

(1) The performance verification of mobile cargo handling equipment for the use of cargo handling in container yards can be carried out by referring to **the Structural Standards for Cranes**.

(2) In addition to the provision of the preceding paragraph, the performance verification shall be carried out for the functions of mobile cargo handling equipment in accordance with the characteristics of the port , which is specified in **1.2.3 Appropriate Collision Prevention Functions for Automated or Remotely Controlled Mobile Cargo Handling Equipment** below.

1.2.3 Appropriate Collision Prevention Functions for Automated or Remotely Controlled Mobile Cargo Handling Equipment

(1) Automated or remotely controlled mobile cargo handling equipment shall be provided with the appropriate functions to prevent collisions and to prevent automatic or remote controlled operation in emergency situations.

(2) Collision Prevention Devices

Collision prevention devices mean devices such as barriers, fences and sensors, which have the function of preventing cargo handling equipment from colliding with persons, vehicles or other cargo handling equipment.

(3) Appropriate Functions to Prevent Cargo Handling Equipment from Being Operated Automatically or by Remote Control in emergency situations.

Automated or remotely controlled mobile cargo handling equipment shall be provided with the appropriate functions capable of immediately stopping the equipment, even in emergency situations such as a breakdown in command through wireless communication.

(4) Operation of Cargo Handling

Automated or remotely controlled mobile cargo handling equipment shall be operated with clear distinction between the areas for automated or remotely controlled cargo handling and the areas for manual cargo handling.

1.2.4 Points of Caution When Introducing Automated or Remotely Controlled Mobile Cargo Handling Equipment

(1) When introducing automated or remotely controlled mobile cargo handling equipment for the purposes of ensuring safety and improving the efficiency of cargo handling operation, it is necessary to construct and maintain the pavement in areas for automated or remotely controlled cargo handling operation by paying closer attention to gradients and other factors than to the pavement in normal cargo sorting areas because unevenness in the paved surfaces will affect the accuracy of automated or remotely controlled mobile cargo handling.

1.2.5 Maintenance of Automated or Remotely Controlled Mobile Cargo Handling Equipment

(1) For the specific inspection and diagnosis methods and contents of maintenance and repair methods for the maintenance of mobile cargo handling equipment, refer to **the Guidelines for the Establishment of a Maintenance Plan for Port Cargo Handling Equipment**[1)] and **the Guidelines for the Inspection and Diagnosis of Port Cargo Handling Equipment**.[2)] For automated or remotely controlled mobile cargo handling equipment, it is necessary to give due consideration to the maintenance of collision prevention devices.

(2) For hazard prevention measures for automated or remotely controlled mobile cargo handling equipment, it is preferable to establish operation rules which stipulate the requirements for safety ensuring devices, their maintenance, ancillary facilities and the formulation of structures assuming responsibility while giving due consideration to achieving both efficient and safe port operation.

[References]

1) Ministry of Land, Infrastructure, Transport and Tourism: "Guidelines for Establishment of Maintenance and Management Plan for Cargo Handling Equipments in Port", July., 2016(in Japanese).

2) Ministry of Land, Infrastructure, Transport and Tourism: "Guidelines for Inspection and Diagnosis of Cargo Handling Equipments in Port", July., 2014(in Japanese).

2 Movable Passenger Boarding Facilities

[Ministerial Ordinance] (Performance Requirements of Movable Passenger Boarding Facilities)

Article 51

The performance requirements of movable passenger boarding facilities shall be as prescribed respectively in the following items for the purpose of the safe and smooth embarkation and disembarkation of passengers in consideration of the structural types of the facilities.

(1) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable passengers to safely and smoothly embark and disembark.

(2) Damage to movable passenger boarding facilities, etc. due to the actions of self-weight, Level 1 earthquake ground motion, surcharge loads and winds, etc. shall not adversely affect the continuous use of movable passenger boarding facilities without impairing the functions of the movable passenger boarding facilities.

[Public Notice] (Performance Criteria of Movable Passenger Boarding Facilities)

Article 93

The performance criteria of movable passenger boarding facilities shall be as prescribed in the following items:

(1) Passages shall satisfy the following requirements to ensure safe and smooth embarkation and disembarkation of passengers:

a) Passages shall have appropriate widths and gradients;

b) Passages shall be provided with anti-slip measures or made of non-skid materials;

c) Passages shall have side walls and handrails on both sides.

(2) Passages shall not be provided with staircases. However, when staircases need to be installed, the heights of rises shall be installed appropriately for ensuring the safety of users and stair landings shall be installed appropriately as necessary.

(3) Passenger boarding facilities shall not double as vehicle boarding facilities. Provided, however, that this shall not apply in cases where pedestrian zones are separated from roadways.

(4) Mobile ranges in a vertical direction at the tips of movable bridges on passenger boarding facilities shall be appropriately set in accordance with tide levels, fluctuations in the drafts of ships, and the rolling and pitching of ships.

(5) The risk of losing the soundness of members in a permanent state in which the dominating action is self-weight shall be equal to or less than the threshold level.

(6) The risk of losing the stability of the facilities due to uplifting of the leg sections of the facilities shall be equal to or less than the threshold level in a variable situation, in which the dominating actions are Level 1 earthquake ground motion, surcharge loads and winds.

[Interpretation]

14 Mobile Facilities

(2) The Performance Criteria of Movable Passenger Boarding Facilities (Article 51 of the Ministerial Ordinance and the interpretation related to Article 93 of the Public Notice)

The performance requirements of movable passenger boarding facilities in a permanent situation in which the dominating action is self-weight of facilities, or in a variable situation in which the dominating actions are Level 1 earthquake ground motions, surcharges and winds, shall be serviceability. The performance verification items and standard indexes to determine limit values with respect to the action shall be as shown in **Attached Table 14-1**. In **Attached Table 14-1**, the standard index to determine limit values for the soundness of members shall be appropriately set when carrying out the performance verification of members. Furthermore, in **Attached Table 14-1**, the standard index to determine limit values for the stability shall be appropriately set when carrying out the performance verification of the stability of foundation sections and the uplift of leg sections.

Attached Table 14-1 Performance Verification Items and Standard Indexes to Determine the Limit Values of Movable Passenger Boarding Facilities under the Respective Design Situations (Except in Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
51	–	2	93	–	5	Serviceability	Permanent	Self-weight	Surcharge, earth pressure, water pressure	Soundness of member	–
					6		Variable	Level 1 earthquake ground motion	Self-weight, surcharge, earth pressure, water pressure	Uplift of leg section	–
								(Surcharge)	(Self-weight, earth pressure, water pressure)		
								(Wind)	(Self-weight, surcharge, earth pressure, water pressure)		

*[ ] means alternative dominating actions to be studied as design situations.
*( ) means alternative non-dominating actions to be studied in accordance with the dominating actions.

2.1 General

The related laws and regulations for passenger facilities include **the Act concerning the Promotion of Smooth Transport of Elderly and Physically Disabled Persons Using Public Transportation** (Act No. 68 of 2000).

2.2 Fundamentals of Performance Verification

(1) Movable passenger boarding facilities shall have functions enabling passengers to safely and smoothly embark and disembark ships and shall be separated from vehicle boarding facilities in principle.

(2) Movable passenger boarding facilities shall not cause passengers to feel like they are in danger and shall have stable structures with respect to winds and the rolling and pitching of ships.

(3) Structural Types

① The structural requirements for movable passenger boarding facilities are as follows.

(a) The widths of passages shall be appropriately set so as to be equal to or more than 75 cm, taking into consideration the usage conditions of fixed passenger boarding facilities. Furthermore, it is preferable that the passages have a width of 1.2 m or more in consideration of the convenience of elderly and physically disabled persons.

(b) Passages shall have side walls and handrails on both sides. In addition, passages shall be provided with anti-slip measures on their surfaces or the surfaces shall be finished with non-skid materials.

(c) The heights of the rises of staircases shall be set in consideration of the safety of users and staircases shall be provided with landings as appropriate. Generally, the heights of the rises and the widths of the treads on the staircases can be about 16 cm and 30 cm or more, respectively. It is preferable that staircases with overall heights exceeding 3 m have landings with a width of 1.2 m or more at vertical intervals of 3 m or less.

(d) Movable passenger boarding facilities shall not double as vehicle boarding facilities, provided, however, that this shall not apply in cases where pedestrian zones are separated from roadways.

(e) The gradients of boarding passages shall be appropriately set in consideration of the safety of users. In the case of slip ways, their gradients are generally set at 12% or less; however, the gradients are preferably 5 to 8% or less in consideration of the convenience of elderly and physically disabled persons.

② The mobile ranges in a vertical direction at the tips of movable bridges on movable passenger boarding facilities shall be appropriately set in accordance with tide levels, fluctuations in the drafts of ships, and the rolling and pitching of ships. Such mobile ranges may be set at the values obtained by adding 1 m to the difference between mean monthly-highest water levels and mean monthly-lowest water levels.

③ Those movable passenger boarding facilities for use in public transportation shall give due consideration to the safe use of persons in wheelchairs, taking into consideration the convenience of elderly and physically disabled persons. In this situation, refer to **the Guidelines of Facilities for Elderly and Handicapped People in Public Transport Terminals**.[1)]

2.3 Performance Verification

(1) For the performance verification of movable passenger boarding facilities, refer to the Specifications and Commentary for Highway Bridges[2)] and the Technical Standards and Commentary of Grade Separation Facilities for Pedestrians.[3)]

(2) Because movable passenger boarding facilities are used in corrosive environments, they shall be provided with corrosion control so as not to damage their durability.

2.4 Ancillary Facilities

(1) Movable passenger boarding facilities shall be provided with the necessary ancillary facilities in consideration of passenger safety.

(2) Handrails have functions not only for preventing falls but also for providing smoother passenger traffic by alleviating passenger concerns about safety. Although the height of handrails can be set at 1.1 m or higher to prevent adults of average size from climbing over them, such a height might not enable handrails to effectively prevent users such as infants, children and persons in wheelchairs from falling. Thus, handrails shall have structures comprising columns, crosspieces and metal meshes as fall prevention measures to ensure users a sense of safety.

(3) Movable passenger boarding facilities shall be provided with fences, ropes or chains at entrances to safely guide passengers to the facilities. The height of fences and others necessary measures to ensure passenger safety may be set at 70 cm. When using ropes or chains, they shall be tightened firmly without sagging beyond what is necessary.

(4) When covers are installed over movable passenger boarding facilities, the height of the covers can be set at 2.1 m or higher.

(5) Movable passenger boarding facilities shall be provided with emergency exits when their lengths exceed 60 m. The intervals between entrances and emergency exits or between emergency exists shall be 60 m or less. In addition, guide and indication signs for emergency exits shall be installed along passages.

[References]

1) Transport Ecology and mobility Foundation: Guideline of the Facilities for Elderly and Handicapped People in Public Transport Terminals, 2001

2) Japan Road Association: Specifications and Commentary for Highway Bridges, Maruzen Publications, 2002

3) Japan Road Association: Technical Standards and Commentary of Grade Separation Facilities for Pedestrians, 1979

Chapter 11 Other Port Facilities

[Ministerial Ordinance] (Items Necessary for Other Port Facilities)

Article 57

The items necessary for the performance requirements for fixed and mobile facilities for passenger boarding, waste disposal sites, beaches, and plazas and green spaces as specified in this Chapter by the Minister of Land, Infrastructure, Transport and Tourism and other requirements shall be provided by Public Notice.

[Public Notice] (Other Port Facilities)

Article 94

The items to be specified by Public Notice under Article 57 of the Ministerial Ordinance concerning the performance requirements for fixed facilities for passenger boarding, waste disposal sites, beaches, and plazas and green spaces shall be as provided in the following Article through Article 98.

1 Fixed Passenger Boarding Facilities

[Ministerial Ordinance] (Performance Requirements for Fixed Passenger Boarding Facilities)

Article 53

The provisions of Article 51 apply mutatis mutandis to the performance requirements for fixed passenger boarding facilities.

[Public Notice] (Performance Requirements for Fixed Passenger Boarding Facilities)

Article 95

(1) The provisions of Article 93 (excluding Item (vi)) apply mutatis mutandis to the performance requirements for fixed passenger boarding facilities depending on the types of the facilities.

(2) In addition to the provision of the preceding paragraph, the performance requirements for fixed passenger boarding facilities shall be such that the risks of losing the integrity of members and stability of foundation sections under variable situations, in which the dominating actions are level 1 earthquake ground motions, surcharge loads or winds, shall be equal to or less than the threshold level.

[Interpretation]

15 Other Port Facilities

(1) The performance criteria of fixed passenger boarding facilities (Article 53 of the Ministerial Ordinance and the interpretation related to Article 95 of the Public Notice)

The performance requirements for fixed passenger boarding facilities under a permanent state in which the dominating action is self-weight, or under variable situations in which the dominating actions are level 1 earthquake ground motions, surcharges or winds shall be serviceability. The performance verification items and standard indexes to determine the limit values with respect to the actions shall be as shown in **Attached Table 15-1**. In **Attached Table 15-1**, the standard index to determine the limit values for the soundness of members and the stability of foundation sections shall be appropriately set when carrying out the performance verification of the members and the foundation sections, respectively.

Attached Table 15-1 Performance Verification Items and Standard Indexes to Determine Limit Values of Fixed Passenger Boarding Facilities under Their Respective Design States (Except in Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
53	–	–	95	–	–	Serviceability	Permanent	Self-weight	Surcharge, earth pressure, water pressure	Soundness of member	–
							Variable	Level 1 earthquake ground motion [Surcharge] [Wind]	Self-weight, surcharge, earth pressure, water pressure (Self-weight, earth pressure, water pressure) (Self-weight, surcharge, earth pressure, water pressure)	Soundness of member Stability of foundation	–

*[ ] means an alternative dominating action to be assessed as a design state.
*( ) means an alternative non-dominating action to be assessed in accordance with the dominant actions.

The performance verification of fixed passenger boarding facilities can be carried out with reference to **Part III, Chapter 10, 2 Mobile Facilities for Passenger Boarding**.

2 Waste Disposal Sites

[Ministerial Ordinance] (Performance Requirements for Waste Disposal Sites)

Article 54

1 The performance requirements for waste disposal sites shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied so as to appropriately dispose of waste materials and protect disposal sites.

2 The provisions of Article 16 apply mutatis mutandis to the performance requirements for waste disposal sites.

[Public Notice] (Performance Criteria of Waste Disposal Sites)

Article 96

1 The provisions of Article 39 apply mutatis mutandis to the performance criteria of waste disposal sites.

2 In addition to the provision of the preceding paragraph, the performance criteria of waste disposal sites shall be such that the sites are appropriately arranged and have the dimensions necessary to prevent washing out of the waste materials by waves, storm surges, design tsunamis, etc. in consideration of the environmental conditions to which the facilities are subjected.

[Interpretation]

15 Other Port Facilities

(2) Performance criteria of waste disposal sites (Article 54 of the Ministerial Ordinance and the interpretation of Article 96 of the Public Notice)

The required performance criteria and interpretation for seawalls shall be applied to those for waste disposal sites.

2.1 General

(1) The definitions of waste differ depending on whether the reference is made to the **Waste Disposal and Public Cleansing Act** (Act No. 137 of 1970, hereinafter referred to as the “Waste Disposal Act”) or the **Act for the Prevention of Marine Pollution and Maritime Disasters** (Act No. 136 of 1970, hereinafter referred to as the “Marine Pollution Prevention Act”). The main difference in the definitions between the two acts is that earth and sand are not reflected in the Waste Disposal Act, but they are in the Marine Pollution Prevention Act. In this code, waste subject to the Waste Disposal Act, as well as earth and sand subject to the Marine Pollution Prevention Act, is collectively defined as waste. In the Marine Pollution Prevention Act, the definition of waste does not include the type of earth and sand which is sufficiently managed by contractors and has a quality considered acceptable as reclamation materials (refer to the Circular Notice of the Transport Minister’s Secretariat, No. 289 of 1972 “Concerning the implementation of the Marine Pollution Prevention Act”). The Marine Pollution Prevention Act is applicable to the waste discharged from ships to the sea.

(2) In the Waste Disposal Act, waste is classified into: general waste; the industrial waste specified in (1) to (6), (a), Item 3, Article 6 of the **Enforcement Ordinance of the Waste Disposal Act** (hereinafter referred to as “stable-type industrial waste”); the industrial waste specified by (c), Item 14, Article 7 of the **Enforcement Ordinance of the Waste Disposal Act** (hereinafter referred to as “controlled-type industrial waste”); and the industrial waste specified in (1) to (5), (c), Item 3, Article 6 and (1) to (6), (a), Item 3, Article 6-5 of the **Enforcement Ordinance of the Waste Disposal Act** (hereinafter referred to as “shielding-type industrial waste”).

(3) In the Marine Pollution Prevention Act, dredged soil is classified into: hazardous dredged soil specified in Items 4 and 5, Paragraph 2, Article 5 of the **Enforcement Ordinance of the Marine Pollution Prevention Act**; and special, designated and other dredged soil (hereinafter referred to as “general dredged soil”) specified in Item 1, Paragraph 1, Article 5 of the **Enforcement Ordinance of the Marine Pollution Prevention Act**.

(4) Depending on the types of waste subject to landfill disposal, waste disposal sites are classified into: stabilized waste disposal sites for the disposal of stabilized waste and general dredged soil; controlled-type waste disposal sites for

the disposal of general waste and controlled-type waste; and strictly controlled-type waste disposal sites for the disposal of strictly controlled-type waste.

(5) Waste disposal sites are constructed not only for the landfill disposal of waste, but also for the land use after the disposal operation. Therefore, the areas and layouts of landfill sites and landfill methods shall be determined based on the demands for land use and constraints in cases where waste disposal is subjected to the Waste Disposal Act.

(6) After the abolition of final disposal sites, controlled-type waste disposal sites are maintained as port facilities. However, waste disposal sites still need to maintain their water impermeability, and there are cases where groundwater levels in landfilled areas need to be controlled. Thus, the facility layouts of final disposal sites shall be assessed while taking into consideration not only the land use, but also the maintenance after the abolition of the final disposal sites. In addition, it is desirable to assess the facility layouts in close collaboration with the environmental sections in charge of waste disposal.

(7) There has been no practical example of waste disposal sites constructed in areas other than sea areas and it is expected that almost all waste disposal sites will be constructed in sea areas in the future. Thus, the waste disposal sites discussed in this code shall be those to be constructed in sea areas. Therefore, the characteristics of waste disposal sites are such that ① they shall be constructed with due consideration to the actions of waves and earthquake ground motions, and ② revetments and seepage control work shall be stabilized with properly controlled levels of retained water in the case of controlled-type waste disposal sites.

2.2 Purposes and Types of Waste Disposal Sites

(1) The purposes of constructing waste disposal sites are to develop sea area waste disposal sites and to protect the sites and hinterlands from storm surges, tsunamis and waves. These purposes shall be valid even after landfilled areas are used for other land use.

(2) The environmental safety and compatibility of waste disposal sites shall be achieved not only by their performance as revetments, but also by how adequately they manage waste for disposal. Thus, it is necessary to assess how to manage waste for disposal in close collaboration with the authorities who manage the waste.

2.2.1 Stabilized Waste Disposal Sites

(1) Stabilized waste disposal sites are facilities that enable the disposal of stabilized industrial waste and general dredged soil. They do not require construction permits for final disposal sites based on the Waste Disposal Act.

(2) Stabilized waste disposal sites shall have the function of preventing waste in landfill sites from being washed out.

(3) Waste which is subject to the Waste Disposal Act and which is to be disposed of at stabilized waste disposal sites shall conform to the landfill disposal standards specified in the Waste Disposal Act.

2.2.2 Controlled-Type Waste Disposal Sites

(1) Controlled-type waste disposal sites are facilities that enable the disposal of general waste (excluding specially-controlled general waste) and controlled-type industrial waste. Controlled-type waste disposal sites for waste under the Waste Disposal Act are subject to construction permits for final disposal sites based on the Waste Disposal Act and the technical standards in the Waste Disposal Act (the **Ministerial Ordinance Prescribing the Technical Standards concerning the Final Landfill Sites of General Waste and Industrial Waste** (Ordinance of the Prime Minister's Office and the Ministry of Health and Welfare No. 1 of 1977), hereinafter referred to as the "Ministerial Ordinance for Final Disposal") until confirmation of the abolition of the final landfill sites based on the Waste Disposal Act. In addition, controlled-type waste disposal sites shall be managed as designated areas under the Waste Disposal Act even after the confirmation of the closure of the landfill sites (refer to (5)). The laws and regulations applicable to controlled-type waste disposal sites corresponding to each stage of life are summarized in **Fig. 2.2.1**.

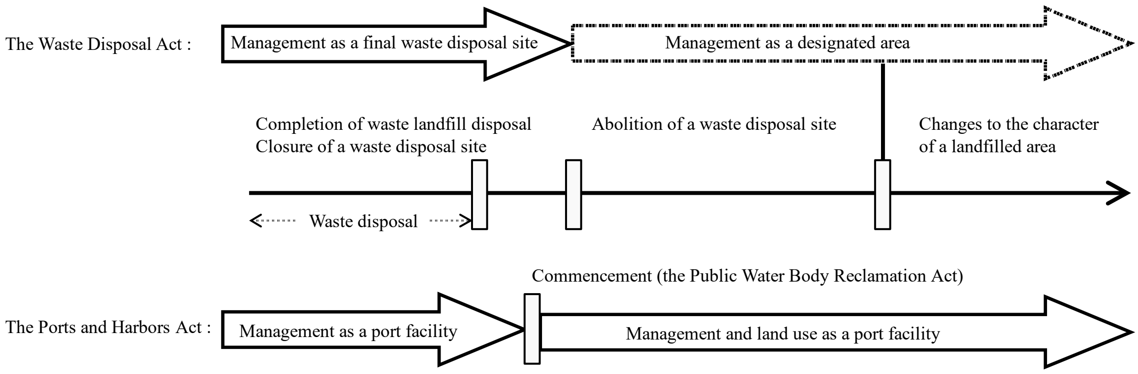

Fig. 2.2.1 The Laws and Regulations Applicable to Controlled-Type Waste Disposal Sites for Waste Subject to the Waste Disposal and Public Cleansing Act (Modification of the Literature[2)])

(2) Controlled-type waste disposal sites shall have the required water impermeability to keep retained water in waste landfill sites from leaking out, in addition to preventing waste in landfill sites from being washed out. It is preferable that water impermeability be ensured through the introduction of a fail-safe concept. Here, the fail-safe concept means the introduction of at least one of the following measures: double seepage control structures where water impermeability can be maintained by either one of the structures when the other is damaged; water level control to keep hydraulic gradients in a direction opposite to the directions of external leakages; confirmation of water impermeability through inspections and monitoring; and ensuring prompt restorability of damage.

(3) Waste which is subject to the Waste Disposal Act and which is to be disposed of at controlled-type waste disposal sites shall conform to the landfill disposal standards prescribed in the Waste Disposal Act depending on the types of waste.

(4) After the landfill of waste is completed under the Waste Disposal Act at controlled-type waste disposal sites, notifications shall be made accordingly and landfill sites shall be closed in accordance with the Ministerial Ordinance for Final Disposal. Then, the final disposal sites shall obtain approval for abolition based on confirmation that the sites conform to the abolition standards prescribed in the Ministerial Ordinance for Final Disposal.

(5) The landfilled areas of the final disposal sites which are abolished based on (4) above are normally registered as designated areas subjected to Article 15-17 of the Waste Disposal Act. Then, the preliminarily approval of prefectural governors shall be required for any possible changes to the character of the landfilled areas along with the construction or improvement of port facilities. As reference, the following literature summarizes the notification items and procedures in cases where notifications are not required and the specific contents of the enforcement standards: the **Enforcement Guidelines concerning Changes in the Character of Landfilled Areas of Final Disposal Sites**[3)] (Notice of Waste Management Division, Waste Management and Recycling Department, Ministry's Secretariat, Ministry of the Environment No. 050606001, Industrial Waste Management Division, Waste Management and Recycling Department, Ministry's Secretariat, Ministry of the Environment No. 050606001 of June 6, 2005). In particular, when constructing pile foundations penetrating the bottom impervious layers for the intensive land use of landfilled areas, refer to the **Guidelines for the Intensive Land Use of Controlled-Type Sea Area Final Disposal Sites in Ports and Harbors (Draft): For the Construction of Foundation Piles Penetrating Bottom Impervious Layers**.[4)]

(6) When utilizing landfilled areas before getting approval for the abolition of final waste disposal sites, it is necessary to follow the maintenance standards in the Ministerial Ordinance for Final Disposal and to refer to the literatures introduced in (5) above.

2.2.3 Strictly Controlled-Type Waste Disposal Sites

(1) Strictly controlled-type waste disposal sites are facilities that primarily enable the disposal of strictly controlled-type industrial waste. For the disposal of waste subject to the Waste Disposal Act, strictly controlled-type waste disposal sites are subject to construction permits for final disposal sites based on the Waste Disposal Act and the Ministerial Ordinance for Final Disposal.

(2) Because the waste to be disposed of contains harmful substances, strictly controlled-type waste disposal sites need to have structures that can completely isolate landfill sites from the outside. However, this code refrains from any

further comments on strictly controlled-type waste disposal sites as there have been no cases of the construction of such sites in port and harbor areas.

2.3 Fundamentals of Performance Verification

(1) The performance verification of revetments of waste disposal sites can be carried out with reference to **Part III, Chapter 4, 14.6 Performance Verification**.

(2) Unlike the revetments of protective facilities for harbors, the purpose of revetments of waste disposal sites is to provide the site to accept waste; such revetments usually have a long landfilling period and often remain in a structurally unstable state for a long period of time before the areas behind revetments are landfilled. Therefore, it is necessary to pay particular attention to ensuring structural safety during construction. One of the most effective approaches is to give priority to waste dumping behind the revetments so that sufficient structural stability is achieved early by balancing the actions in the front and back of the revetments.

(3) It is necessary to give due consideration to waste disposal methods so as to reduce earth pressure acting on the back side of revetments of waste disposal sites and thereby achieving cross-sectional stability of the revetments of waste disposal sites after the completion of waste landfill operations.

(4) There may be cases where the future land use of landfilled areas requires waste disposal sites to ensure certain soil bearing capacities. In such cases, it is necessary to give due consideration to the types of waste to be accepted for landfill and the acceptance methods.

(5) Revetments of waste disposal sites shall have structures which can prevent waste from being washed out into sea areas, not only under variable and permanent action situations, but also against the actions of level 2 earthquake ground motions; provided, however, that this provision shall not apply to stabilized waste disposal sites for the disposal of general dredged soil.

2.4 Performance Verification

(1) The performance verification of revetments of controlled-type waste disposal sites can be carried out as follows the case for revetments with reference to **Part III, Chapter 4, 14.6 Performance Verification**, or by the following procedures.

(2) Rapid implementation of landfill disposal near revetments of controlled-type waste disposal sites may cause cohesive soil ground to undergo circular slip failures, thereby impairing the functions of revetments of the controlled-type waste disposal sites. Therefore, due consideration shall be given to the setting of areas and rates of landfill disposal when relying on the increase in foundation strength due to consolidated drainage to achieve structural stability.

(3) The following are the requirements for seepage control work applied to controlled-type waste disposal sites as specified in the Ministerial Ordinance for Final Disposal.

① **For cases where no seepage control work is required (Item 5(a), Paragraph 1, Article 1 of the Ministerial Ordinance for Final Disposal)**

Seepage control work is not required in cases where there exists a continuous layer (impervious layer) with a thickness of 5 m or more and a permeability factor of $k = 1 \times 10^{-7}$ m/s (1×10^{-5} cm/s) or less at the bottom and sides of the landfill area.

② **For cases where no impervious layer exists on the entire underground surface of a landfill area (Item 5(a)(1), Paragraph 1, Article 1 of the Ministerial Ordinance for Final Disposal)**

The Ministerial Ordinance for Final Disposal stipulates that seepage control work in cases where no impervious layer exists shall fulfill the following requirements (called surface seepage control work), or shall have seepage control effects equivalent to or greater than the surface seepage control work. In addition, the Ministerial Ordinance for Final Disposal sets a provision on the protection of surface seepage control work with light-blocking, nonwoven cloth in the case of possible degradation of water sealing sheets due to exposure to sunlight.

(a) Seepage control work with water sealing sheets laid on cohesive soil layers having a thickness of 50 cm or more and a permeability factor of $k = 1 \times 10^{-8}$ m/s (1×10^{-6} cm/s) or less

(b) Seepage control work with water sealing sheets laid on the surfaces of watertight asphalt concrete having a thickness of 5 cm or more and a permeability factor of $k = 1 \times 10^{-9}$ m/s (1×10^{-7} cm/s) or less

(c) Seepage control work with double water sealing sheets having intermediary protection layers made of nonwoven cloth or synthetic resin in between and laid on the surfaces of nonwoven cloth.

③ For cases where an impervious layer exists on the entire underground surface of a landfill area (Item 5(b), Paragraph 1, Article 1 of the Ministerial Ordinance for Final Disposal)

The Ministerial Ordinance for Final Disposal stipulates that seepage control work in cases where an impervious layer exists shall fulfill the following requirements or shall have seepage control effects equivalent to or greater than the seepage control work. In addition, the seepage control work shall be constructed so as to reach the impervious layer.

(a) Seepage control work constructed in a manner that injects chemicals into the ground down to an impervious layer so that the ground has a Lugeon value of 1 or less when the chemicals solidify.

(b) Seepage control work with a continuous wall having a thickness of 50 cm or more and a permeability factor of $k = 1 \times 10^{-8}$ m/s (1×10^{-6} cm/s) or less installed to an impervious layer.

(c) Seepage control work with steel sheet piles (only those provided with impermeable joints) driven to an impervious layer.

(d) Seepage control work satisfying the requirements in (a) to (c) of ② above.

(4) For inland waste disposal sites, it is often the case that water sealing sheets are used to ensure sufficient seepage control performance at their bottom layers. However, for waste disposal sites located in coastal areas in Japan, it is often the case that cohesive soil below the bottom layers is used to ensure sufficient seepage control work as stated in (3). Therefore, it is necessary to confirm whether cohesive soil layers equivalent to the impervious layers exist adjacent to the bottoms of the disposal sites located in water areas and to confirm whether the cohesive soil layers have seepage control capabilities equivalent to those of the cohesive soil layers specified in the Ministerial Ordinance for Final Disposal.

Whether the seepage control capabilities of specific layers are equivalent to those of cohesive soil layers can be evaluated by their permeation time, which can usually be calculated by the **equation** (**2.4.1**).

$$t = \frac{L^2}{kh} \tag{2.4.1}$$

where

t : the permeation time (s)

L : the permeation distance (thickness of a layer) (m)

k : the permeability factor (m/s)

h : the water level difference in a layer (as shown in **Fig. 2.4.2**) (m)

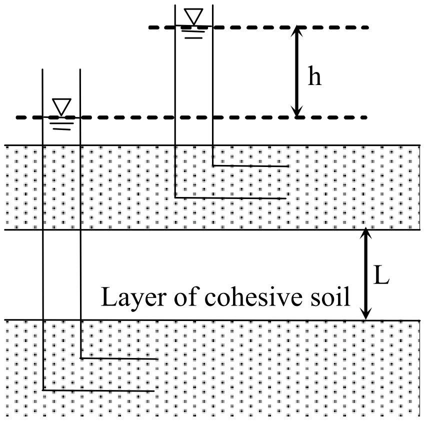

Fig. 2.4.2 Permeation Distance and Water Levels

When calculating a thickness (permeation distance) to achieve the permeation time equivalent to that of an impervious layer (with a thickness of 5 m or more and a permeability factor of $k = 1 \times 10^{-7}$ m/s (1×10^{-5} cm/s) or less) using the **equation (2.4.1)**, a cohesive soil layer with a permeability factor of $k = 1 \times 10^{-8}$ m/s (1×10^{-6} cm/s) is required to have a calculated thickness of 1.6 m or more. The layer thicknesses and continuity of impervious layers shall be confirmed through boring surveys. Layer thicknesses are preferably determined in consideration of allowances for possible inhomogeneity or uneven layer boundaries in cohesive soil layers and possible reductions in layer thicknesses due to deformation or consolidation settlement along with landfill disposal.

(5) Because controlled-type waste disposal sites are provided with seepage control work as specified in the Ministerial Ordinance for Final Disposal and face water areas at their outer sides, they cause water-level differences between the water areas at their outer sides and the retained water at their inner sides. Furthermore, tidal actions at the outer sides cause water-level differences to fluctuate. In contrast, rainwater accumulated in disposal sites at the inner sides needs to be treated (purified) and drained. In order to prevent retained water from leaking out from water area disposal sites, it is preferable to keep the water-levels of retained water lower at the inner sides. However, in the case of seepage control work using water sealing sheets, keeping high water-levels of retained water at the inner sides is considered to be effective in stabilizing the sheets (curbing the lift force on the sheets) until the stability of the seepage control work is improved with the progress of landfill disposal behind the revetments. Thus, it is preferable to appropriately set control water-levels with due consideration to the contradictory conditions. Furthermore, in the case of temporary increases in the water-levels of retained water as a result of abnormal rainfalls or overtopping waves due to storm surges and high waves, it is preferable to ensure the stability of controlled-type waste disposal sites by appropriately setting water-levels in abnormal situations. For the contents of the maintenance of retained water during the time until the abolition of the final disposal sites, refer to the **Collection of the Technical Information on the Abolition of Water Area Final Disposal Sites**.[5)]

(6) The design working life of controlled-type waste disposal sites shall be determined through comprehensive evaluations of the number of years until the retained water inside the sites has reached the abolition standards specified in the Ministerial Ordinance for Final Disposal (Paragraph 3 of Article 1, or Item 3, Paragraph 3 of Article 2), as well as the importance of the facilities, surrounding environments, economic efficiency and land use plans.

(7) The land developed by landfill disposal at controlled-type waste disposal sites may require a long period of time until the landfilled waste has been stabilized to a level satisfying the abolition standards. In addition, controlled-type waste disposal sites may require special considerations in design and construction while depending on the technologies to be used for facilitating early use of the land. In such cases, refer to the **Collection of the Technical Information on the Early Stabilization of Controlled-Type Water Area Final Disposal Sites in Ports and Harbors**.[6)]

(8) Controlled-type waste disposal sites maintain seepage control performance even after the abolition of the waste disposal sites, and, therefore, there may be cases where the controlled-type waste disposal sites are destabilized with the levels of retained water exceeding the control water-levels due to rainfall. Thus, the performance verification of controlled-type waste disposal sites shall include the confirmation of their structural safety with retained water kept at the maximum possible levels after the abolition of the waste disposal sites.

(9) For the performance verification and construction of controlled-type waste disposal sites, refer to the **Manual for the Design, Construction and Maintenance of Controlled-Type Waste Disposal Sites**.[1)]

(10) In a tsunami-resistant design for assessing the stability of the waste disposal sites designated as facilities prepared for accidental incidents with respect to design tsunamis and tsunamis with intensities exceeding the design tsunamis, refer to the **Guideline for Tsunami-Resistant Design of Breakwaters**[7)] and the **Guideline for Tsunami-Resistant Design of Seawalls (Parapet Walls) in Ports**.[8)]

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3 Beaches

[Ministerial Ordinance] (Performance Requirements for Beaches)

Article 55

1 The performance requirements for beaches shall be as prescribed respectively in the following items to facilitate the development of port and harbor environments:

(1) Beaches shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to contribute to the development of port and harbor environments.

(2) Beaches shall be capable of maintaining a stable state over a long term against variable waves and water flows.

2 In addition to the provisions of the preceding paragraph, the beaches which are utilized by an unspecified large number of people shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to ensure the safety of beach users.

[Public Notice] (Performance Criteria of Beaches)

Article 97

1 The performance criteria of beaches shall be as prescribed respectively in the following items:

(1) Beaches shall be appropriately located with the necessary dimensions to ensure the safe and comfortable use by visitors and to contribute to the enhancement of good port environments.

(2) The risk of losing the stability of the beach profile and plan shape shall be equal to or less than the threshold level in a variable situation in which the dominating actions are variable waves and water flows.

2 In addition to the provisions in the preceding paragraph, beaches which are utilized by an unspecified large number of people shall be provided with the dimensions required for securing the safety of the users by taking into consideration the environmental conditions to which the facilities are subjected, and the usage conditions.

[Interpretation]

15. Other Port Facilities

(3) The performance criteria of beaches (Article 55 of the Ministerial Ordinance and the interpretation related to Article 97 of the Public Notice)

Serviciability shall be the performance requirement of beaches under the external forces including waves and water flows. The performance verification items and standard indexes to determine limit values with respect to the actions shall be as shown in **Attached Table 15-2**. In **Attached Table 15-2**, the standard index to determine the limit values for the stability of beaches shall be appropriately set when carrying out their performance verification.

Attached Table 15-2 Performance Verification Items and Standard Index to Determine Limit Values of Beaches under the Respective Design States

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
55	1	2	97	1	2	Serviceability	Variable	Variable waves, water flows	–	Stability of beach shapes	–

3.1 General

(1) Beaches are classified depending on the grain size compositions of sediment: those composed of mud, sand or gravel, and those with exposed bedrock or rocky coasts. Depending on the positional relationships with the tidal zones and beach profiles, beaches are also classified into backshores and foreshores. Furthermore, depending on the ecosystems, such as vegetation, beaches are classified into seagrass meadows and coral reefs. The typical cross section of a beach is shown in **Fig. 3.1.1**.

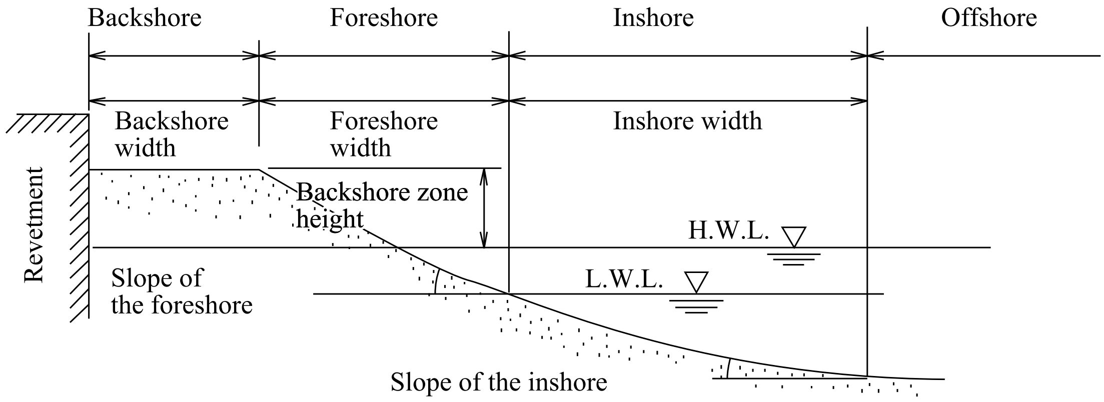

Fig. 3.1.1 Example of the Cross Section of a Typical Beach

(2) Tidal flats are beaches which have flat topography with sandy mud sediment exposed at low tide[1)] and often form complex and valuable natural environments as a result of a mix of various factors such as the repetition of exposure and submergence of bottom sediment, fluctuations in saline concentrations due to river inflow, and geographical deformation due to waves and currents. Other beaches, called shallow waters, have geography similar to tidal flats, except for the depths which range approximately 10 m without the exposure of bottom sediment. Seagrass meadows are shallow sea areas in which large sea algae and seagrass grow densely to form colonies. These colonies are found in water areas with depths ranging from tens of centimeters to tens of meters. Coral reefs are topographic features that are formed by hermatypic organisms such as coral.

(3) Beach nourishment is defined as artificially supplying sand along the shores to develop or restore the beaches. Artificially nourished beaches shall be developed with grain sizes and slopes which are appropriately set. In the cases of artificially nourished beaches where continuous nourishment cannot be expected, jetties and detached breakwaters are preferably installed to stabilize the shapes of the beaches.

(4) Generally, beaches include not only artificially nourished beaches, but also those that are naturally developed. Here, beaches refer to artificially nourished beaches and natural beaches with artificial maintenance or restoration.

3.2 Purposes of Beaches

(1) As shore protection facilities, sand beaches are developed for the purposes of protecting shores from damage due to tsunamis, storm surges, waves and other fluctuations of seawater or ground, and promoting the development and preservation of shore environments as well as the proper public use of shores, thereby contributing to the conservation of national land. The main purpose of such beaches is shore protection. In contrast, beaches are also developed as port environment development facilities for the purpose of developing comfortable living spaces with waterfront amenities. Thus, the main purposes of such beaches include not only shore protection, but also safe and comfortable use by public visitors and the preservation of natural environments.

(2) Beaches provide visitors with waterfront amenities in the form of recreation spaces for collecting shellfish in addition to swimming and fishing, sports spaces for beach volleyball, etc., work spaces for agriculture and fisheries, and natural experiences as well as educational spaces.

(3) The functions of beaches to preserve natural environments include the creation of habitats for a wide variety of organisms by creating favorable environments for them to live and grow, the facilitation of water quality purification through the physical and biological actions of beaches, and the production of marine species supported by the primary production of organic substances.

(4) In addition to providing visitors with waterfront amenities, beaches have functions to alleviate the flow rates of overtopping waves by damping the energy of incident waves through wave breaking and preventing the scouring of dike toes.

3.3 Fundamentals of Performance Verification

(1) Similar to beaches as port environment development facilities, sand beaches as shore protection facilities are defined in **Paragraph 1, Article 2 of the Coast Act**. These two types of beaches are facilities on shorelines and have similar shapes. They also have many features in common with respect to stability against waves. Thus, the performance verification of beaches can be carried out with reference to the **Technical Standards and Commentaries for Shore Protection Facilities**[2)] and the **Planning and Design Manual for Zonal Shore Protective Complexes**.[3)]

(2) Each beach has one or more functions for waterfront amenities, create habitats for living organisms, purify water quality, and produce marine species. Because some of these functions are complementary to each other and others are contradictory, the examination of beach development shall start by setting the appropriate objectives. When setting objectives, it is important to understand the past relationships between the natural environments and the lives of local people. Such an understanding is useful for consultations with the parties concerned and for examining and deciding plans based on sharing concepts concerning nature. In addition, it requires particular attention so that these functions are affected by the stability and maturity of the ecosystems and environmental fluctuations.

3.4 Topography of Beaches

(1) With regard to long-term topographical changes at beaches, it can be said that constructed beaches are stable in the long term if the beach profiles immediately after their nourishment are stable with respect to the dominant waves. Although short-term topographical changes are affected mainly by the phenomenon of littoral drift in cross-shore directions, this phenomenon has remained unexplained. Therefore, it is necessary to examine the appropriate measures, such as an examination of stabilization measures by means of jetties and detached breakwaters, the selection of grain sizes of sand in accordance with the characteristics of waves, and the replenishment for eroded sand. The shapes of the initial shorelines shall be close to those of stabilized beaches as a result of the actions of waves in relation to the arrangements of jetties and detached breakwaters.

(2) In addition to the topography, the materials to be used for beach nourishment are important factors that affect beach performance and stability and must be selected carefully. When examining the development of beaches involving beach nourishment, due consideration shall be given to the fact that the grain size compositions of the sand used for beach nourishment affect not only the stability and cross-sectional profiles of the beaches, but also the satisfaction of beach users and the habitats of the organisms living there. Furthermore, due consideration shall be given to the selection of materials for beach nourishment so as to prevent the sediments from being washed out and adversely affecting the surrounding water areas.

(3) When developing plans to install stone structures in tidal flats or rocky shores, it is necessary to give sufficient consideration to placing them appropriately to ensure the safety of users and the stability of the structures.

(4) Jetties and detached breakwaters are preferably positioned in a manner that ensures the stability of the shapes of the beaches and allows for sufficient tidal exchanges to prevent the deterioration of water quality. When utilizing beaches for swimming, they shall be provided with jetties or detached breakwaters to prevent generation of strong and complex currents, such as rip currents, which cause incidents for swimmers. In addition, when installing jetties and detached breakwaters, it is necessary to pay attention to the following items to prevent these structures from degrading the landscape.

① Expanding the crown widths of the detached breakwaters to reduce their crown heights or submerging them and thereby preventing them from blocking views of the ocean.

② Positioning the detached breakwaters as far from the beach as possible so that beach users do not feel pressured.

③ Using masonry or stone-cladded jetties to create a greater sense of harmony with the surrounding landscape.

④ Providing planting on jetties to enrich ocean views.

Expanding the intervals between the jetties to the extent possible to create a greater sense of openness.

(5) Regarding the topography of beaches, it is necessary to carry out performance verification for the shapes (widths, elevations and lengths) of beaches and grain sizes. The performance verification can be carried out with reference to the **Technical Standards and Commentaries for Shore Protection Facilities**.[2] In addition, the structural details of the beach topography can be set with reference to the following methods.

① **Crown heights and widths of backshores**

The crown height of the backshore shall be determined based on measurements taken at the sites or at similar coasts located near the sites, or with the proposed estimation formulas.[1), 4), 5)] The crown widths of backshores shall be determined taking into consideration the shorelines' amount of short-term regression during high wave periods estimated by using numerical calculations or historical data.

② **Slopes of the foreshore**

The slopes of foreshores, one of the essential dimensions of beaches, shall be determined by the proposed estimation formulas,[1),4), 5)] or based on measurements taken at the sites or at similar coasts located near the sites, taking into consideration the differences in grain sizes and the wave conditions. In general, tidal flats have milder bottom slopes than sandy beaches (refer to **Fig. 3.4.1**).

③ **Sediment grain sizes**

Sediment grain sizes affect not only the stability and cross-shore beach profiles,[1), 4), 5)] but also the degrees of satisfaction among beach users, the distribution of the habitats of organisms, environment purification functions and permeability (water retention characteristics).[1), 5)] Thus, the grain size distribution of the sediment shall be appropriately determined taking into consideration these factors.

(6) In the verification of the topographical stability of beaches, it is necessary to predict the short- and long-term changes of shorelines, or the changes in water depths and the sediment transport rates by using the appropriate numerical calculations or estimation formulas, taking into consideration the wave control and sediment movement control effects of jetties and detached breakwaters.[2), 4)] The methods for predicting the deformation of beaches include empirical engineering methods, hydraulic model experiments and numerical simulations. For details regarding these methods, refer to **Part II, Chapter 2, 7.6 Estimation of Beach Deformation**. Because the degrees of beach deformation are largely affected by local characteristics, it is necessary to comprehensively evaluate as much local information as possible. Thus, it is preferable to predict beach deformation by combining at least two prediction methods.

(7) In addition to littoral drift control with structures such as jetties and detached breakwaters, there are two other methods for maintaining beaches. One is the sand bypass method, which allows sediment accumulated on the upstream side of coastal structures to flow continuously to the downstream side. The other is the sand back pass method, which transfers sediment to eroded areas at the upstream side of coastal structures.

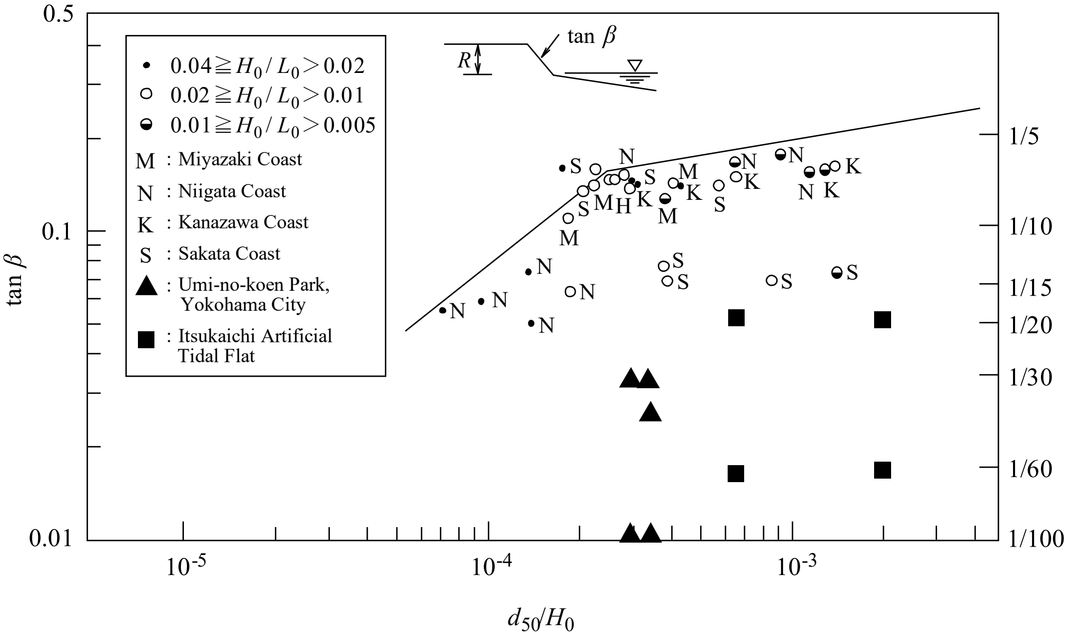

Fig. 3.4.1 Relationship between Seabed Slopes and Sediment Grain Sizes[4)]
(Where: tanβ is a Seabed Slope; d_{50} is the Median Grain Size; and H_0 is the Deepwater Wave Height)

3.5 Waterfront Amenities

(1) The function of beaches to provide visitors with waterfront amenities shall be appropriately evaluated, giving due consideration to the frequency of their use for swimming, shellfish gathering and other purposes.

(2) Beaches are preferably provided with planting and resting areas at the appropriate locations according to their purposes. When examining planting, it is necessary to perform sufficient analyses, taking into consideration the fact that coastal areas are subjected to special environmental conditions such as strong winds, salt water splashes and saline soils. For the determination of planted areas, the selection of tree types and construction and maintenance methods for planting work, refer to the **Design and Construction Manual for the Arrangement of Garden Plants on Port Green Belts**.[6)] Resting areas on beaches, including the necessary component facilities such as benches, trees, shaded areas, public water fountains, hand-wash stations and public restrooms, shall be installed with due consideration to user safety and comfort. In addition, because resting areas are component elements comprising the landscape, it is preferable to arrange them in consideration of their harmonization with the surrounding facilities and vegetation. Furthermore, considering that resting areas do not function individually but collaboratively with the surrounding green zones and facilities, it is preferable to examine the sizes and layouts of the resting areas in consideration of these factors.[7)]

(3) Based on the fact that the main purpose of beaches is human use, it is necessary to give sufficient consideration to ensuring user safety to avoid accidents due to the deformations of beaches. When renovating existing beaches, it is necessary to examine the renovation of facilities based on the collection and organization of materials related to the use conditions of the beaches and conditions that may generate water accidents as well as the extraction of potential problems with the existing facilities. Once a newly constructed or restored beach is opened for public use, it is necessary to conduct periodic patrols and inspections to confirm that the safety measures are functioning properly. In particular, it is important to take measures to prevent sand outflows from nourished beaches, which may cause collapses or create cavities that cannot be recognized from the surface, and to continuously check for and monitor phenomena that may affect user safety by conducting periodic patrols and inspections after the beach is opened for public use.

(4) Beaches provide spaces where people can relax and enjoy recreational activities. However, the safety of beach users is occasionally threatened by tidal waves, storm surges and tsunamis. Therefore, beaches shall be provided with emergency communication equipment, as needed, such as alarm equipment and telephones to allow beach users to evaluate their own safety.

(5) Beaches preferably enable use by visitors such as physically disabled and elderly persons as well as infants who require additionally greater care for their safety than healthy adults. Thus, beaches and adjacent parking lots shall be developed with due consideration to the safe use of all people.

3.6 Conservation of Natural Environment

(1) Beaches have functions to conserve natural environments including those for creating habitats for organisms, purifying seawater and producing marine species. Because some of these functions are complementary to each other while others are contradictory, the examination of beach development shall start with setting the appropriate objectives. Setting objectives based on understanding the relationships between the natural environments and the lives of local people in the past is useful when consulting with the parties concerned and conducting examinations and deciding plans based on sharing concepts about nature.

(2) Depending on the ecosystem, beaches are classified into seagrass beds, tidal flats and coral reefs. For the functions required for each ecosystem and the importance of these types of beaches for conserving natural environments, refer to the following items.[8)]

① **Seagrass beds**

Seagrass beds are shallow coastal sea areas in which large seaweed and seagrass grow densely to form colonies. These colonies are found in water areas with depths ranging from tens of centimeters to tens of meters. Some fish and shellfish use seaweed and seagrass, which make up seagrass beds, as spawning sites. Seaweed and seagrass also have an effect of reducing sea water currents so they contribute to stabilizing sediment and act as nurseries for juvenile fish. Seagrass beds function as feeding grounds for marine species; i.e., abalone and turban shells feed on seaweed and seagrass, and black rockfish and rock trout feed on small sea animals breeding in seagrass beds. In addition, seaweed and seagrass purify seawater in a manner that absorbs nitrogen and phosphorus, which cause eutrophication, as well as act as blue carbon ecosystems in a manner that increase the net carbon dioxide absorption amount with the carbon dioxide concentration in water reduced as a result of photosynthesis of seaweed and seagrass, and also allows organic substances to be stored in bottom sediment.[9)]

The formation of seagrass beds is subjected to not only the characteristics of sea bottoms that will become nursing grounds but also physical factors such as external forces due to waves, biochemical factors such as the balance between photosynthesis rates and oxygen consumption, and biological factors related to herbivorous animals. For example, seaweed beds which are the seagrass beds on sandy mud sea bottoms have constraints from cross-shore directions as shown in **Fig. 3.6.1**.[10)]

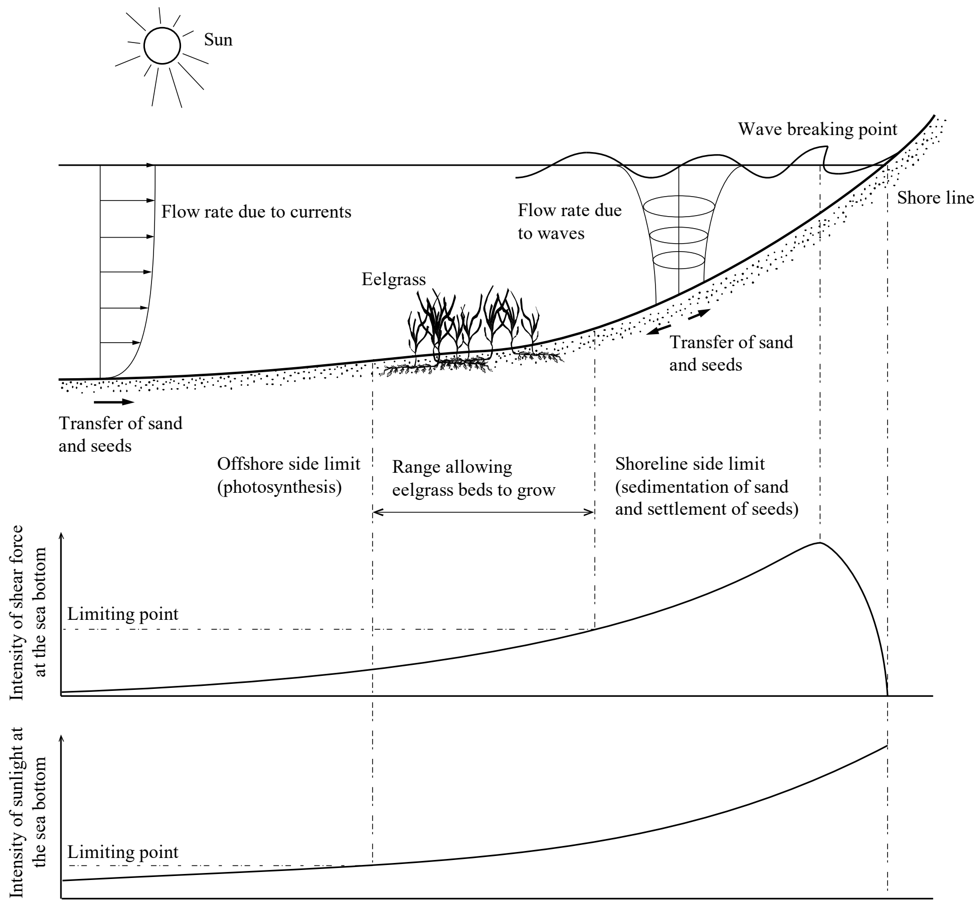

Fig. 3.6.1 Cross-Shore Directional Range of Seaweed Beds That Allows Colonies to Grow

② **Tidal flats**

Tidal flats are water areas with sandy mud sediment, which is repeatedly exposed and submerged along with the tidal fluctuation of ebb and flood, on mild seabed slopes. Natural tidal flats are developed as a result of the balance among the actions of tides, waves and rivers. Because of the environmental diversity produced by the topography and tides, tidal flats allow a wide variety of organisms to make their habitats, including benthic species such as clams and sand worms, as well as benthic algae, aquatic organisms, plankton, fish and birds. Material cycles enhanced by the activities of these organisms enable the tidal flats to serve several functions including purifying the seawater through filtration of the bivalves and producing marine species through the vigorous activities of the primary producers (such as algae and plankton-producing organic substances through photosynthesis). In addition, tidal flats are important for the habitats of rare species such as amphioxus, lingula and limuli, which maintain their primitive forms (species which used to flourish and have widespread habitats but currently survive in only a few locations), and endemic species such as eel gobies and mudskippers (species which make their habitats only in specific areas).

Tidal flats can be formed only when the local topographic conditions and surrounding environments (locational conditions) allow sandy mud to deposit. Flat topography dissipates wave energy, thereby creating calm water areas. The surrounding rivers create brackish environments with a supply of sediments as well as nutrients. Because of these specific topographic and environmental conditions, tidal flats can nurture biodiversity, enhance the productivity of marine species, and maintain smooth material cycle, thereby fulfilling their unique functions. (Refer to **Fig. 3.6.2**.)

When developing tidal flats, it is necessary to examine the foundation to maintain their topography and measures to prepare habitable environments for organisms and enhance the migration and settlement of the organisms. In this regard, refer to the Ecohabitat chart and Optimal design for Biodiversity and Topographical Stability of Tidal Flats.[11a)11b)] Here, the term “topographical stability” includes fluctuations that maintain specific ranges (dynamic stability) and cyclic fluctuations (rhythmic stability).

For foreshore and estuarine tidal flats in which the dominant actions are waves and currents, the widths, backshore crown heights and foreshore slopes are determined in consideration of stability with respect to the waves and currents. In determining these dimensions, the following basic items can be used as references.

(a) The crown heights of backshores shall generally be set at H.W.L or more.

(b) The crown heights of backshores and the slopes of foreshores shall be determined based on actions due mainly to waves with a special focus on maintaining the stability of the topography of the tidal flats by avoiding situations where the foreshores and backshores are subjected to frequent exposure to large waves.

(c) The slopes of outer tidal flats are remarkably mild and, therefore, cannot be determined primarily by waves.

(d) For estuarine tidal flats, it is necessary to examine the stability of bottom sediment with respect to water currents.

(e) For muddy tidal flats, it is necessary to keep them as flat as possible with sufficiently long foreshore sections for the stability of the foundations.

(f) The foreshore sections of foreshore tidal flats may have the topography of flat sections combined with bars and troughs. There may be cases where benthic organisms (shells and sand worms) make their habitats on the sea bottoms below M.W.L.[12)]

In the case of lagoon tidal flats where peripheral environments are of importance, the elevations, slopes and vegetation shall be determined while taking into consideration the exchange of seawater and the maintenance of water quality. In determining these dimensions, the following basic items can be used as references.

(a) The exchange of seawater with the surrounding water areas is required to maintain the water environment including water levels, salinity, nutrients and dissolved oxygen, and the habitats of organisms including the provision of larvae and migration of organisms.

(b) Seawater can be exchanged through channels and training jetties, and water gates can be used to control the volume of seawater to be exchanged. It is preferable to examine the water balance while taking into consideration freshwater inflow, seawater exchange, evaporation, rainfalls, overflow and underground seepage.

(c) Depending on the species, birds show different preferences in terms of water depths and bottom slopes for their feeding grounds. (Snipes and plovers prefer water areas with a depth of approximately 0.5 m or less.)[13)] Because water retention characteristics and the hardness of sediments in tidal flats have a close relationship with the habitat conditions and biological activities of organisms, it is preferable to appropriately evaluate and consider these factors.[14), 15)]

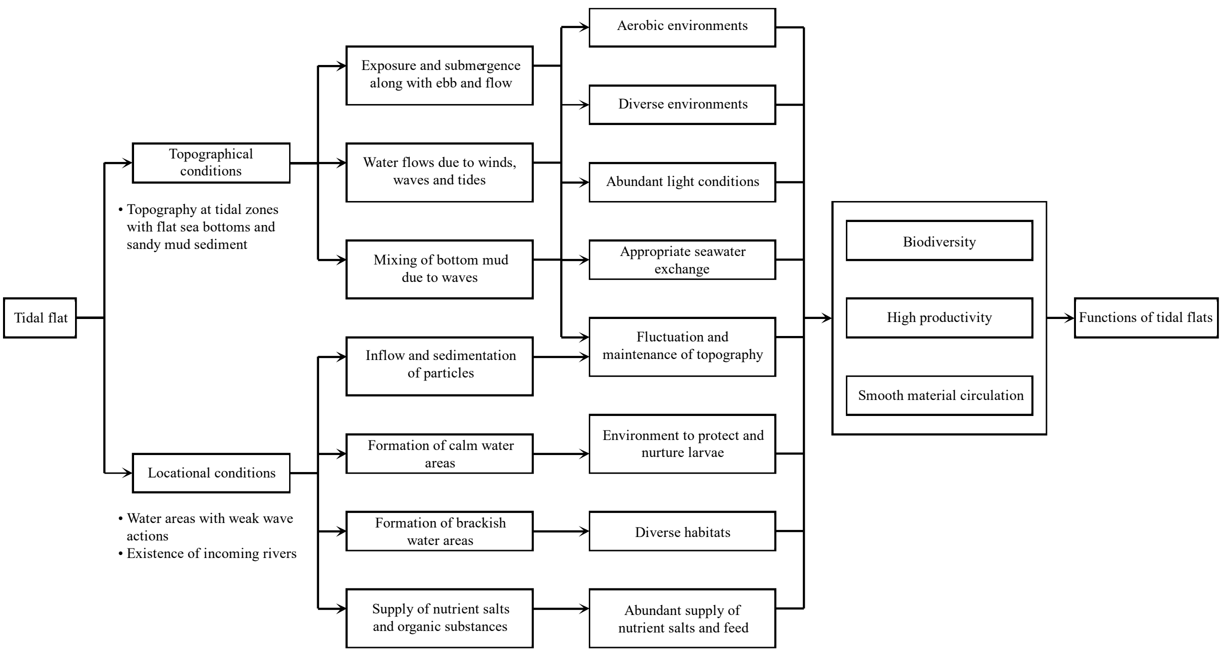

Fig. 3.6.2 Topographical and Environmental Conditions (Locational Conditions) Supporting the Functions of Tidal Flats[1)]

③ **Coral reefs**

Coral reefs are topography formed mainly by cnidarian hermatypic coral. In Okinawa, coral reefs extend across a wide range, from a few hundred meters to a few kilometers from the shoreline. Some coral reefs have the topography of reef lagoons, reef ridges and reef slopes, which include seaweed beds and sand beds (refer to **Fig. 3.6.3**). Live coral communities, which have high organism production owing to the actions of zooxanthella cohabiting in coral, can be found mainly in reef lagoons and reef ridges. Coral reefs generate diverse environments capable of cultivating many marine species, and, therefore, have functions to create habitats for organisms and purify seawater as is the case with tidal flats. Because of their topographical characteristics, some coral reefs contribute to shore protection in the form of living breakwaters which block high waves due to typhoons and beaches with sand generated from foraminiferal shells and the bones of dead coral. Particularly in remote islands surrounded by coral reefs in the southern part of Japan, it is important to preserve the functions of the coral reefs to attenuate waves and generate sand and gravel from coral and foraminiferal shells for the conservation of national land.

The growth of hermatypic coral, which is indispensable for the formation of coral reefs, is subjected not only to the growth conditions of coral itself but also to the growth conditions of symbiotic zooxanthella. The important growth conditions of coral include waves, the widths of reef lagoons, and networks with other coral reefs. The important growth conditions of symbiotic zooxanthella include light, turbidity, water temperature and water depths. (Refer to **Fig. 3.6.3**.)

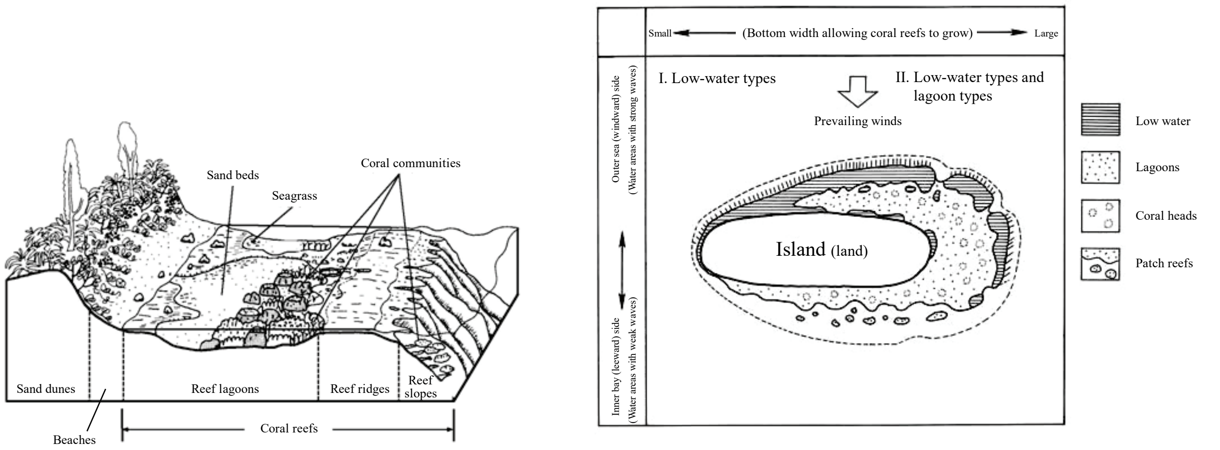

Fig. 3.6.3 Topography and Formation Factors for Coral Reef
(Left: Cross-Sectional Topography; Right: Planar Topography)[16)]

(3) The ecosystems of beaches comprise the ground and substances (topography, sediment and materials) which form the foundations of ecosystems and organisms which grow and make their habitats on the foundations. Thus, when examining the development of beaches, it is important to make assessments from various perspectives with respect to the stability of the ground and substances as well as the appropriate reproduction, migration, inhabitation and predation of organisms so as to enhance the functions according to the respective ecosystems as listed below.

① Stability of ground and substances

(a) Topographical conditions: topography, water depths and slopes

(b) Hydrographic conditions: waves, tides and tidal currents

(c) Meteorological conditions: winds and rainfalls

(d) Sediment and ground conditions: grain sizes, specific gravity and water contents

(e) Hydrological conditions: river discharges and supply of suspended matters

② Appropriate reproduction, migration, inhabitation and predation of organisms

(a) Water quality conditions: water quality

(b) Sediment conditions: sediment and organic accumulation amounts

(c) Biological conditions: migration, use of habitats depending on life cycles, food webs and biological networks

(d) Meteorological conditions: temperature and insolation

(4) When implementing monitoring surveys to evaluate the functions of the ecosystems, it is preferable to examine the scope, frequency and items of monitoring surveys taking into consideration the fluctuation of ecological functions depending on the surrounding environments.

(5) The performance verification of beaches with respect to the preservation of the natural environments can be used not only for the determination of whether the beaches satisfy the performance requirements at the time of verification, but also as part of the adaptive management method (**Fig. 3.6.4**) that incorporates the implementation of continuous monitoring surveys and the feedback of survey results. The adaptive management method is a functional system for achieving both individual and comprehensive goals through improvements in the construction and maintenance methods while taking into consideration fluctuations in the natural environments and changes in social requirements.[17), 18)]

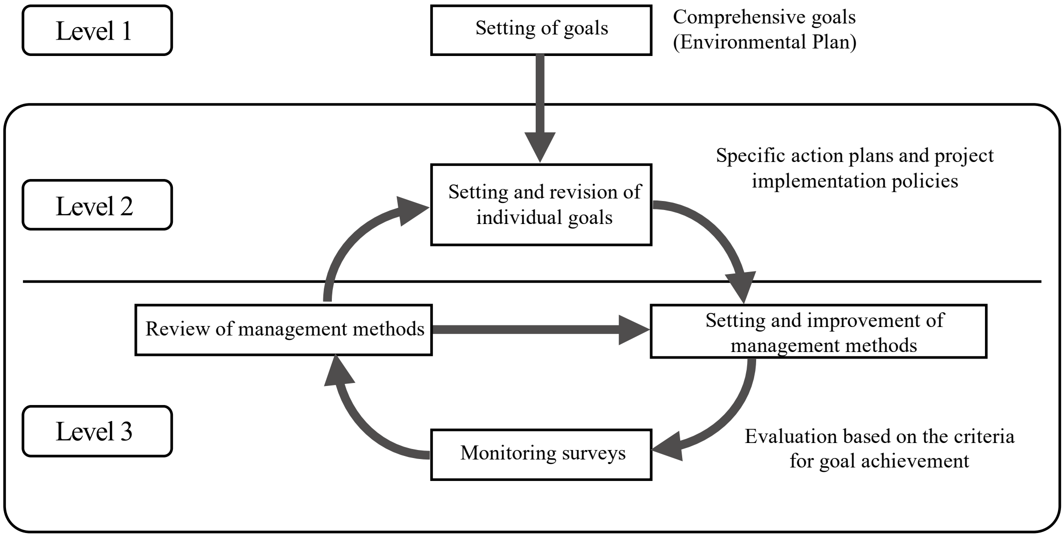

Fig. 3.6.4 Performance Verification through Feedback from Monitoring Survey Results to Achieve Comprehensive Goals[17), 18)]

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4 Green Spaces and Plazas

(English translation of this section from Japanese version is currently being prepared.)

5 Passenger Building

5.1 General

(English translation of this section from Japanese version is currently being prepared.)

5.2 Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

5.3 Ancillary Facilities

(English translation of this section from Japanese version is currently being prepared.)