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.

$$\gamma_i \delta_d / \delta_a \leq 1.0 \tag{1.1.6}$$

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)**:

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

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.

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

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.

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

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)**:

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

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.

$$\sigma_{se} = \frac{M_d}{A_s jd} \tag{1.2.5}$$

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)

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

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

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

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)**.

$$\gamma_i\, y_d / y_{lim} \le 1.0 \tag{1.2.12}$$

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.

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

(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.

[References]

1) JSCE: Standard Specification for Concrete Structures, Design, 2018
2) JSCE: Standard Specification for Steel and Composite Structures - General principle and Design, 2016
3) JSCE: Standard Specification for Hybrid Structures, 2014
4) Nagao, T: Reliability based design method for flexural design of caisson type breakwaters, Jour. JSCE No. 696/I-58, pp,173-184, 2002
5) Nagao, T.: Studies on the Application of the Limit State Design Method to Reinforced Concrete Port Structures, Rept. of PHRI Vol. 33 No.4, 1994, pp.69-113
6) Nagao, T.: Case Studies on Safety Factors about Seismic Stability for the Slob of Caisson Type Quaywalls, Technical Note of PHRI No.867, 1997
7) JSCE: Standard Specification for Concrete Structures, Construction, 2018
8) JSCE: Recommendation for design and construction of concrete structures using epoxy-coated reinforcing steel bars, Concrete library 112, 2003
9) JSCE: Recommendations for design and construction of concrete structures using stainless steel bars, Concrete library 130, 2008
10) Japan Road Association: Specifications for highway bridges and commentaries, 2017
11) JSCE: Concrete library 88, 1996
12) JSCE: Recommendations for design and constructions of prestressed concrete structures using advanced prestressing steel coated by epoxy resin, Concret library 133, 2010
13) JSCE: Recommendation for design and construction of ultra high strength fiber reinforced concrete structures, Concrete library 113, 2004
14) JSCE: Recommendation for design and construction of high performance fiber reinforced cement composite with multiple fine cracks (HPFRCC), Concrete library 127, 2007
15) JSCE: Recommendation for concrete repair and surface protection of concrete sturctures, Concrete library 119, 2005
16) JSCE: Guideline on design application methods of silicate-based surface penetrants used for concrete structures, Concrete library 137, 2012

17) JSCE: Recommendation for design and construction of electrochemical corrosion control method, Concrete library 107, 2001

18) JSCE: proposals for improving concrete construction productivity while ensureing quality, Concrete library 148, 2016

19) JSCE: An example of design calculations based on JSCE specifications for conrete structures - reinforced concrete deck in open-piled pier -, Concrete library 116, 2005

20) Nogami, S., E. Kato, Y. Kawabata and T. Sato: Life-Cycle cost analysis of maintenance scenario for concrete superstructure of open-type wharf, Technical Note of PARI No.1296, 2014

21) Yamaji, T., S. Nakano and H. Hamada: Study on the estimatino method of surface chloride ion content in port concrete structures, Rept. of PARI Vol. 44 No. 3, pp.39-75, 2005

22) Yamaji, T., H. Yokota, S. Nakano and H. Hamada: Study on the verification of steel corosion in port reinforced concrete structures based on site survey and long term exposure test, Jour. JSCE Vol.64, No.2, pp.335-347, 2008

23) JSE: Test method for effective diffusion coefficient of chloride ion in concrete by migration (JSCE-G 571), Standard speciications for concrete structures, Test methdos and specifications, 2013

24) JSE: Test method for apparent diffusion coefficient of chloride ion in concrete by submergence in salt water (JSCE-G 572), Standard speciications for concrete structures, Test methdos and specifications, 2013

25) Yonamine, K., T. Yamaji, E. Kato and Y. Kawabata: Study on chloride ion diffusion property of concrete based on long-term exposure test and investigation to real structure, Technical Note of PARI No.1339, 2018

26) Yamaji, T., T. Aoyama and H. Hamada: Effect of exposure environment and kinds of cement on durability of marine concrete, Proceedings of annual conference on concrete engineering, Vol.23, No.2, pp.577-582, 2001

27) Coastal Development Institute of Technology: Corrosion protection and repair manual for port and harbor steel structures, 2009

28) Coastal Development Institute of Technology: Corrosion protection and repair manual for port and harbor steel structures, pp.74-80, 2009

29) Yamaji, T., et al.: Rust prevention and control Japan 52(2), pp.41-44, 2008

30) Tado, H., et.al.: Statistical evaluation of the cathodic protection effect in port facilities using test pieces, 36 th Bosei Boshoku Gijyutu Happyo Taikai Koen Yokoshu, pp.45-48, 2016

31) Coastal Development Institute of Technology: Corrosion protection and repair manual for port and harbor steel structures, pp.73, 2009

32) Yamaji, T., et al.: Study on the cathodic protection design in consideration of the soil resistivity and cathodic protection characteristics in seabed soil of the harbor steel structures, Technical Note of PARI No.1314, 2015

33) Karasawa, T. and Kaneko, S.: Experiment on electrolytic protection of a masonry structure in port facilities, Proceedings of Fushoku Boshoku '91, pp.199-202, 1991

34) Coastal Development Institute of Technology: Corrosion protection and repair manual for port and harbor steel structures, pp.79, 2009

35) Trans-Tokyo Bay Highway Corporation: Guidelines for Trans-Tokyo Bay Highway bridge and steel structures (draft), Jacket type steel revetment edition, p.65, 1988

36) Coastal Development Institute of Technology: Corrosion protection and repair manual for port and harbor steel structures, pp.52-65, 2009

37) Yamaji, T., et al.: Studies on the ewtimation of durability of protective methods for steel pipe pile based on the long term marine exposure test (after 30 years), Technical Note of PARI No.1324, 2016

38) PWRI: Cooperative investigation on protecteve technologies for concrete structures in marine splash environment, No.480, 2016

39) JSCE: Standard Specification for Steel and Composite Structures -General principle and Construction, 2016

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)

<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 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="3">7</td><td rowspan="3">1</td><td rowspan="3">–</td><td rowspan="3">23</td><td rowspan="3">–</td><td rowspan="3">1</td><td rowspan="3">Serviceability</td><td rowspan="3">Permanent</td><td rowspan="3">Self-weight</td><td rowspan="3">Water pressure, subgrade reaction, surcharge, earth pressure</td><td>Cross-sectional failure of bottom slab and footing</td><td>Design ultimate capacity</td></tr>
<tr><td>Concrete stress in cross section of bottom slab and footing</td><td>Bending compressive stress</td></tr>
<tr><td>Extrusion of bottom slab and footing from partition wall (yield of reinforcing bars)</td><td>Design yield stress</td></tr>
</table>

(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)

<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 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">23</td><td rowspan="4">-</td><td rowspan="4">1</td><td rowspan="4">Serviceability</td><td rowspan="4">Variable</td><td rowspan="2">Variable waves*1)</td><td rowspan="3">Self-weight, water pressure, subgrade reaction, surcharge, earth pressure</td><td>Cross-sectional failure of bottom slab and footing</td><td>Design ultimate capacity</td></tr>
<tr><td>Extrusion of bottom slab from partition wall (yield of reinforcing bars)</td><td>Design yield stress</td></tr>
<tr><td>Variable waves*2)</td><td>Cracking in cross section of bottom slab and footing</td><td>Crack widthcaused by bending</td></tr>
<tr><td>Cyclic action of waves*3)</td><td></td><td>Fatigue failure of bottom slab and footing</td><td>Design fatigue strength</td></tr>
</table>

*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)

<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 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">23</td><td rowspan="4">-</td><td rowspan="4">1</td><td rowspan="4">Serviceability</td><td rowspan="4">Variable</td><td rowspan="2">Water pressure during flotation</td><td rowspan="2">Self-weight</td><td>Cross-sectional failure of bottom slab and footing</td><td>Design ultimate capacity</td></tr>
<tr><td>Cracking in cross section of bottom slab and footing</td><td>Crack width caused by bending</td></tr>
<tr><td rowspan="2">Level 1 earthquake ground motion</td><td rowspan="2">Self-weight, water pressure, subgrade reaction</td><td>Cross-sectional failure of bottom slab and footing</td><td>Design ultimate capacity</td></tr>
<tr><td>Extrusion of bottom slab from partition wall (yield of reinforcing bars)</td><td>Design yield stress</td></tr>
</table>

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

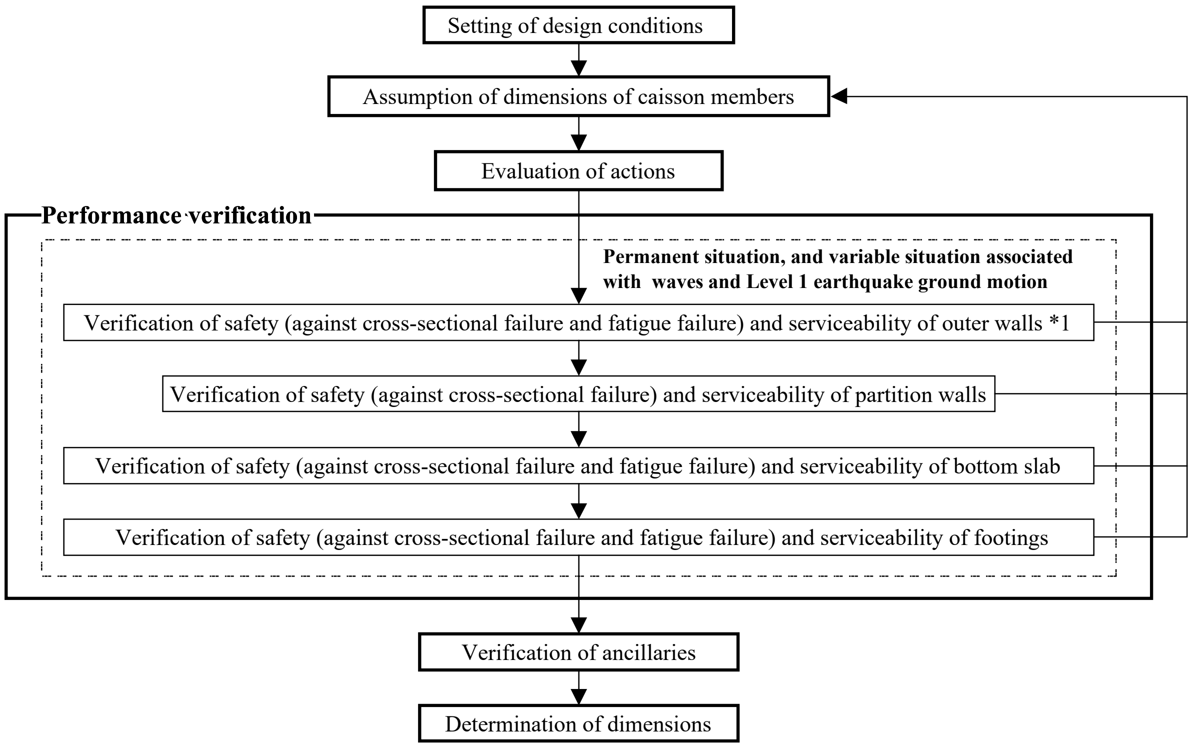
Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	23	-	4	Serviceability	Variable	Water pressure during flotation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

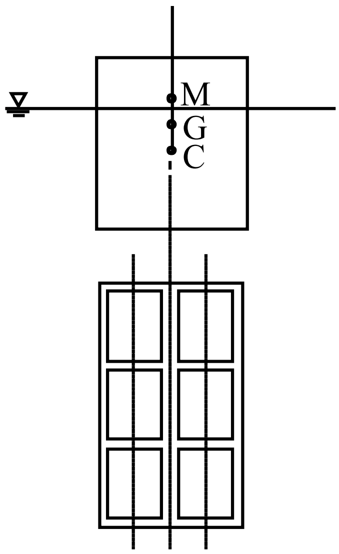


Fig. 2.2.2 Stability of Caisson

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)

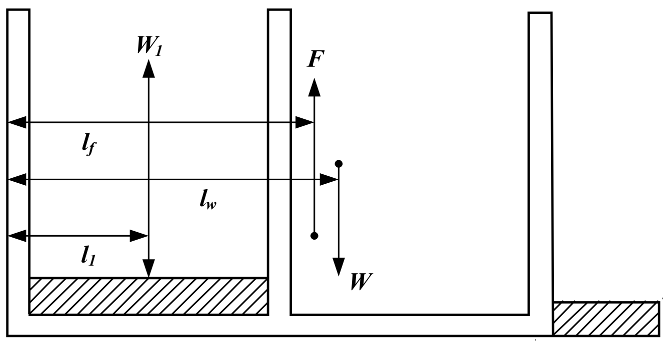


Fig. 2.2.3 Stability of Caisson with Counter Ballast

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**.

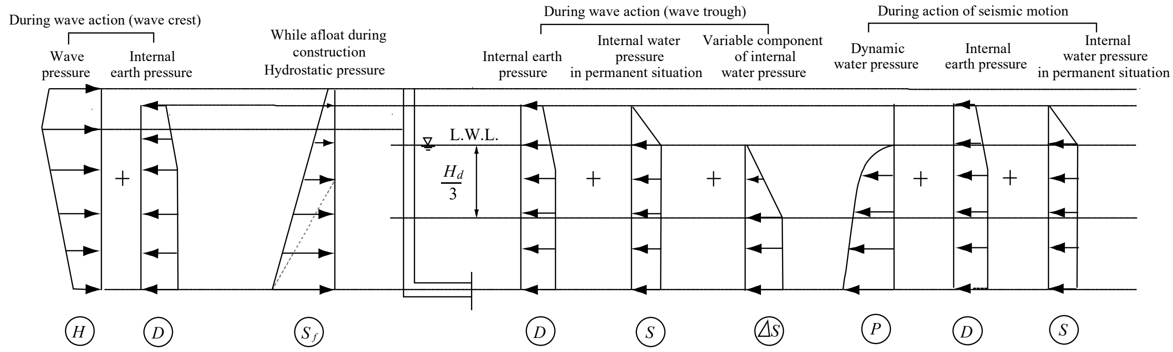
Table 2.2.1 Combinations of Actions and Load Factors
(a) Breakwaters

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: Installation	Loads during construction: Still water	Remarks
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.)
In service	Permanent situation associated with internal earth pressure			1.1 (1.0)	1.1 (1.0)							Outer wall
In service	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.)
In service	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 while afloat
During construction	Variable situation associated with water pressure while afloat										1.1 (0.5)	Outer wall while afloat
During construction	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	$1.1D+1.1S+1.2\Delta S$	$1.0D+1.0S+1.0\Delta S$
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**.

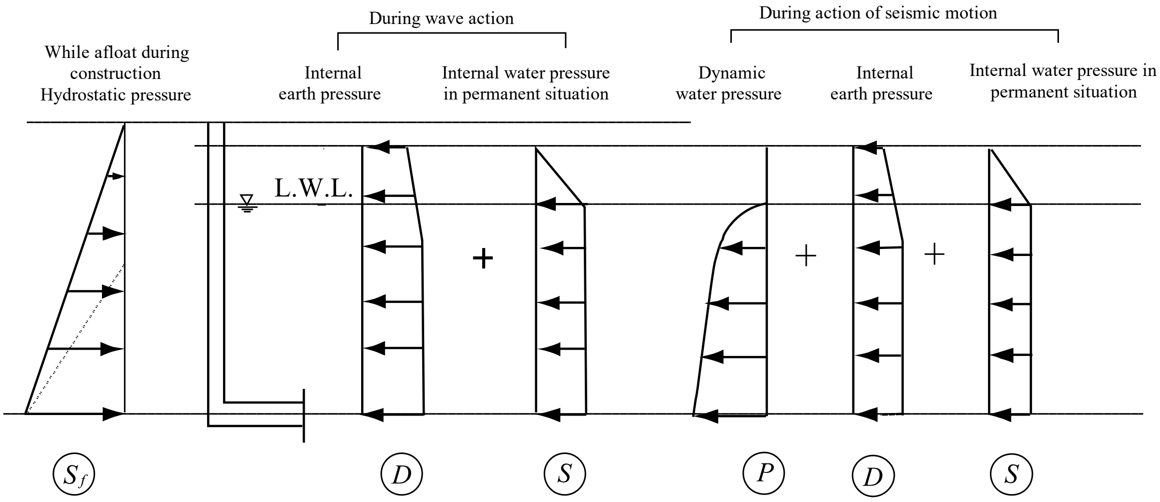
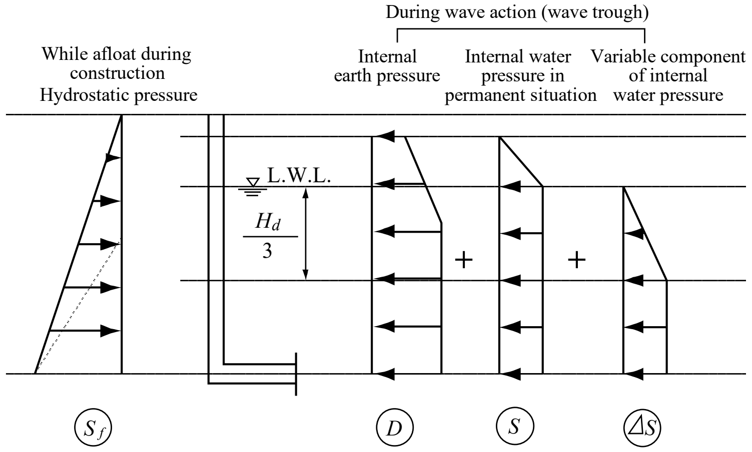


Fig. 2.2.5 Actions on Rear Wall (parallel to face line: landward side) (Breakwater)

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.

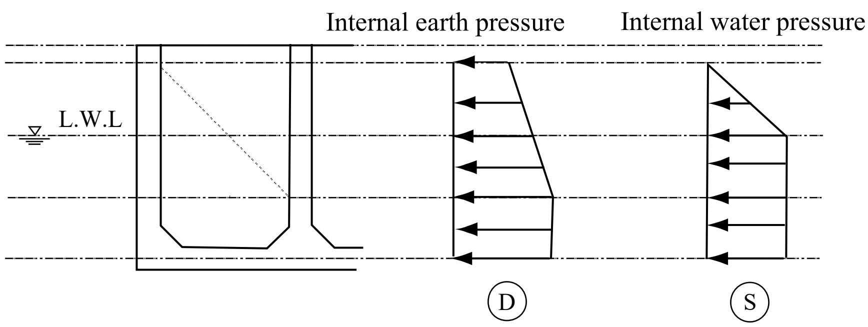
Fig. 2.2.6 Actions on Side Wall (perpendicular to face line) (Breakwater)

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

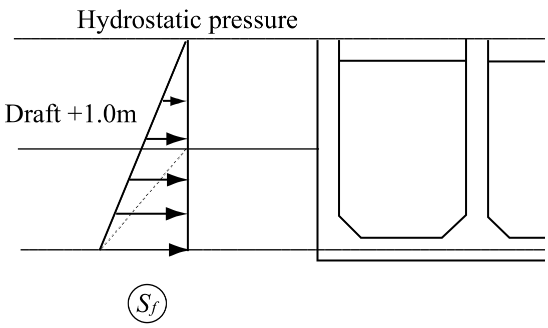
Design situation	Direction of action	Safety (against cross-sectional failure)	Serviceability
Variable situation associated with water pressure while afloat during construction	From outside of caisson	$1.1S_f$	$0.5S_f$
Variable situation associated with waves during action of wave trough	From inside of 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**.

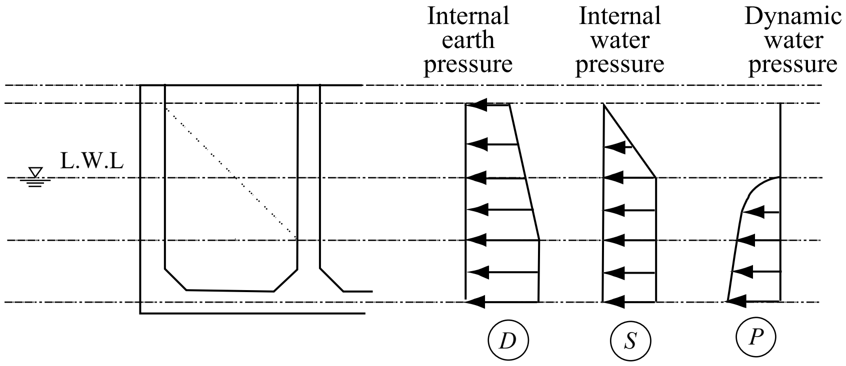


(a) Under calm conditions (actions from inside)



(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**.

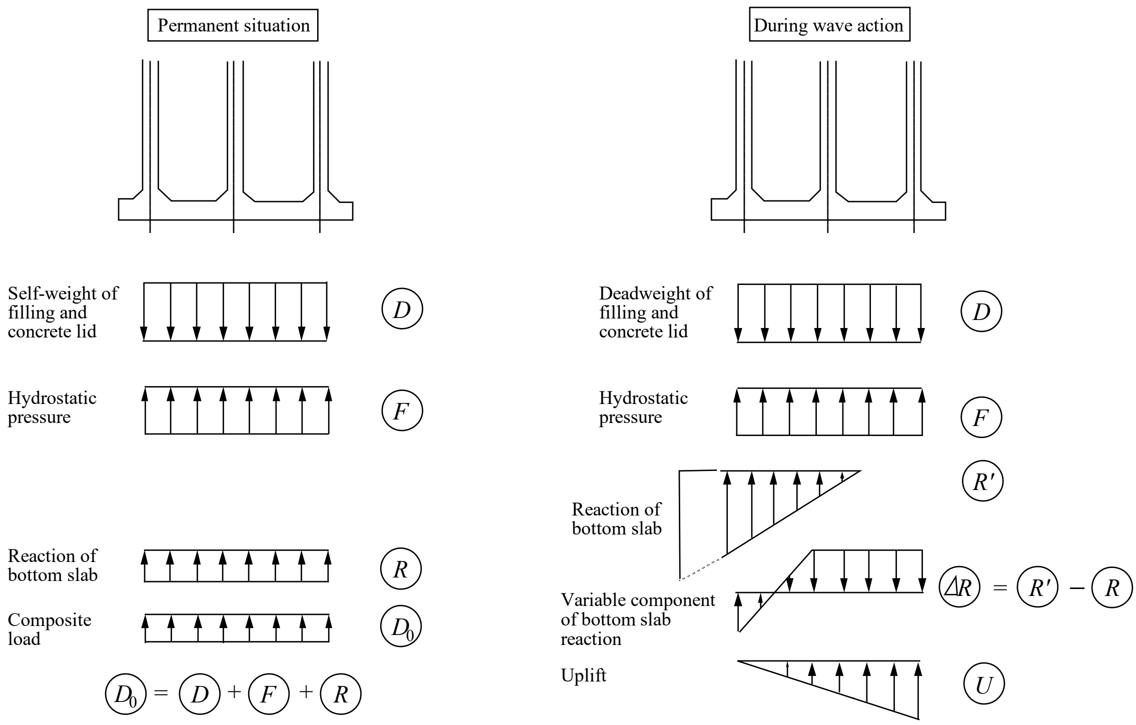


Fig. 2.2.8 Actions on Bottom Slab (Breakwater)

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

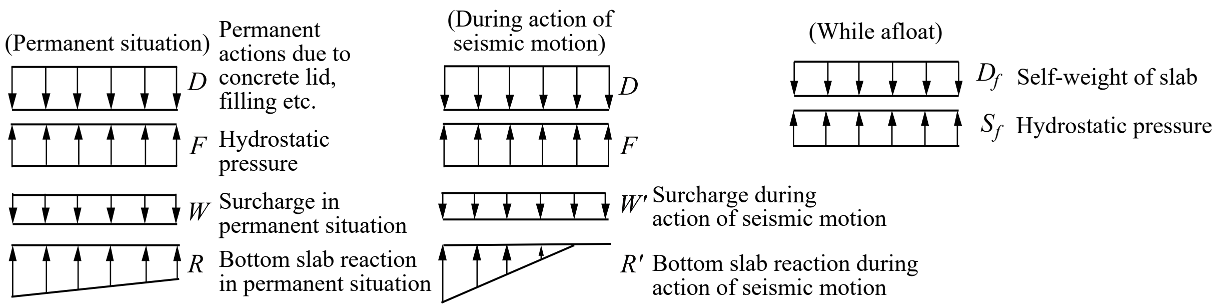
Design situation	Combination of actions
Permanent situation	$1.0D_0+1.0F+1.0R$
Variable situation associated with waves	$1.0D_0+1.0\,\Delta R+1.0U$

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**.

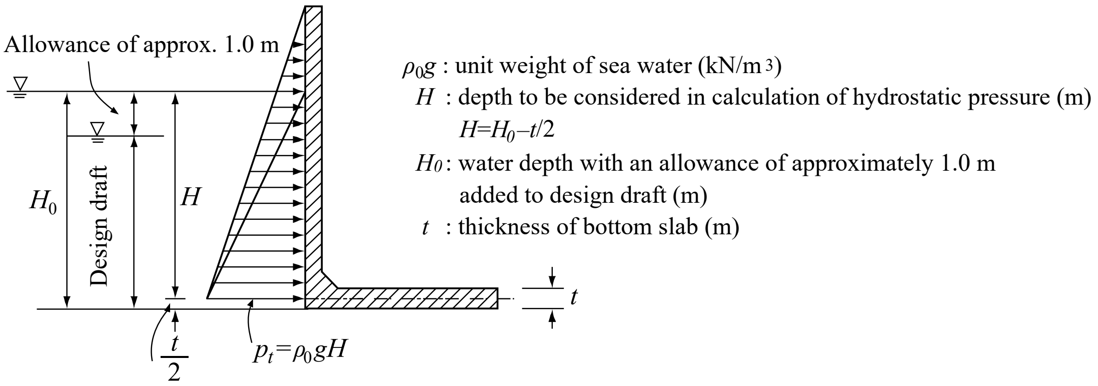


Fig. 2.2.10 Water Pressure Acting on Outer Wall

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**.

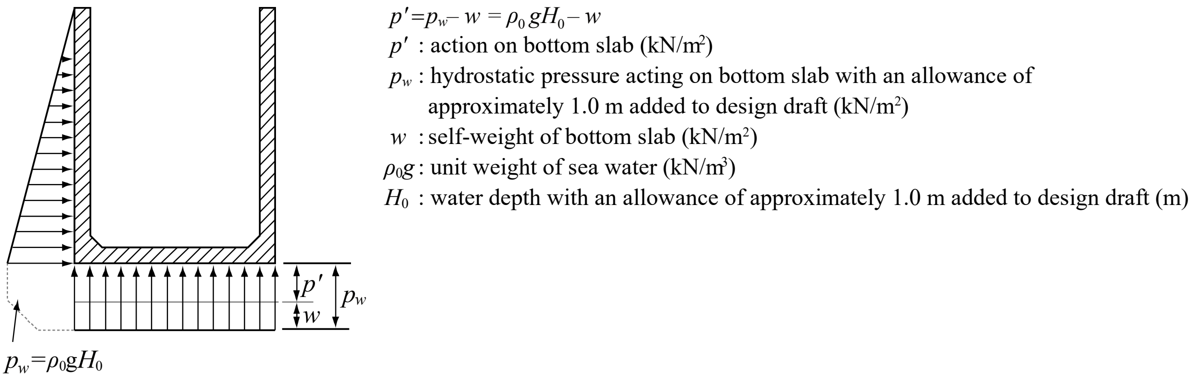


Fig. 2.2.11 Actions on Bottom Slab

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**.)

$$T=\frac{1}{2}\rho_0 C_D V^2 A \qquad \textbf{(2.2.5)}$$

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)

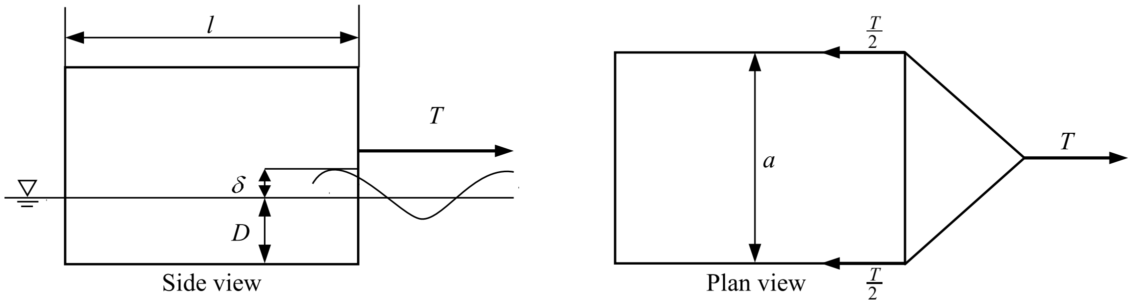


Fig. 2.2.12 Tensile Force during Towing

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**.

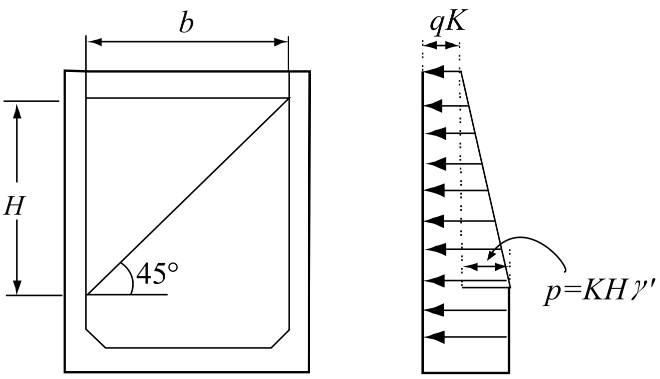


Fig. 2.2.13 Determination of Internal Earth Pressure

(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.

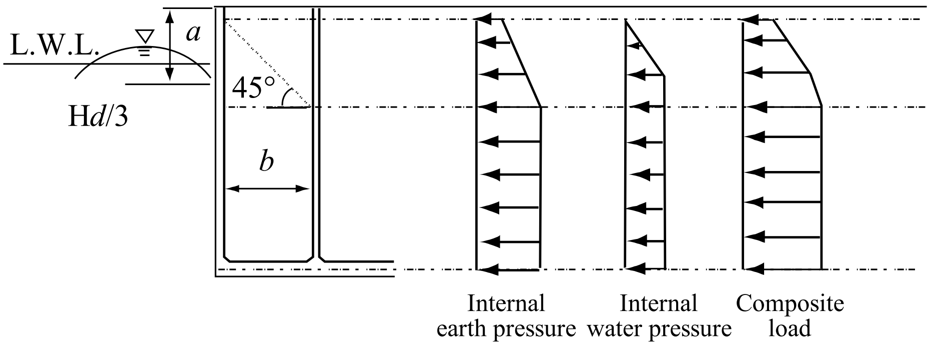
(d) The way of determining the internal earth pressure shown in **Fig. 2.2.13** was established for convenience based on past records and experiments, not based on measurements. For example, according to **Reference 7)**, the earth pressure distribution of filling sand increases almost monotonically 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)**.

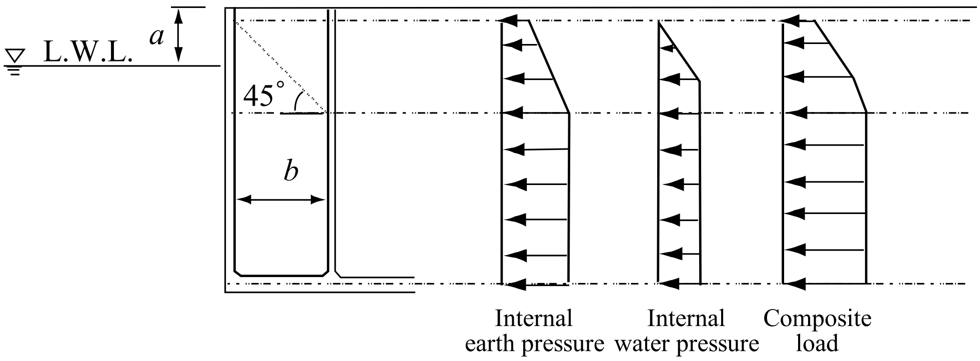
④ For the front wall of a breakwater caisson, the wave force shall be taken into account when wave crests act on the wall surface.[8), 9)]

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

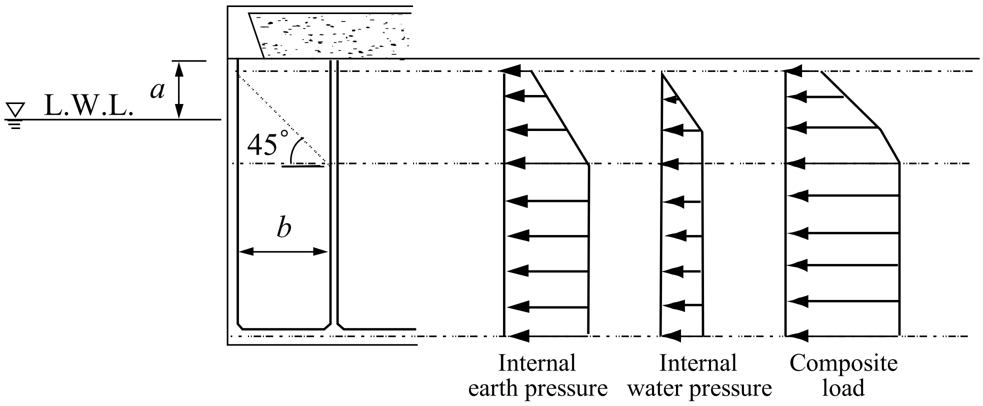


*In this figure, H_d stands for design wave height.
In verification of the safety (against cross-sectional failure), $H_d = H_{max}$ may be assumed.

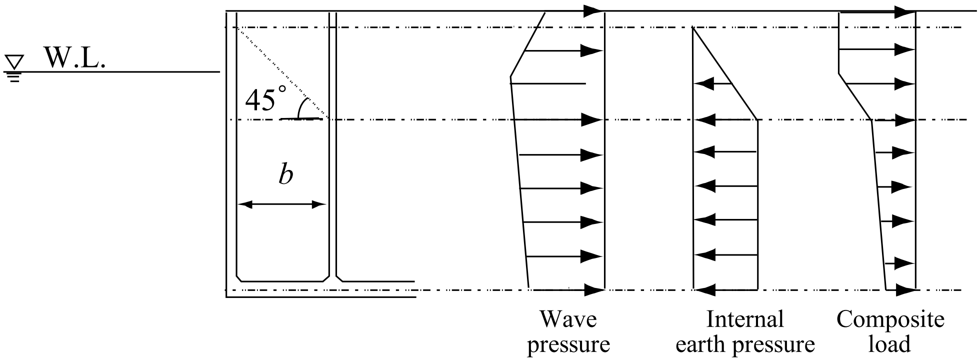
(a) Breakwaters (front wall and side walls perpendicular to face line)



(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**.

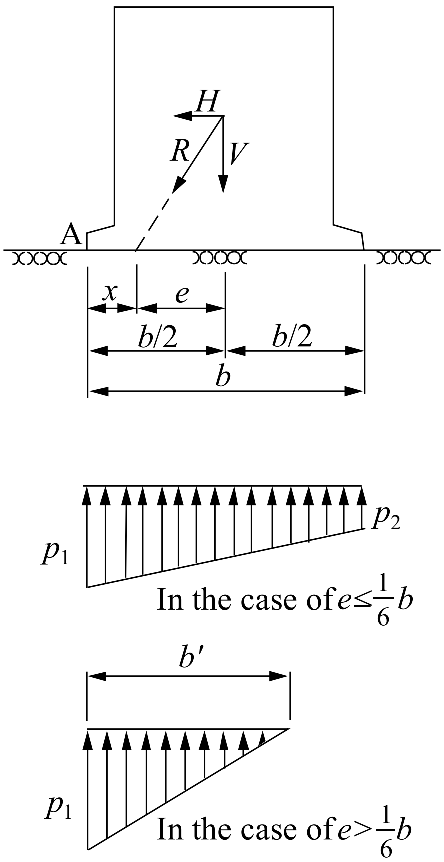


Fig. 2.2.15 Bottom Reaction

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

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 $e \le \frac{1}{6}b$

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

(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**.

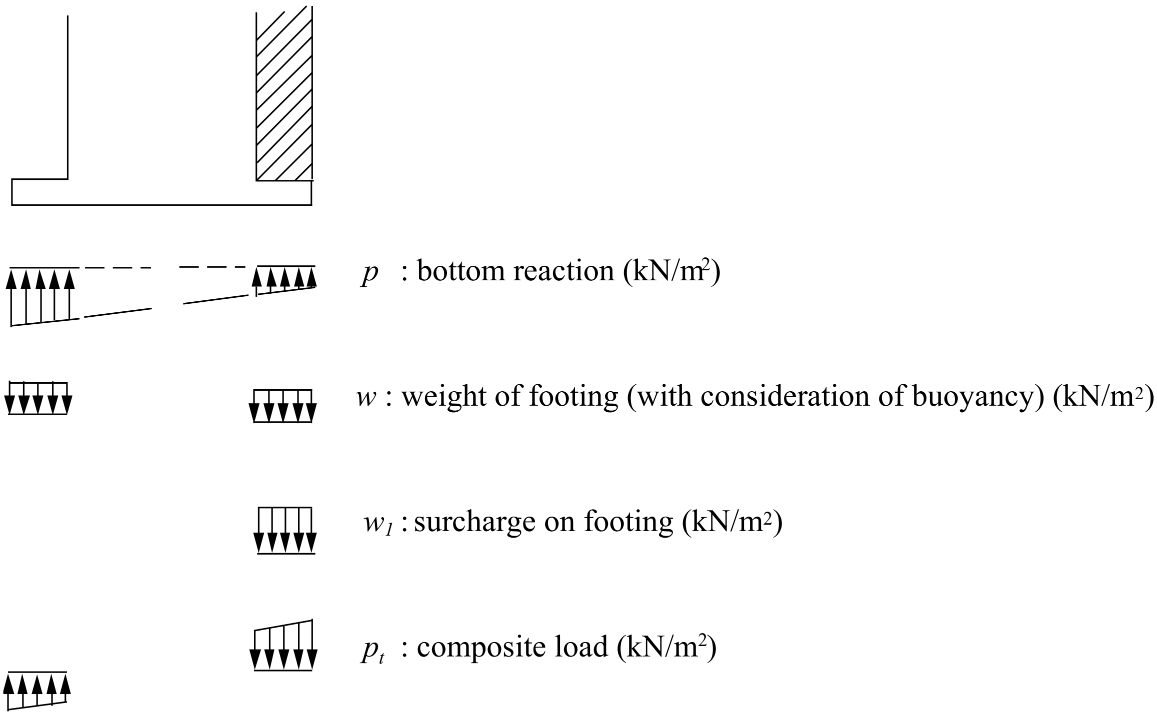


Fig. 2.2.16 Actions on Footings

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**).

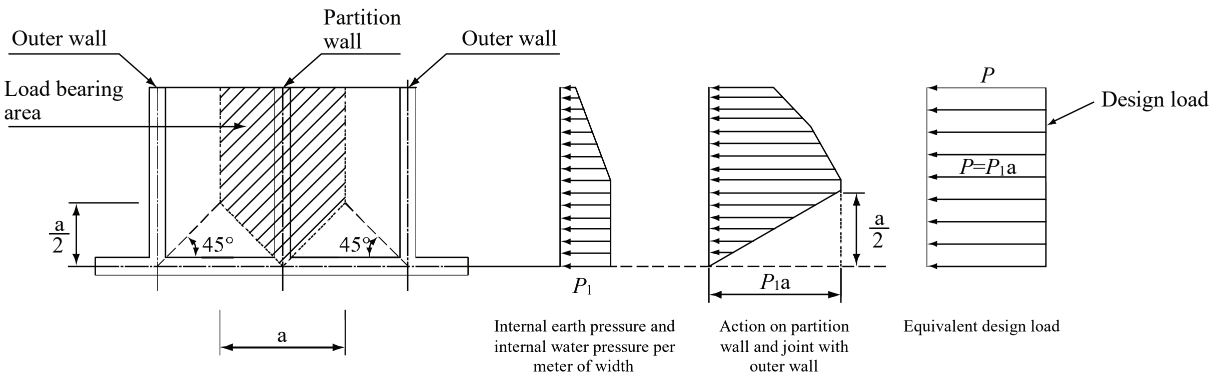


Fig. 2.2.17 Actions to be Considered in Examination of Extrusion of Outer Wall from Partition Wall

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**).

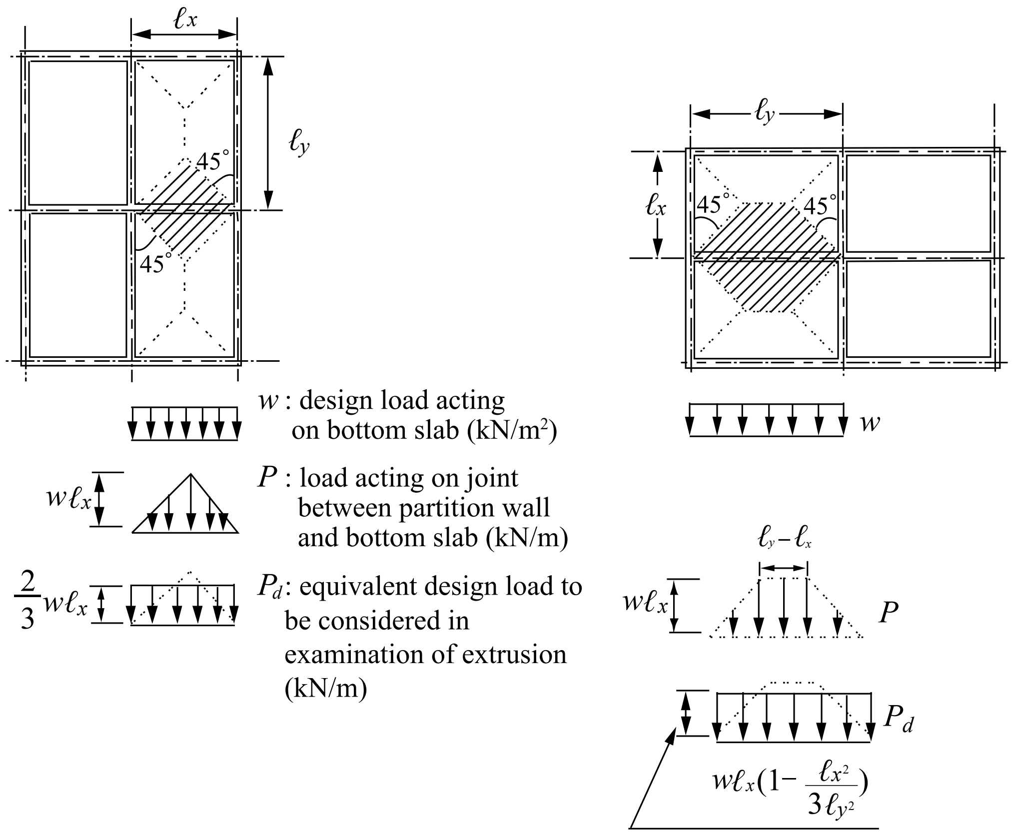


Fig. 2.2.18 Actions to be Considered in Examination of Extrusion of Bottom Slab from Partition Wall

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.

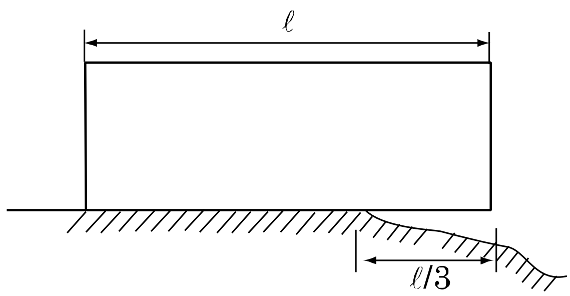


Fig. 2.2.19 Examination of Action due to Non-uniformity of Ground Bearing Capacity

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.

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

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.

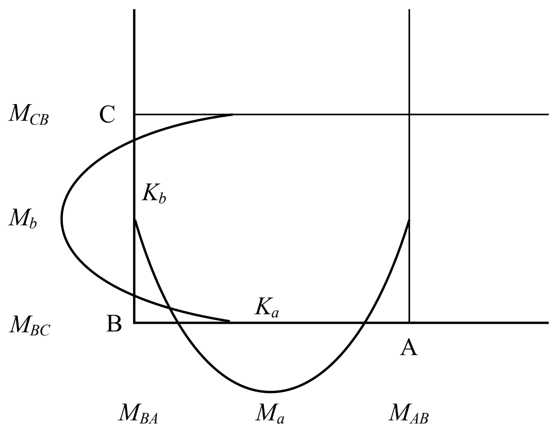


Fig. 2.2.20 Example of Distribution of Unbalanced Moments

(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)

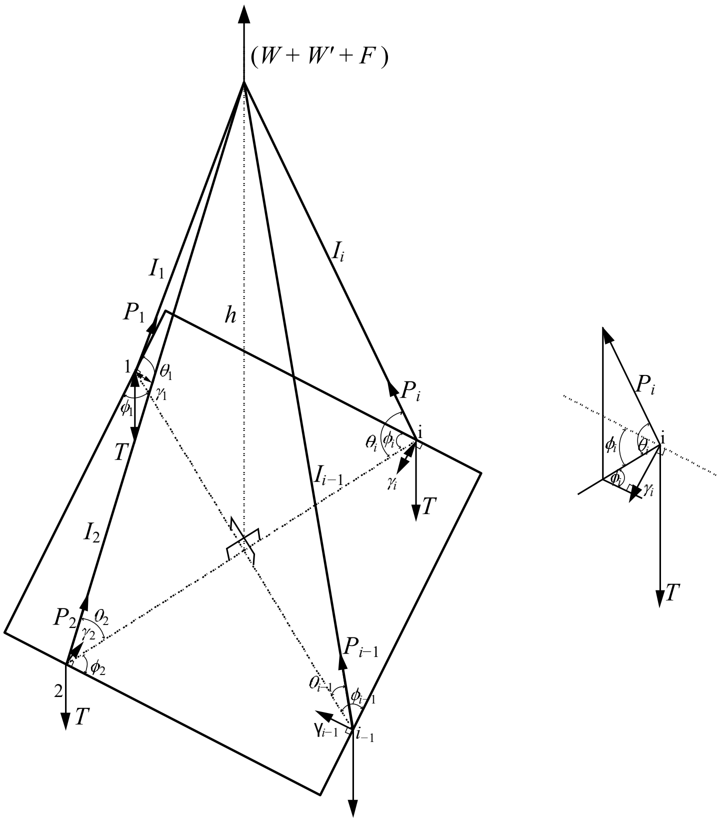


Fig. 2.2.21 Actions on Suspension Hooks during Direct Lifting

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.

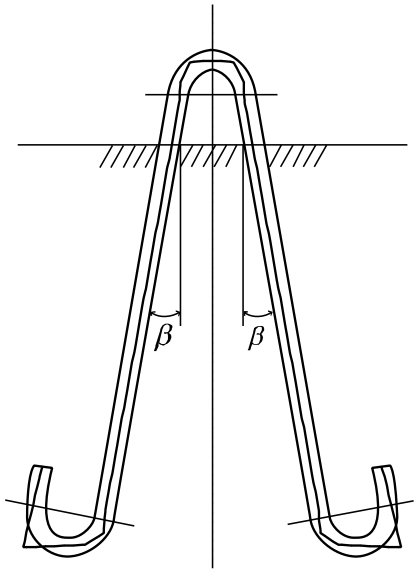


Fig. 2.2.22 Shape of Suspension Hook

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)

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

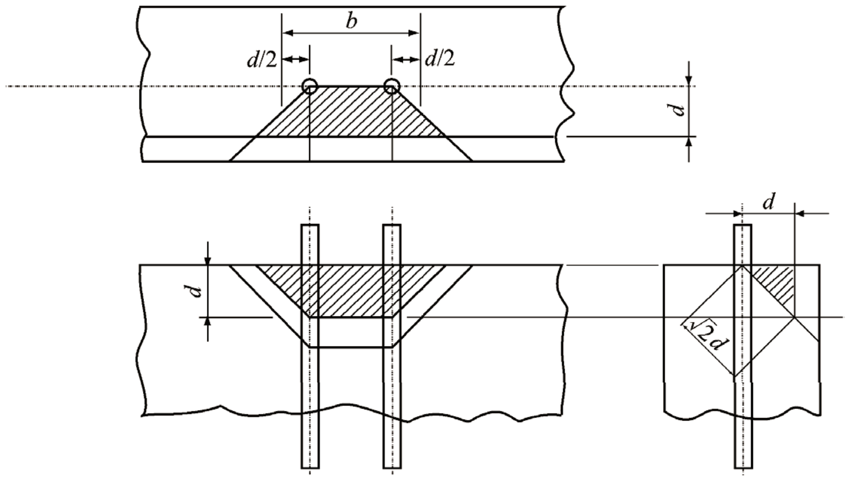
A_s : amount of horizontal reinforcing bars in shear plane (mm^2)

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)

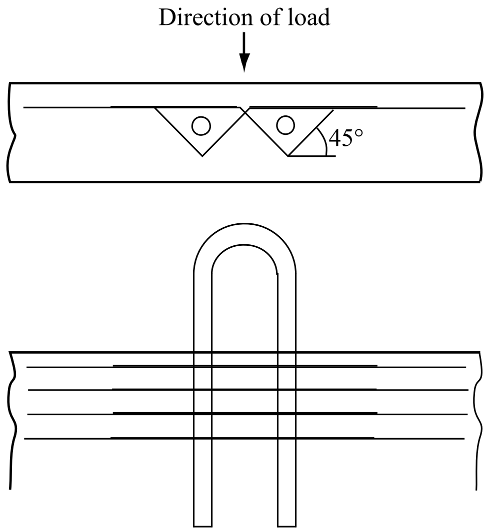


Shaded areas in the figure show the broken part.

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.



Reinforcing bars are arranged in the part from top of caisson to a depth 3 to 4 times the diameter of the suspension hook.

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

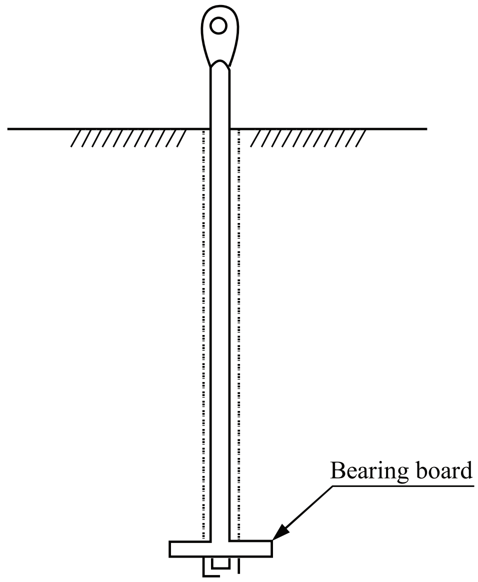


Fig. 2.2.25 Shape of Improved Suspension Hook (Nut-Shaped)

(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**).

$$L = l_1 + l_2 + l_3 \tag{2.2.14}$$

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)

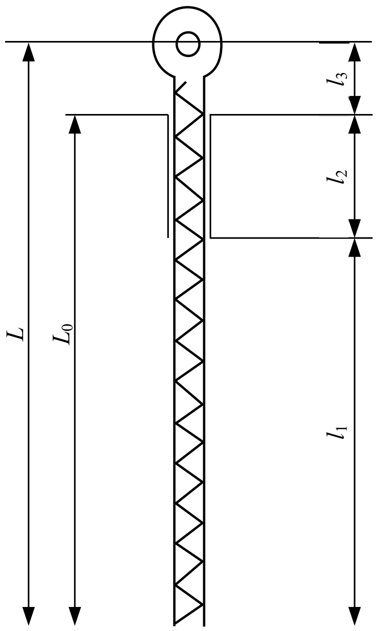


Fig. 2.2.26 Embedded Length of Suspension Hook

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**).

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

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

$$\frac{P_d}{V_{1d}} \leq 1.0 \qquad \textbf{(2.2.15)}$$

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

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)

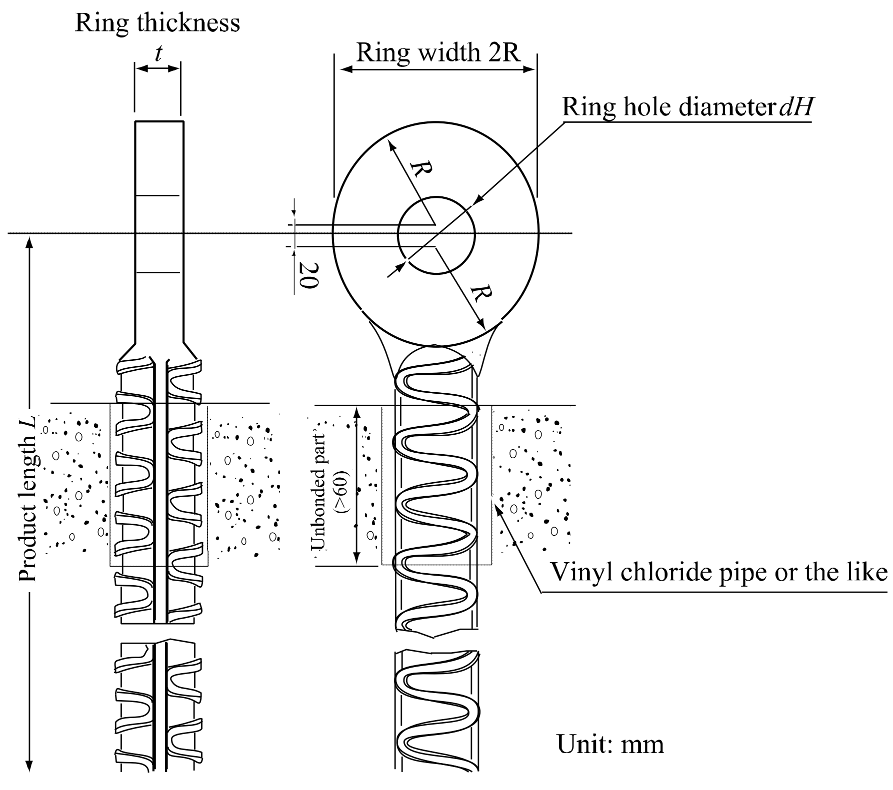


Fig. 2.2.27 Detailed Drawing of Suspension Hook

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

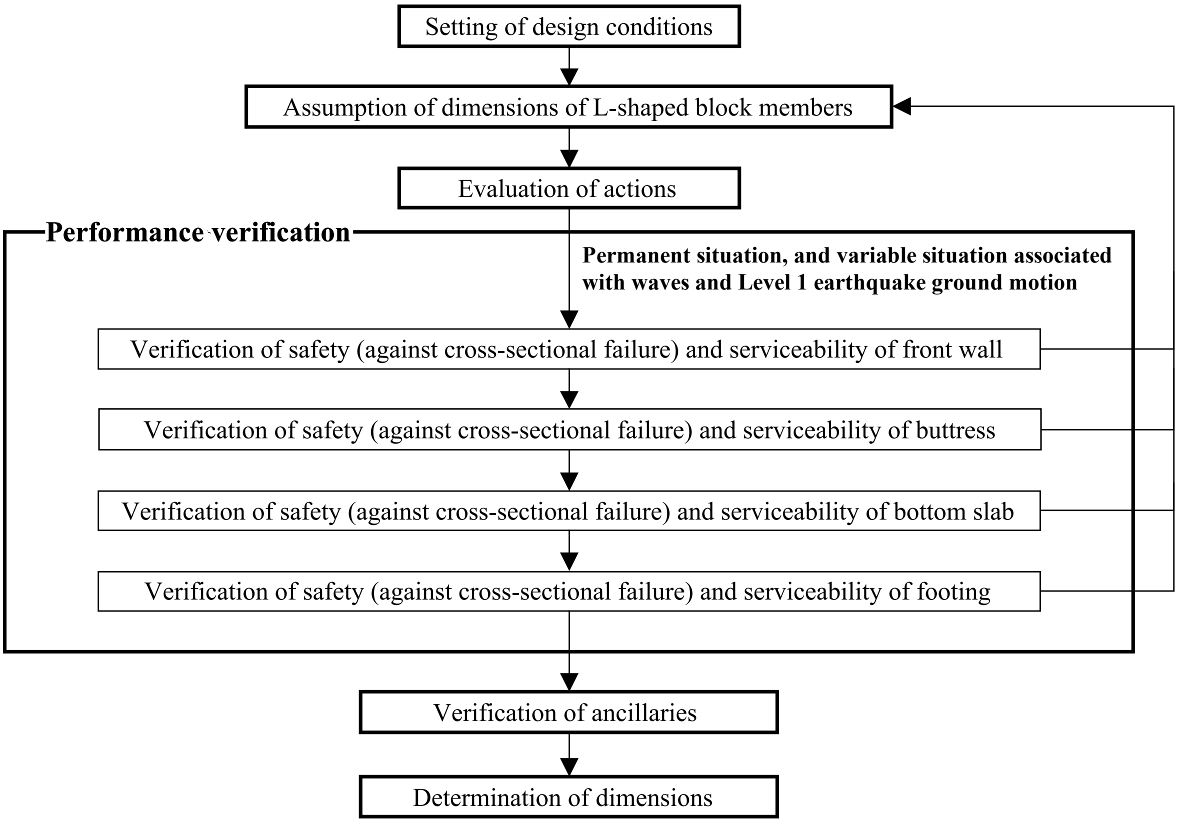
Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
7	1	-	24	-	-	Serviceability	Permanent	Earth pressure	Water pressure, 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:

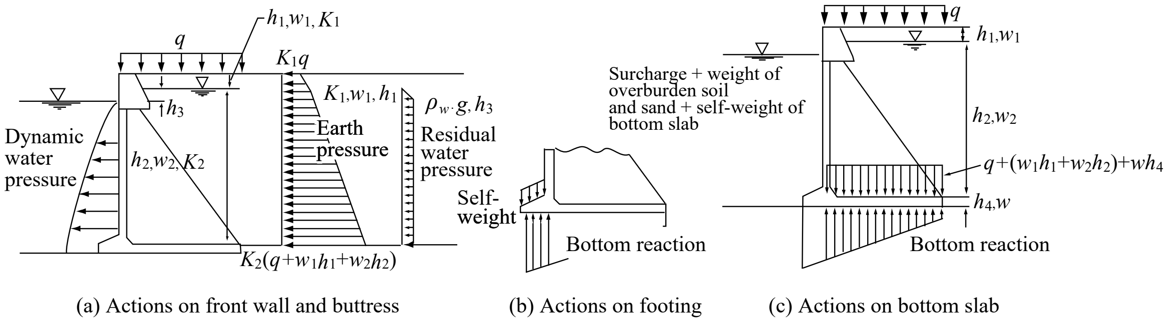
① Capacity of L-shaped block fabrication facilities
② Hoisting capacity of crane
③ Water depth at which L-shaped blocks are to be installed (mound water depth)
④ Service conditions after installation of L-shaped blocks (backfilling and superstructure construction)

(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)

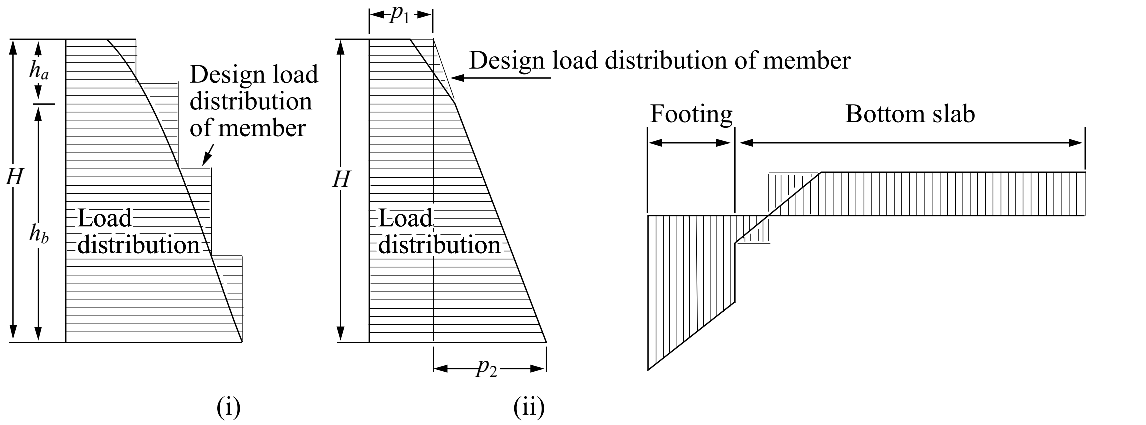
Fig. 2.3.2 Actions on L-shaped Blocks

(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.

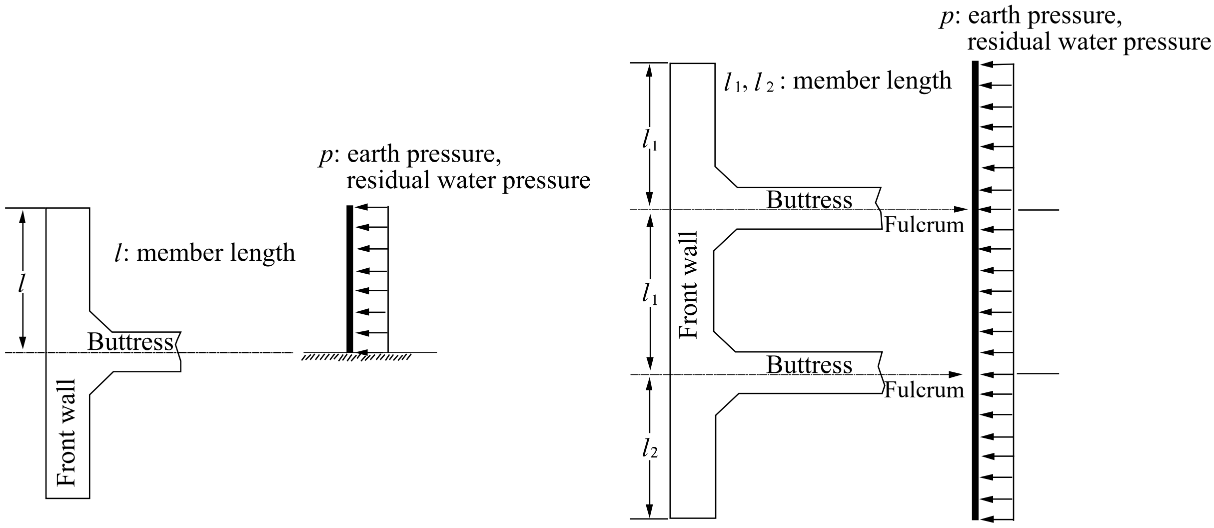
② In cases where the front wall is supported by a single buttress or by two or more buttresses, performance verification can be performed assuming the front wall as a buttressed cantilever slab or a continuous slab respectively.

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

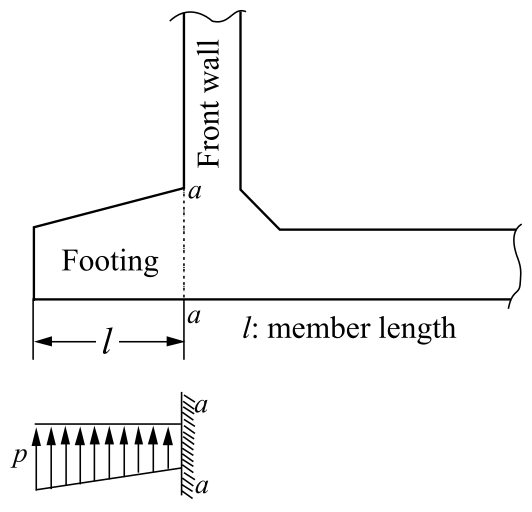
Fig. 2.3.4 Member Length of Front Wall and Actions on It

(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**.



$p =$ (bottom reaction) – (self-weight of footing)

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.

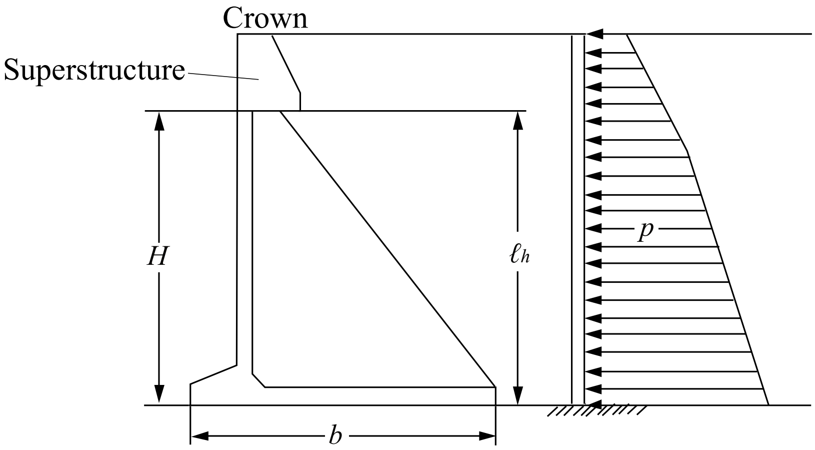
③ Performance verification of a buttress shall be performed for the cross sections parallel to the bottom slab.

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

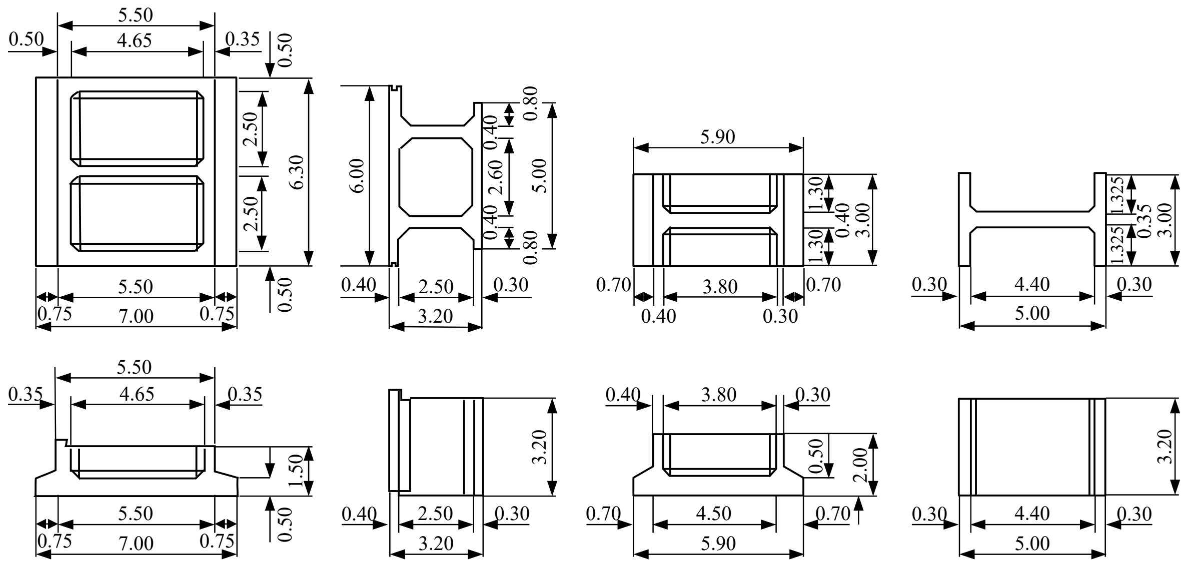
(5) Performance Criteria of Cellular Blocks (Article 7, paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 25 of the Public Notice)

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.

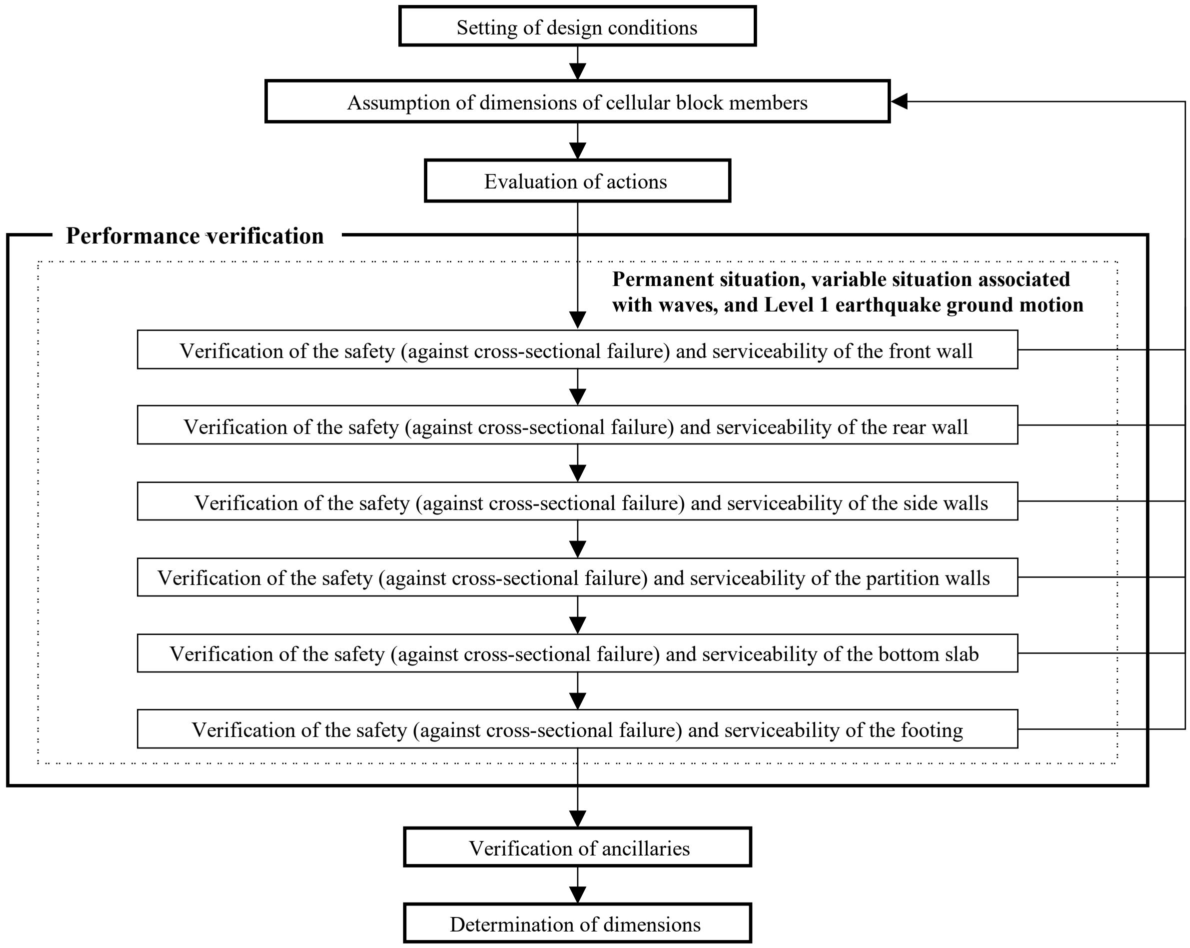
(2) Cellular blocks have various cross-sectional shapes. **Fig. 2.4.1** shows examples of cellular blocks.



(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.

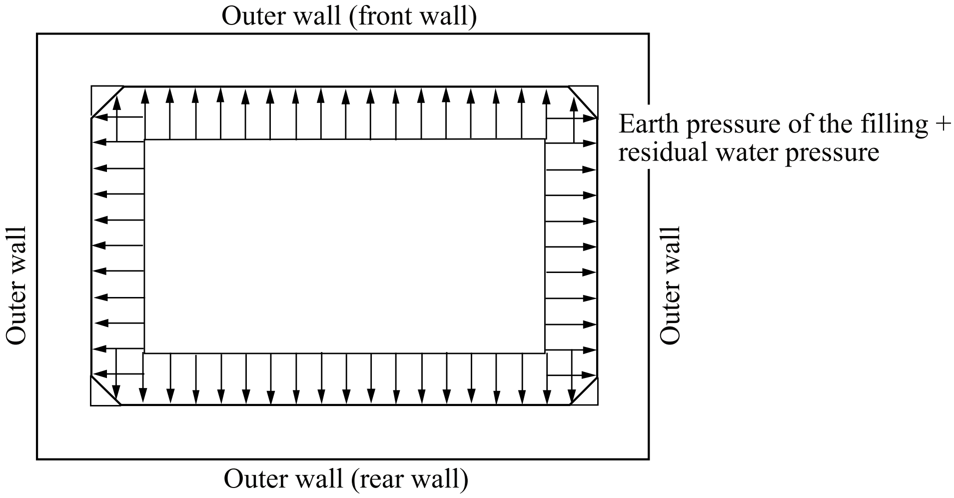


Fig. 2.4.3 Actions on the Cellular Block]

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

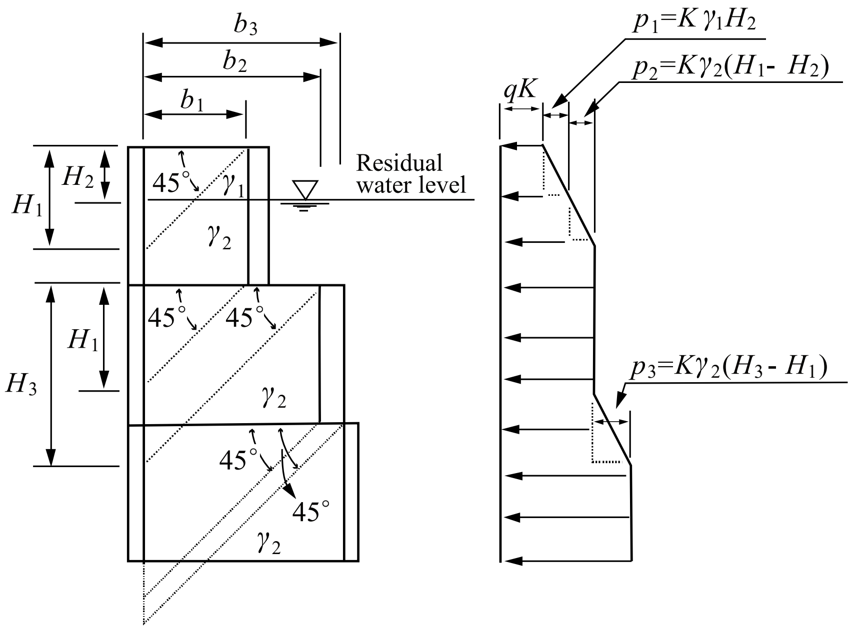
① As actions on the front wall, rear wall, and side walls of a cellular block, the internal earth pressure and residual water pressure shall be taken into account. However, in cases wherein cast-in-place concrete is placed on top of the cellular block to a degree such that the surcharge may not affect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

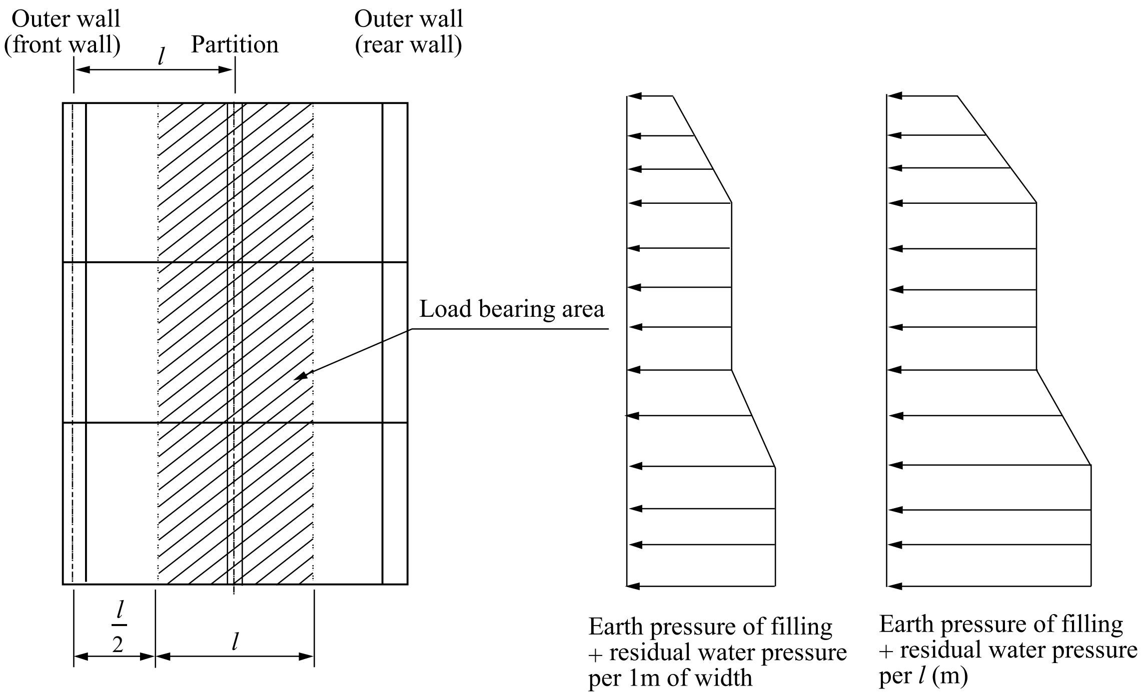


Fig. 2.4.5 Determination of Actions That Affect the Extrusion of the Outer Walls from the Partition Wall

(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)**.

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

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

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

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

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

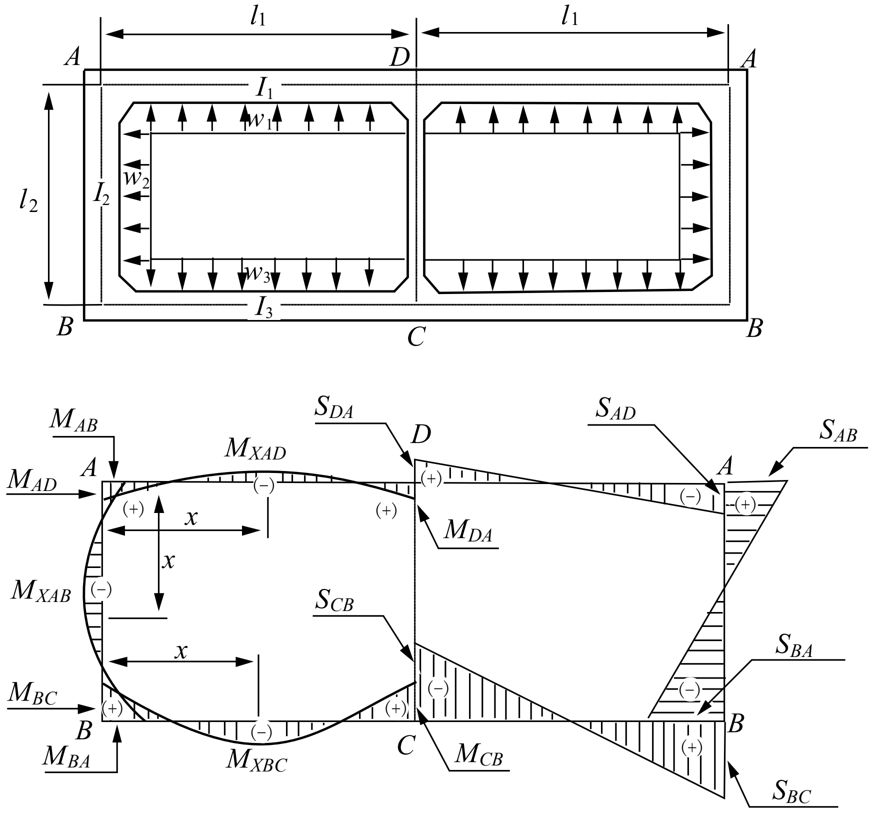


Fig. 2.4.6 Actions and Stresses on Rigid Frame

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

Bending moment of member BC

$$M_{x_{BC}} = M_{BC} + S_{BC}x + \frac{w_3}{2}x^2$$

Bending moment of member AB

$$M_{x_{AB}} = M_{AB} + S_{AB}x + \frac{w_2}{2}x^2$$

Bending moment of member AD

$$M_{x_{AD}} = M_{AD} + S_{AD}x + \frac{w_1}{2}x^2$$

(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).

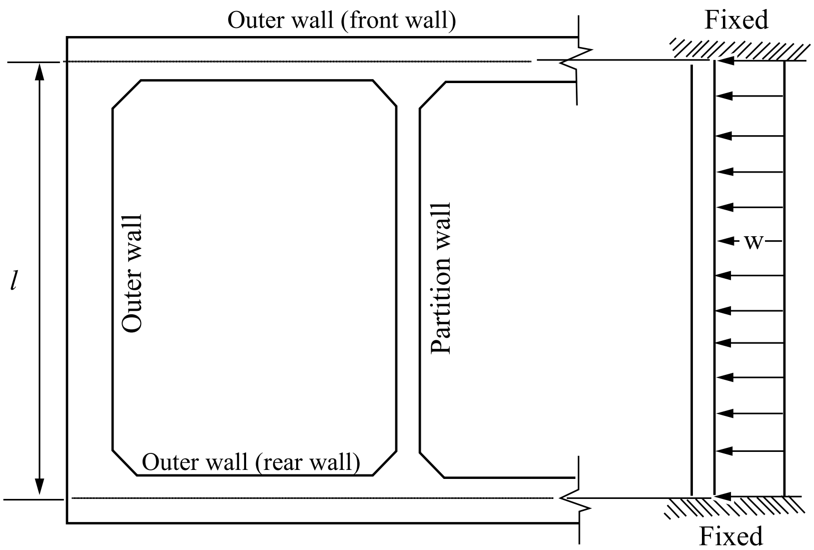


Fig. 2.4.7 Determination of the Member Length of the Partition Wall and the Actions on It

(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.

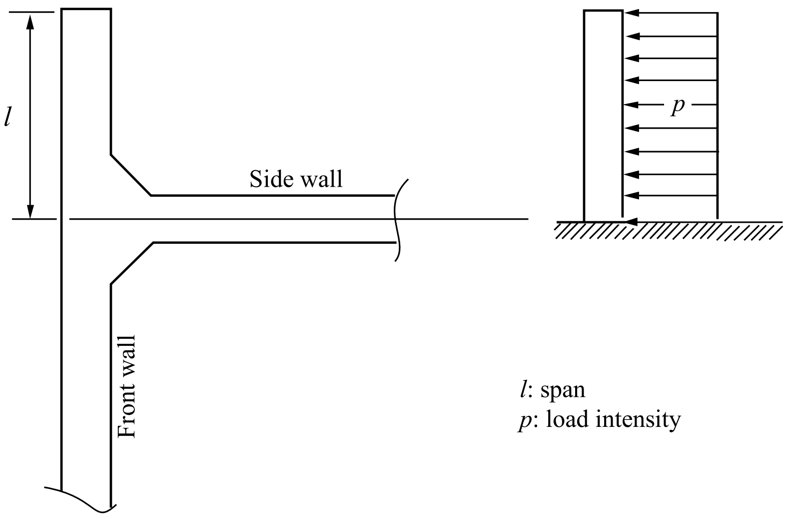


Fig. 2.4.8 Member Length of the Front Wall and the Actions on It on a Cross Section with a Projected Edge

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**.

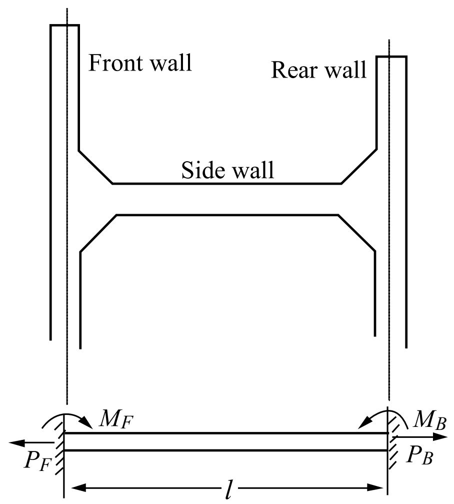


Fig. 2.4.9 Member Length of the Side Wall and the Actions on It on a Cross Section with Projected Edges

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.

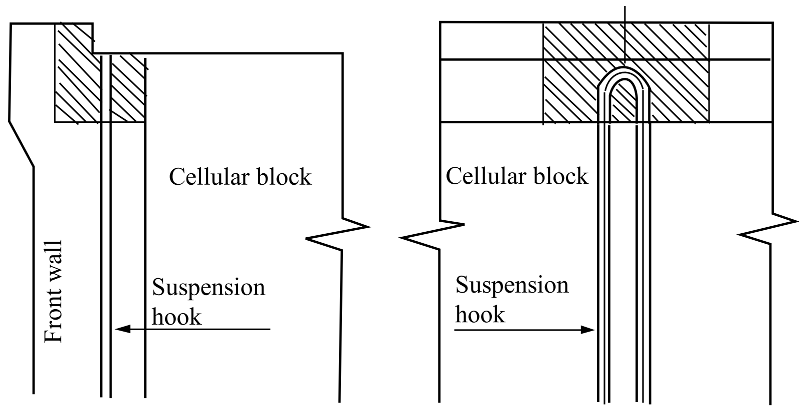


Fig. 2.4.10 Example of Recessing the Surface in which a Suspension Hook is Embedded

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.

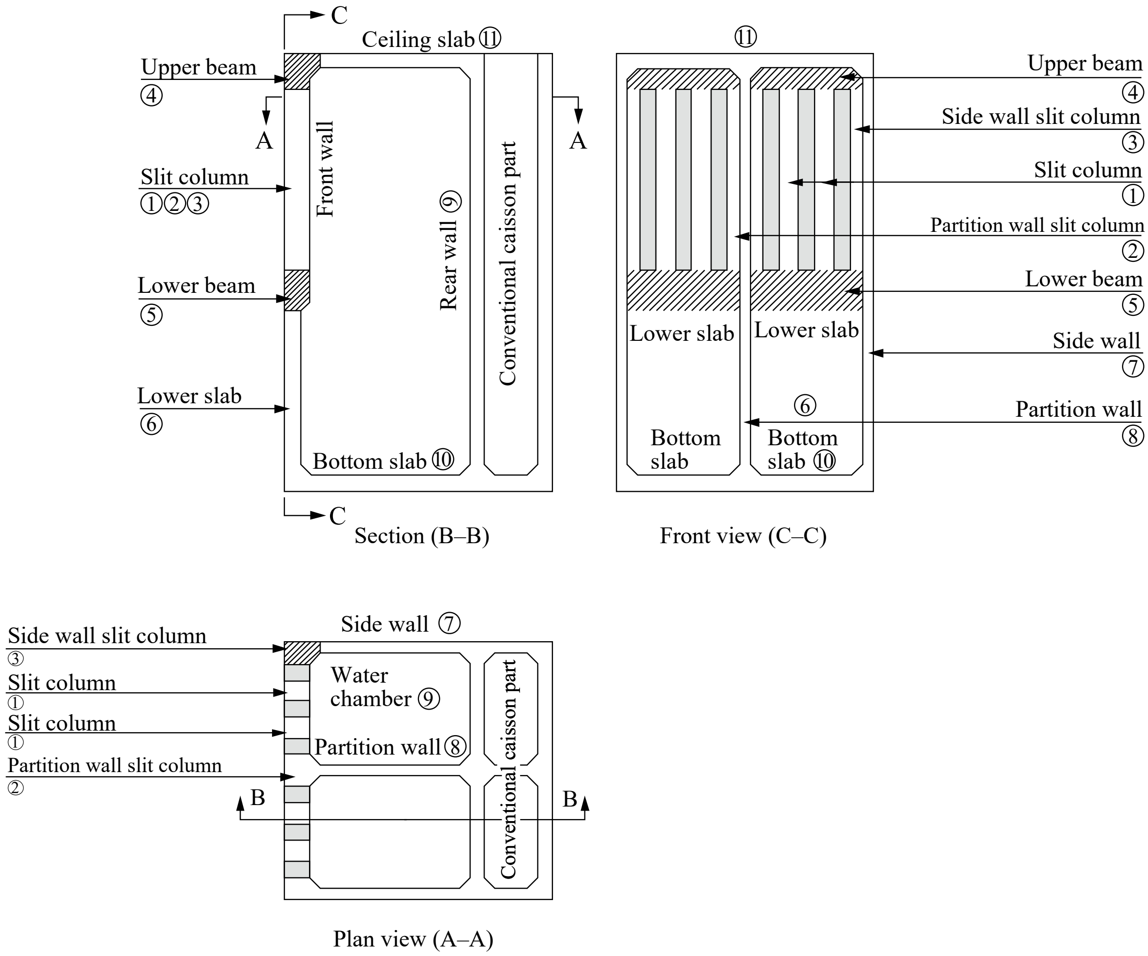


Fig. 2.5.1 Names of the Members of a Vertical Slit-Wall Caisson

(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)].

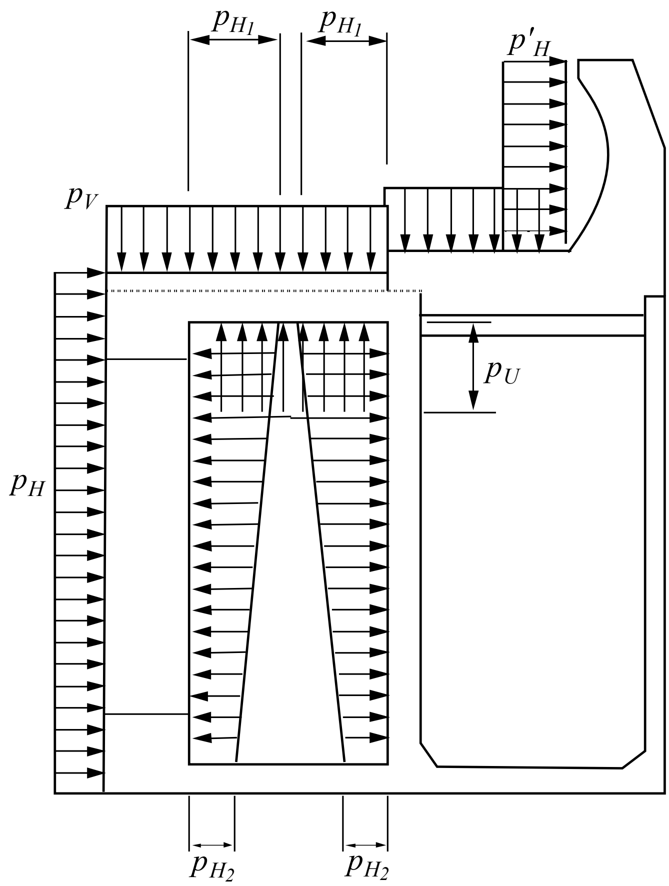


Fig. 2.5.2 Example of the Distributions of Wave Forces Acting on Members

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

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.

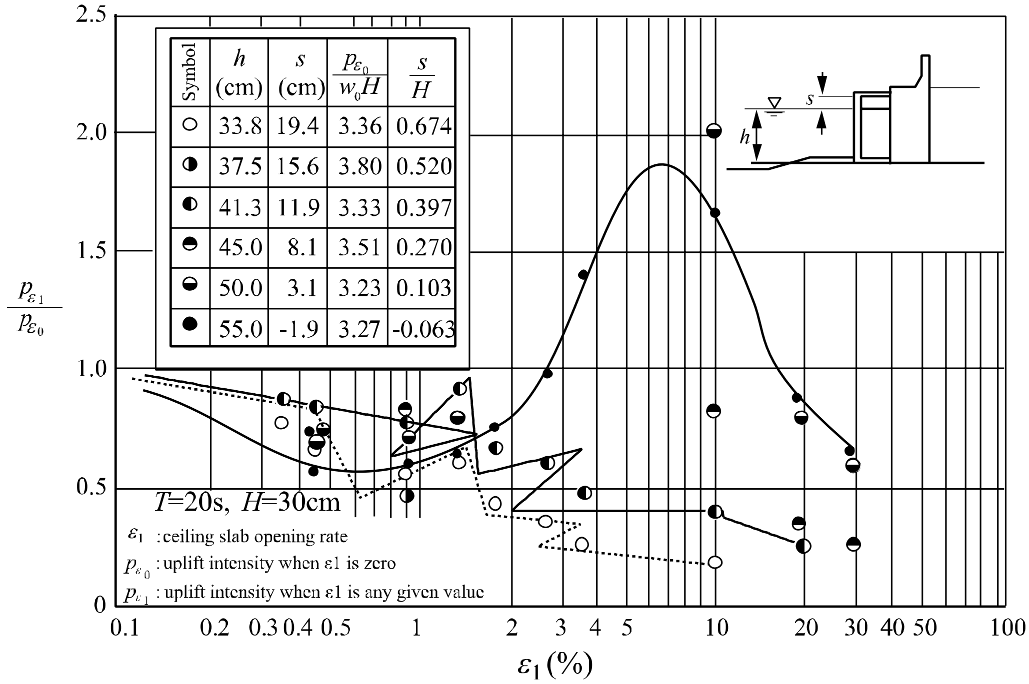


Fig. 2.5.3 Example of an Experiment Indicating the Changes in Uplift Intensity Depending on Ceiling Slab Opening Rate[17)]

(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

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

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]**)

$$P_a = p_a' \cdot l \tag{2.5.2}$$

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

$$P_{H_1} = p_{H_1}' B_1 \tag{2.5.3}$$

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

$$P_{H_2} = p_{H_2}' B_2 \tag{2.5.4}$$

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.

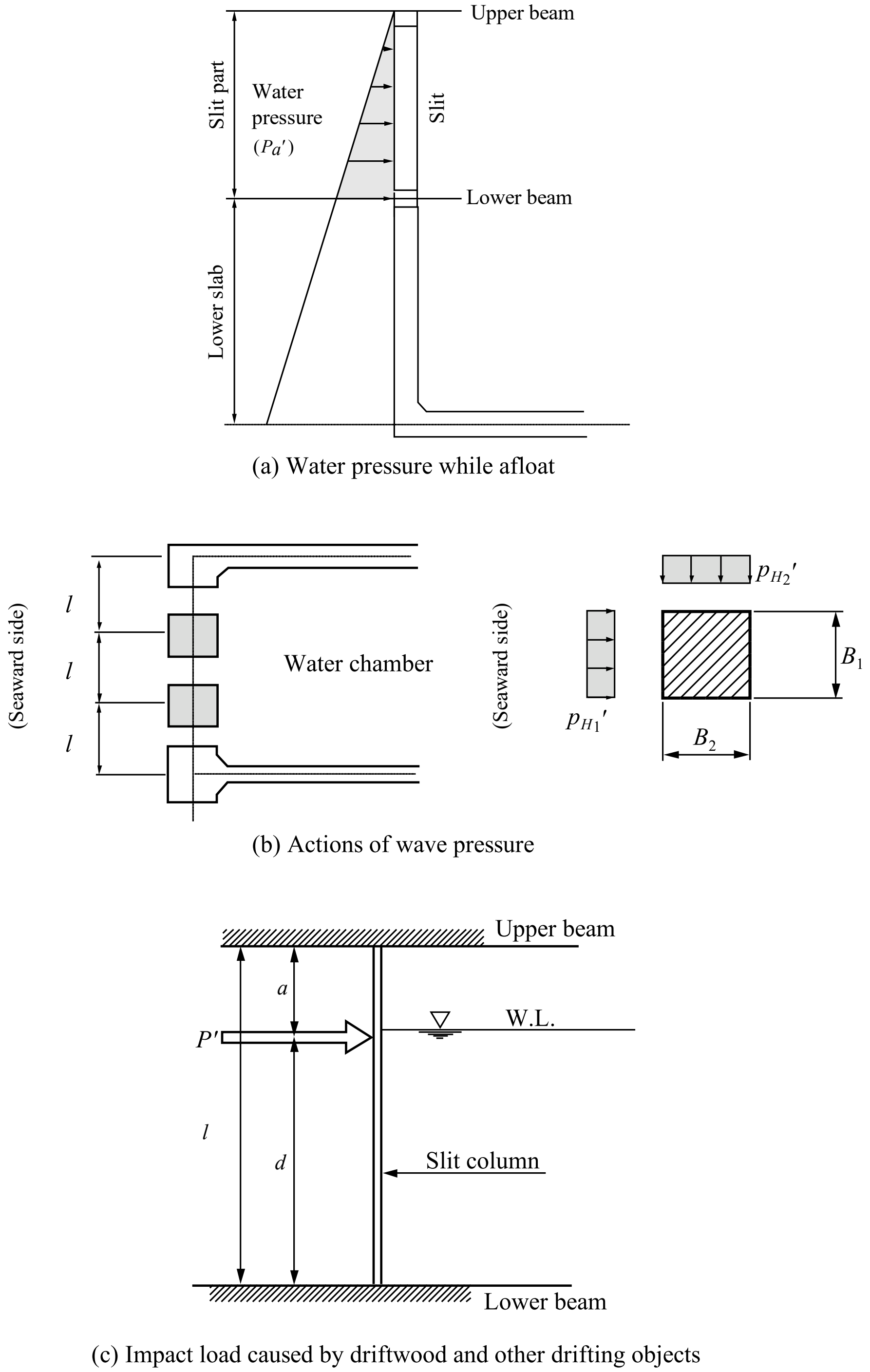


Fig. 2.5.4 Actions on Slit Columns

(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**.

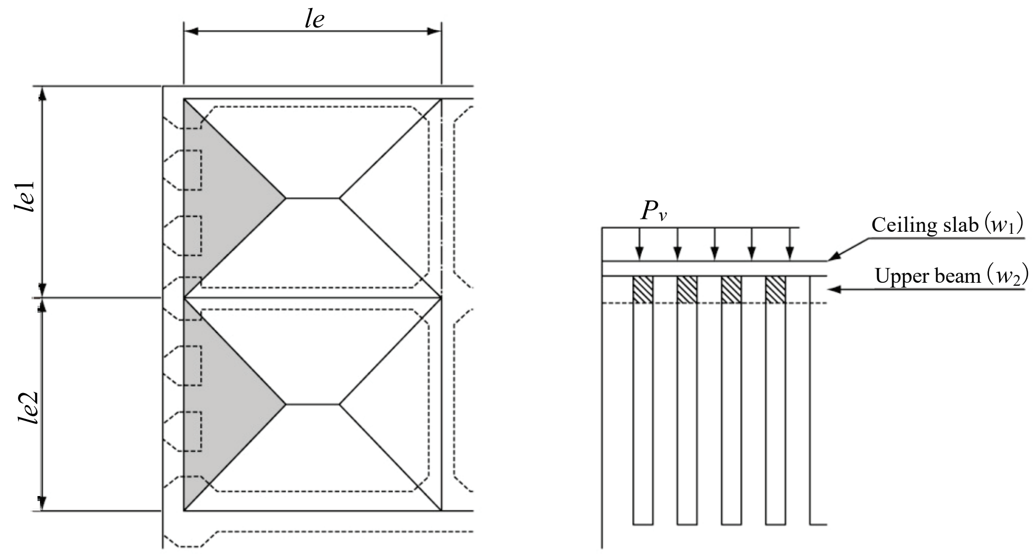


Fig. 2.5.5 Axial Actions on Slit Columns

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**.

$$P_p = p_a\left(b + l_0/2\right) \qquad \textbf{(2.5.9)}$$

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).

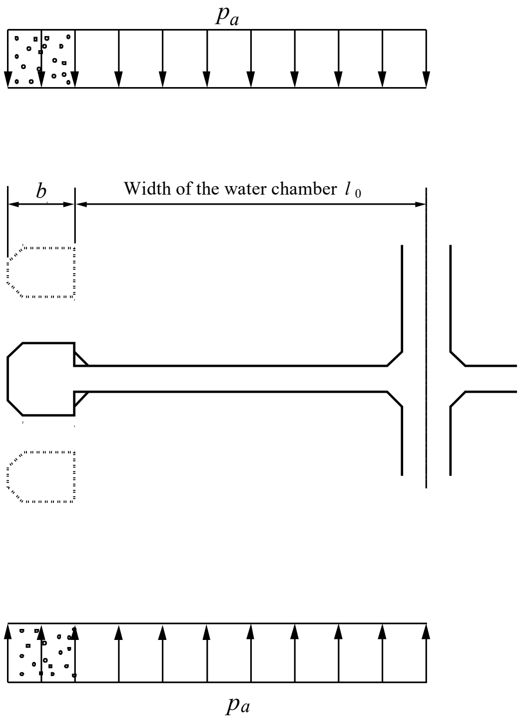


Fig. 2.5.6 Wave Pressure Acting on Partition Wall Slit Column

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**.

$$P_p = p_a\left(b + l_0/2\right) \tag{2.5.10}$$

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).

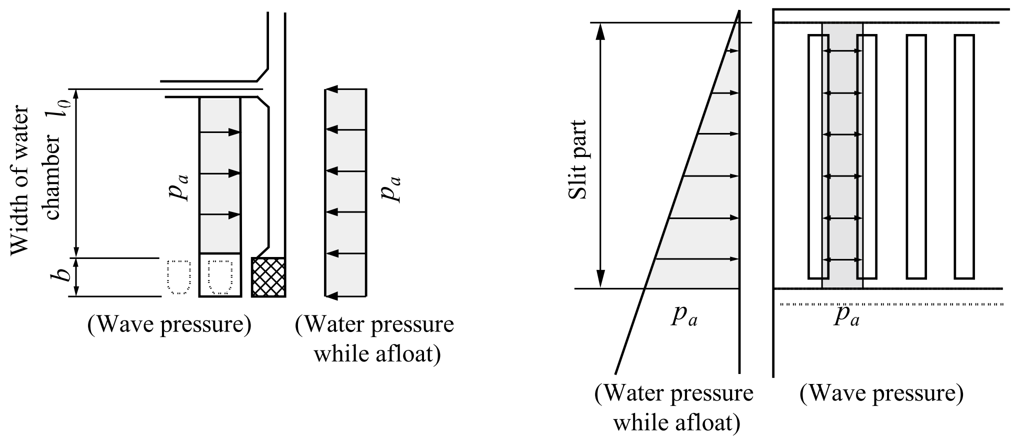


Fig. 2.5.7 Water Pressure and Wave Pressure Acting on the Side Wall Slit Column

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.

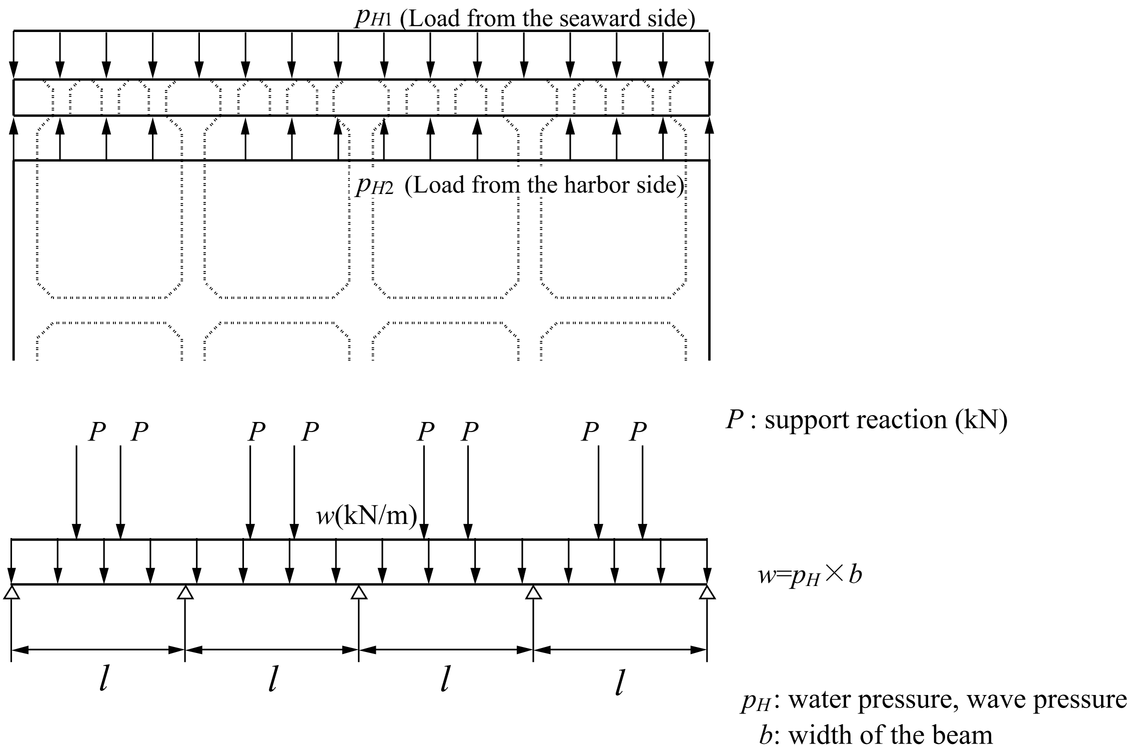


Fig. 2.5.8 Actions on the Upper Beam

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.

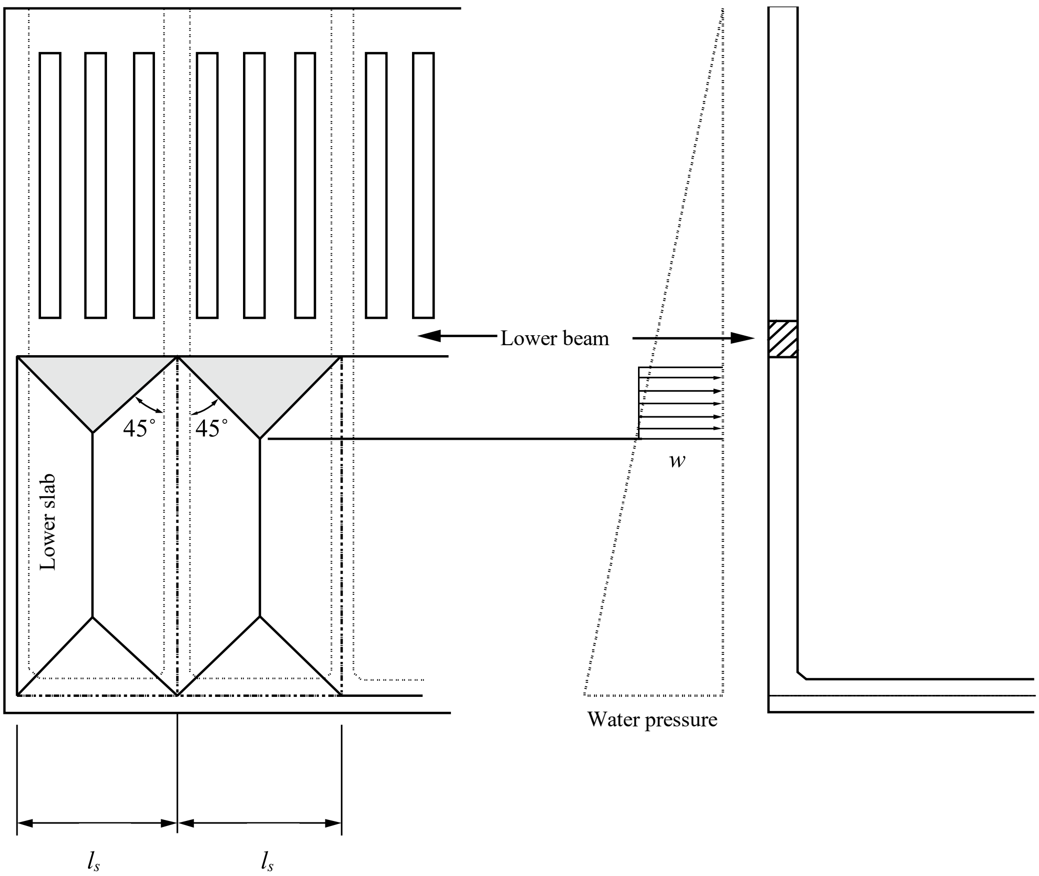


Fig. 2.5.9 Actions on the Lower Beam

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.

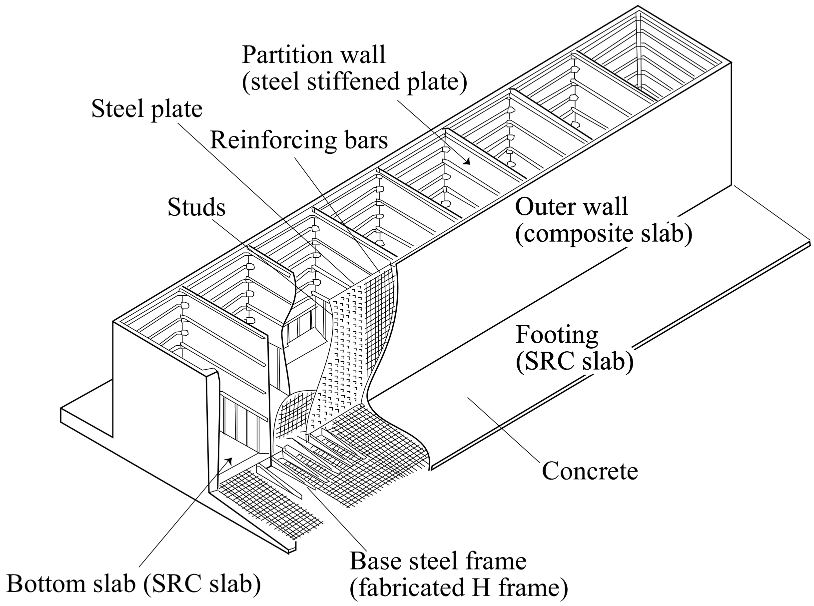


Fig. 2.6.1 Example of a Hybrid Caisson Structure

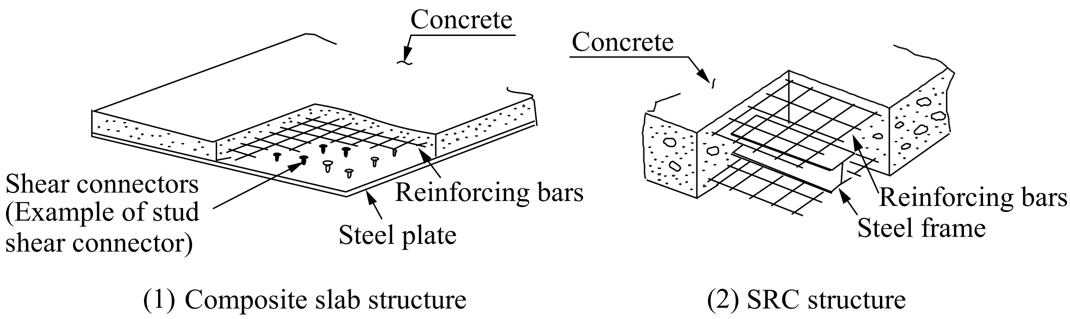


Fig. 2.6.2 Hybrid Caisson Structural Members

(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**.

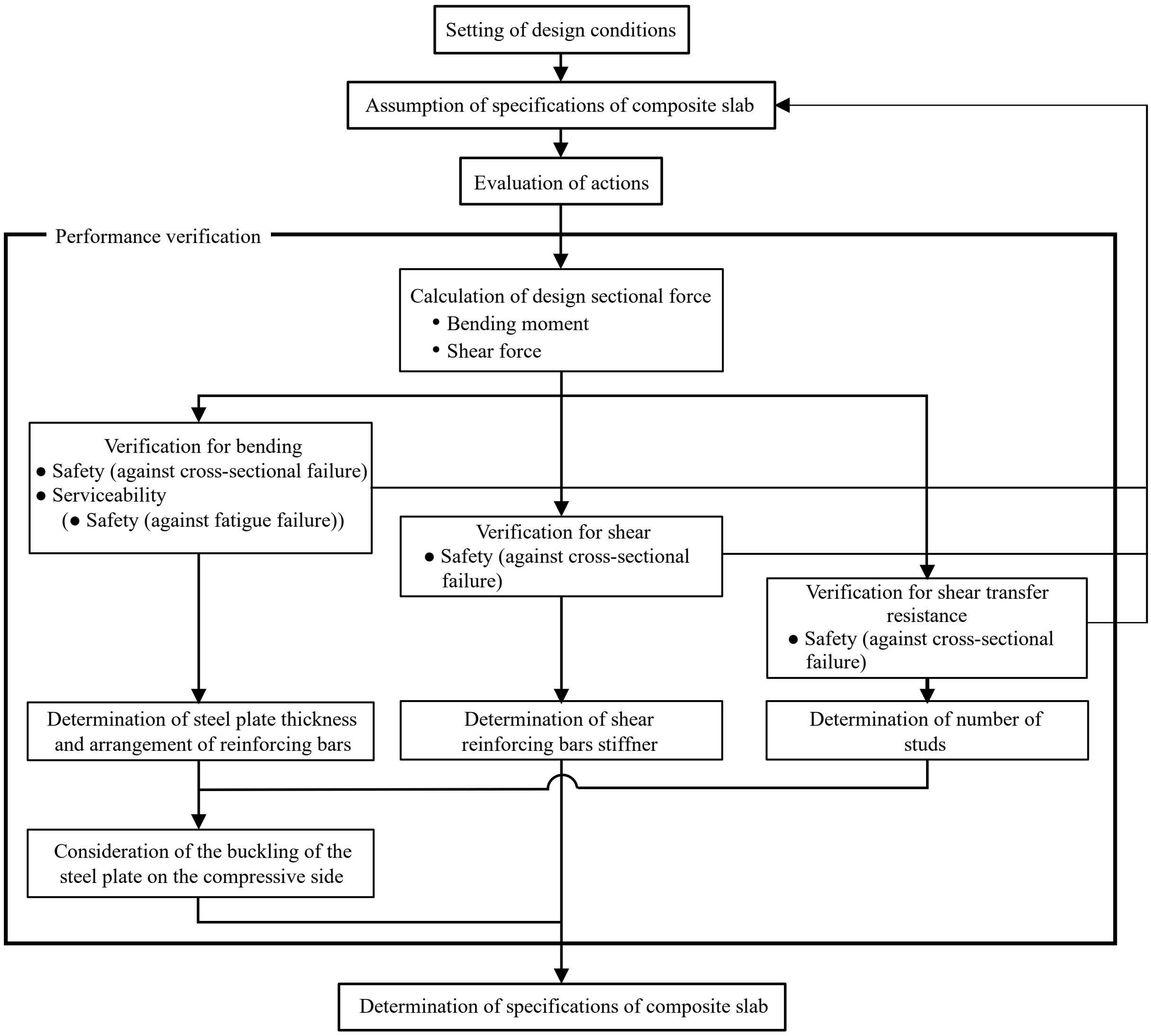


Fig. 2.6.3 Example of the Performance Verification of the Composite Slab of Hybrid Caisson

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 (=$2h/3$).

[References]

1) Uno, K., E. Kato and Y. Kawabata: Fundamental study on rational design of structural member of caisson in breakwater, Technical Note of PARI No.1329, 2016
2) Ishimoto, K.: Conversion plan of concrete structure in port -As example case of breakwater caisson-, Concrete journal 54(1), pp.93-98, 2016
3) Nagao, T: Reliability based design method for flexural design of caisson type breakwaters, Jour. JSCE No. 696/I-58, pp,173-184, 2002
4) Nagao, T.: Case Studies on Safety Factors about Seismic Stability for the Slob of Caisson Type Quaywalls, Technical Note of PHRI No.867, 1997
5) Moriya, Y., M. Miyata and T. Nagao: Design method for bottom slab of caisson considering surface roughness of rubble mound, Technical Note of National Institute for Land and Infrastructure Management No. 94, 2003
6) Nagao T., M. Miyata, Y. Moriya and T. Sugano: A method for designing caisson bottom slabs considering mound unevenness. Jour. JSCE C, Vol. 62, No.2, pp. 277-291, 2006
7) Kikuchi, Y., K. Takahashi and T. Ogura: Dispersion of Earth Pressure in Experiments and Earth Pressure Change due to the Relative Movement of the Neighboring Walls, Technical Note of PHRI No. 811, 1995
8) Tanimoto, K., K. Kobune and M. Osato: Wave Forces on a Caisson Wall and Stress Analysis of the Wall. for Prototype Breakwaters, Technical Note of PHRI No.224, pp. 25-33, 1975
9) Shiomi, M., H. Yamamoto, A. Tsugawa, T. Kurosawa and K. Matsumoto: Damages and countermeasures of breakwaters due to the wave force increase at discontinuous points of wave-absorbing blocks, Proceedings of the 41st conference on Coastal Eng. JSCE, pp.791-795, 1994
10) Miyata, M., Y. Moriya, T. Nagao and T. Sugano: Effects of surface roughness of rubble mound on section force of bottom slab of caisson, (Part 2), Technical Note of National Institute for Land and Infrastructure Management No.93, 2003
11) Yokota, H., K. Fukushima, T. Akimoto and M. Iwanami: Examination for Rationalizing Structural Design of Reinforced Concrete Caisson Structures, Technical Note of PHRI No. 995, 2001
12) Kawabata, Y., E. Kato and M. Iwanami: A study on the design method of RC caissons for breakwaters against impact loads considering maintenance strategy, Technical Note of PARI No.1279, 2013
13) Nishibori, T. and T. Urae: Dynamic characteristics of metal fitting for hanging of large caisson, Proceedings of 29th Conference of JSCE, 1974
14) Coastal Development Institute of Technology: Technical Manual for L-shape block wharves, 2006
15) Takahashi, S., K. Shimosako and H. Sasaki: Experimental Study on Wave Forces Acting on Perforated Wall Caisson Breakwaters, Rept. of PHRI Vol. 30 No. 4, pp. 3-34, 1991
16) Takahashi, S. and K. Tanimoto: Uplift Forces on a Ceiling Slab of Wave Dissipating Caisson with a Permeable Front Wall (2nd Report)-Field Data Analysis-, Rept. of PHRI Vol. 23 No. 2, 1984
17) Tanimoto, K., S. Takahashi and T. Murakami: Uplift Forces on a Ceiling Slab of Wave Dissipating Caisson with a Permeable Front Wall- Analytical Model for Compression of an Enclosed Air Layer-, Rept. of PHRI Vol. 19 No. 1, pp.3-31, 1980
18) Coastal Development Institute of Technology: Design Manual for Hybrid caisson, 1999
19) Yokota, H.: Study on Mechanical Properties of Steel-Concrete Composite Structures and Their Applicability to Marine Structures, Technical Note of PHRI No.750, 1993
20) JSCE: Standard Specification for Hybrid Structures, 2014
21) Coastal Development Institute of Technology: Design Manual for Composite caisson, 1991

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.

$$q_{\mathrm{d}} = \frac{1}{m_{\mathrm{B}}}\left(\beta \rho_{1\mathrm{k}} g \frac{B}{2} N_{\gamma\mathrm{k}} + \rho_{2\mathrm{k}} g D\left(N_{q\mathrm{k}} - 1\right)\right) + \rho_{2\mathrm{k}} g D \qquad \textbf{(3.2.1)}$$

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.

$$q_k = \beta \rho_{1k} g \frac{B}{2} N_{\gamma k} + \rho_{2k} g D N_{qk} \qquad \textbf{(3.2.2)}$$

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:

$$N_{q_k} = \frac{1+\sin\phi_k}{1-\sin\phi_k}\exp(\pi\tan\phi_k) \quad \text{(Solution of Prandtl)}$$

$$N_{\gamma_k} = (N_{q_k} - 1)\tan(1.4\phi_k) \quad \text{(Solution of Meyerhof)}$$

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)**.

$$q_d - \rho_{2k} g D = \frac{1}{m_B}\left(\beta \rho_{1k} g \frac{B}{2} N_{\gamma k} + \rho_{2k} g D (N_{qk} - 1)\right) \tag{3.2.3}$$

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

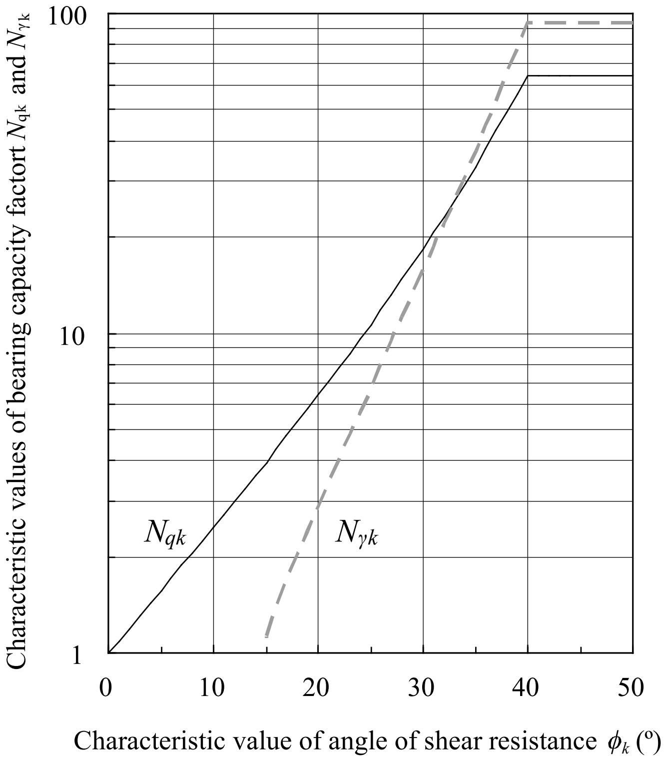


Fig. 3.2.1 Relationship between Bearing Capacity Coefficients N_γ and N_q and Angle of Shear Resistance ϕ

(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 $\tan\phi$ when local shear failure is expected, which can be adopted.

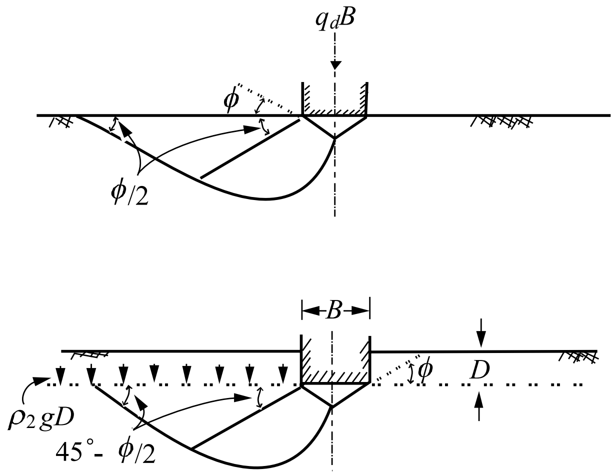


Fig. 3.2.2 Plastic Equilibrium Condition in the Ground below Continuous Foundation

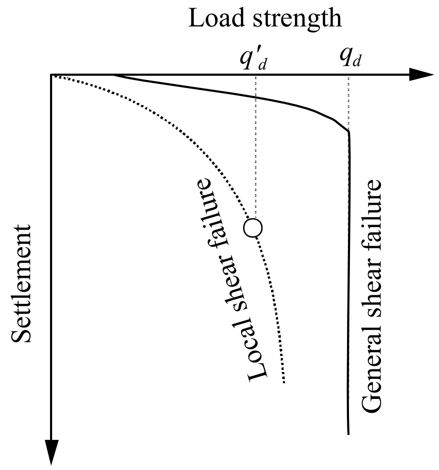


Fig. 3.2.3 Relation between Load Strength and Settlement in Loading Test

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.

$$q_{\mathrm{d}} = \frac{1}{m_{\mathrm{B}}} N_{c0\mathrm{k}} \left(1 + n\frac{B}{L} \right) c_{0k} + \rho_{2\mathrm{k}} gD \tag{3.2.4}$$

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.

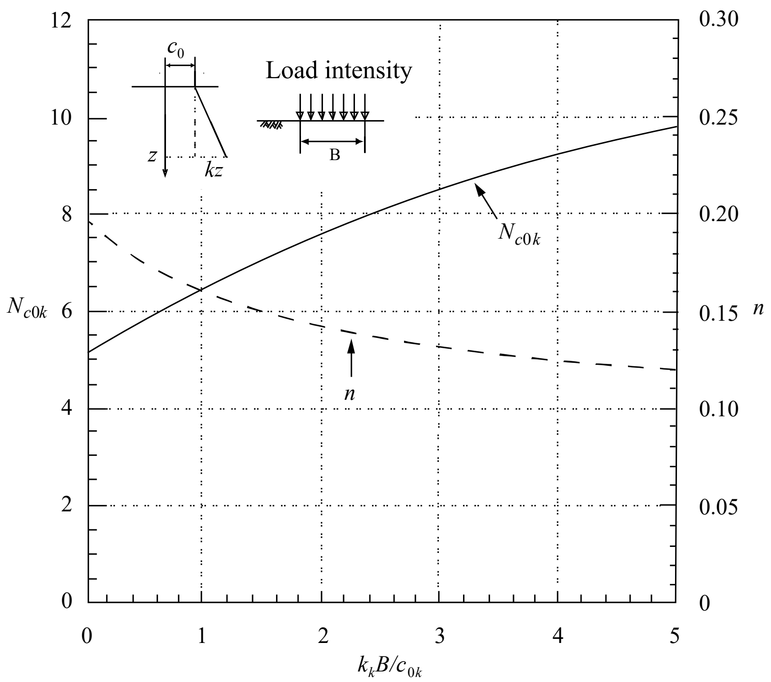


Fig. 3.2.4 Bearing Capacity Coefficient N_{c0k} of Clayey Ground in which Strength Increases in Depth Direction and Shape Factor n

(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.

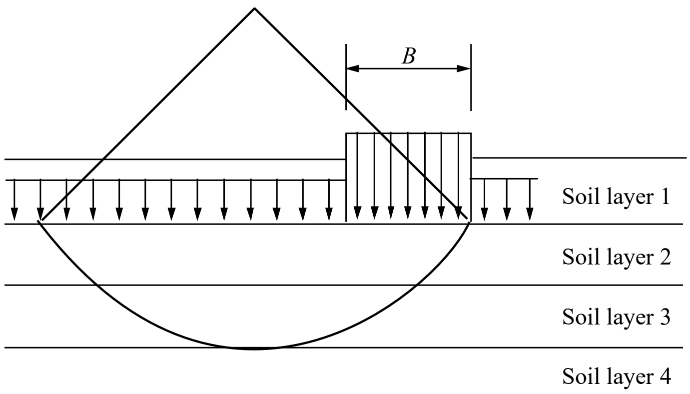


Fig. 3.2.5 Calculation of Bearing Capacity of Multi-layered Ground by Circular Slip Failure Analysis

(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/m^2)

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/m^2)

$\rho_{2k}g$: 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.

$$\frac{m}{\gamma_R}\cdot\frac{\gamma_S\cdot\sum\{(W_k+q_k)\sin\theta+aP_{Hk}/R\}}{\sum\left[\{c_k s+(W'_k+q_k)\tan\phi_k\}\dfrac{\sec\theta}{1+\tan\theta\tan\phi_k/(m/\gamma_R)}\right]}\le 1 \qquad \textbf{(3.2.7)}$$

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:

$$m\cdot\frac{S_d}{R_d}\le 1.0 \qquad S_d=\gamma_S S_k \qquad R_d=\gamma_R R_k \qquad \textbf{(3.2.8)}$$

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.

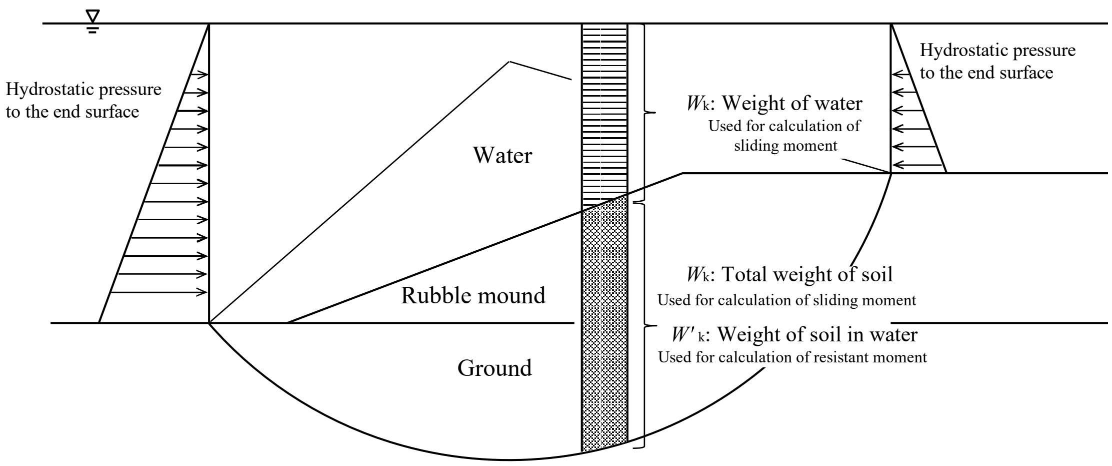


Fig. 3.2.6 Divided Segment in Circular Slip Failure Analysis and Concept of Its Weight

(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; $q=\dfrac{(p_1+p_2)}{4b'}B$

When subgrade reaction has a triangular distribution; $q=\dfrac{p_1 b}{4b'}$

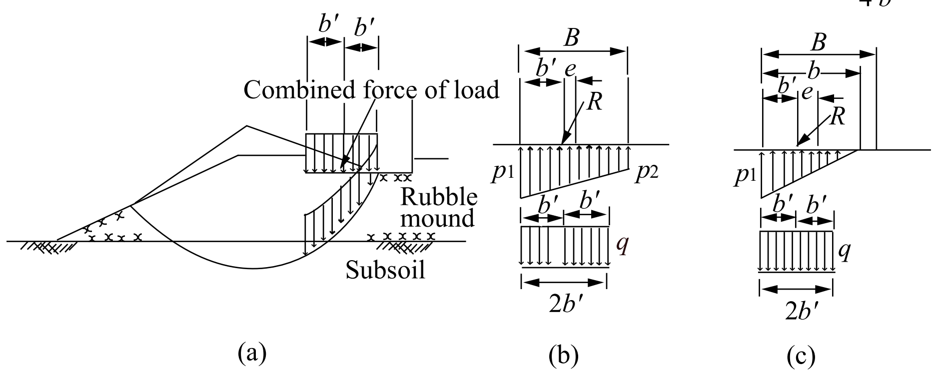


Fig. 3.2.7 Analysis of Bearing Capacity for Eccentric and Inclined Actions

(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 = 35° 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= 35°. 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 = 30° [7)].

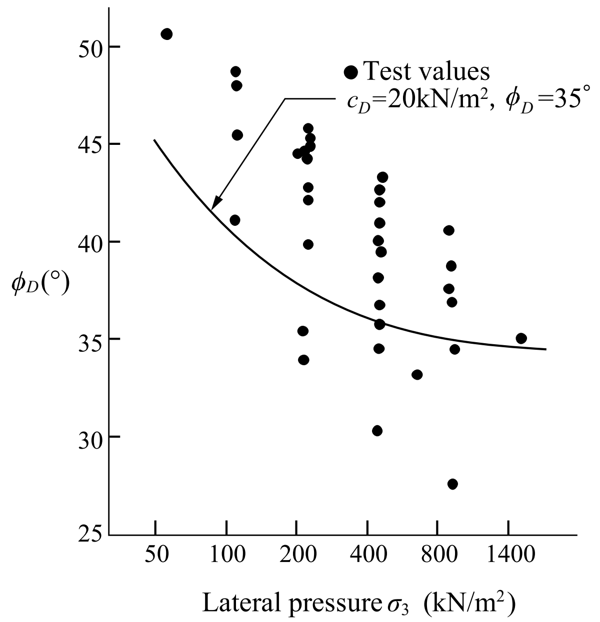


Fig. 3.2.8 Relationship between ϕ_D and Lateral Confining Pressure σ_3 and Apparent Cohesion

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**.

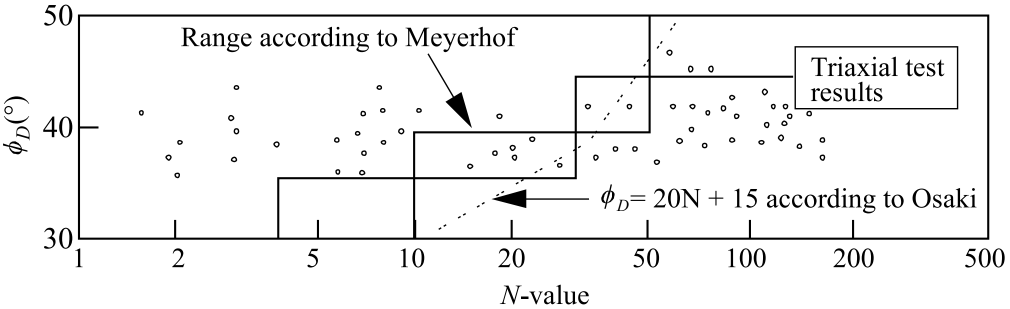


Fig.3.2.9 Relationship between ϕ_D and the SPT-N value Obtained by Triaxial Tests on Undisturbed Sand Samples

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**.

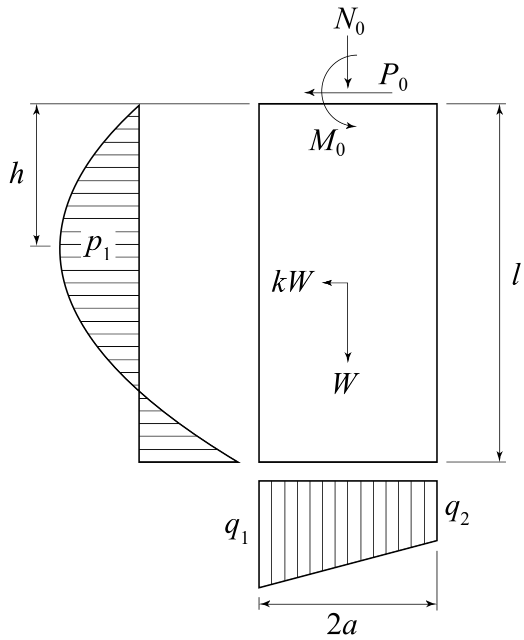


Fig. 3.3.1 Distribution of the Subgrade Reaction When the Action Position of the Resultant Force of Loads Is Inside the Core

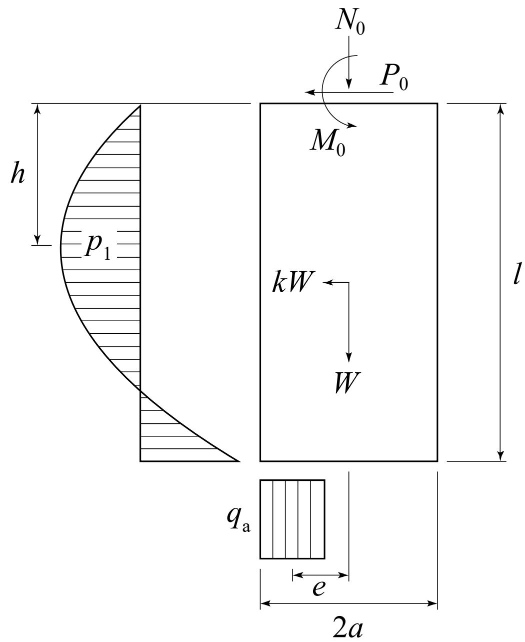


Fig. 3.3.2 Distribution of the Subgrade Reaction When the Action Position of the Resultant Force of Loads Is Outside of the Core

(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.

$$\frac{N_0 + w_1 l}{A} \geq \frac{3aK'\left(kw_1 l^2 + 4P_0 l + 6M_0\right)}{b\left(l^3 + 24\alpha K' a^3\right)} \quad \textbf{(3.3.2)}$$

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.

$$q_1 = \frac{N_0 + w_1 l}{A} + \frac{3aK'\left(kw_1 l^2 + 4P_0 l + 6M_0\right)}{b\left(l^3 + 24\alpha K' a^3\right)} \quad \textbf{(3.3.3)}$$

$$p_1 = \frac{3\left\{kw_1 l^4 + 3P_0 l^3 + 4M_0 l^2 + 8\alpha K' a^3\left(kw_1 l + P_0\right)\right\}^2}{4bl^3\left(l^3 + 24\alpha K' a^3\right)\left(kw_1 l^2 + 4P_0 l + 6M_0\right)} \quad \textbf{(3.3.4)}$$

$$h = \frac{kw_1 l^4 + 3P_0 l^3 + 4M_0 l^2 + 8\alpha K' a^3\left(kw_1 l + P_0\right)}{2l\left(kw_1 l^2 + 4P_0 l + 6M_0\right)} \quad \textbf{(3.3.5)}$$

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.

$$p_1 = \frac{3\left(kWl + 4M_0 - 4N_0 e - 4We + 3P_0 l\right)^2}{4bl^2\left(kWl + 6M_0 - 6N_0 e - 6We + 4P_0 l\right)} \tag{3.3.6}$$

$$h = \frac{l\left(kWl + 4M_0 - 4N_0 e - 4We + 3P_0 l\right)}{2\left(kWl + 6M_0 - 6N_0 e - 6We + 4P_0 l\right)} \tag{3.3.7}$$

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)].

$$2a = \frac{3}{4}D \tag{3.3.8}$$

$$2b = \frac{\pi}{3}D \tag{3.3.9}$$

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.

$$m\frac{\gamma_S p_1}{\gamma_R p_{pk}} \leq 1.0 \tag{3.3.10}$$

$$q_{ad} \geq q_1 \tag{3.3.11}$$

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)**.

$$q_{u2d} = \frac{2}{m_B}\left(1+\frac{B}{L}\right)\frac{D_c}{B}\bar{c}_{ak} \tag{3.3.14}$$

where

D_c : embedment depth of foundation below groundwater level (m)

$\bar{c}_{ak}$: 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.

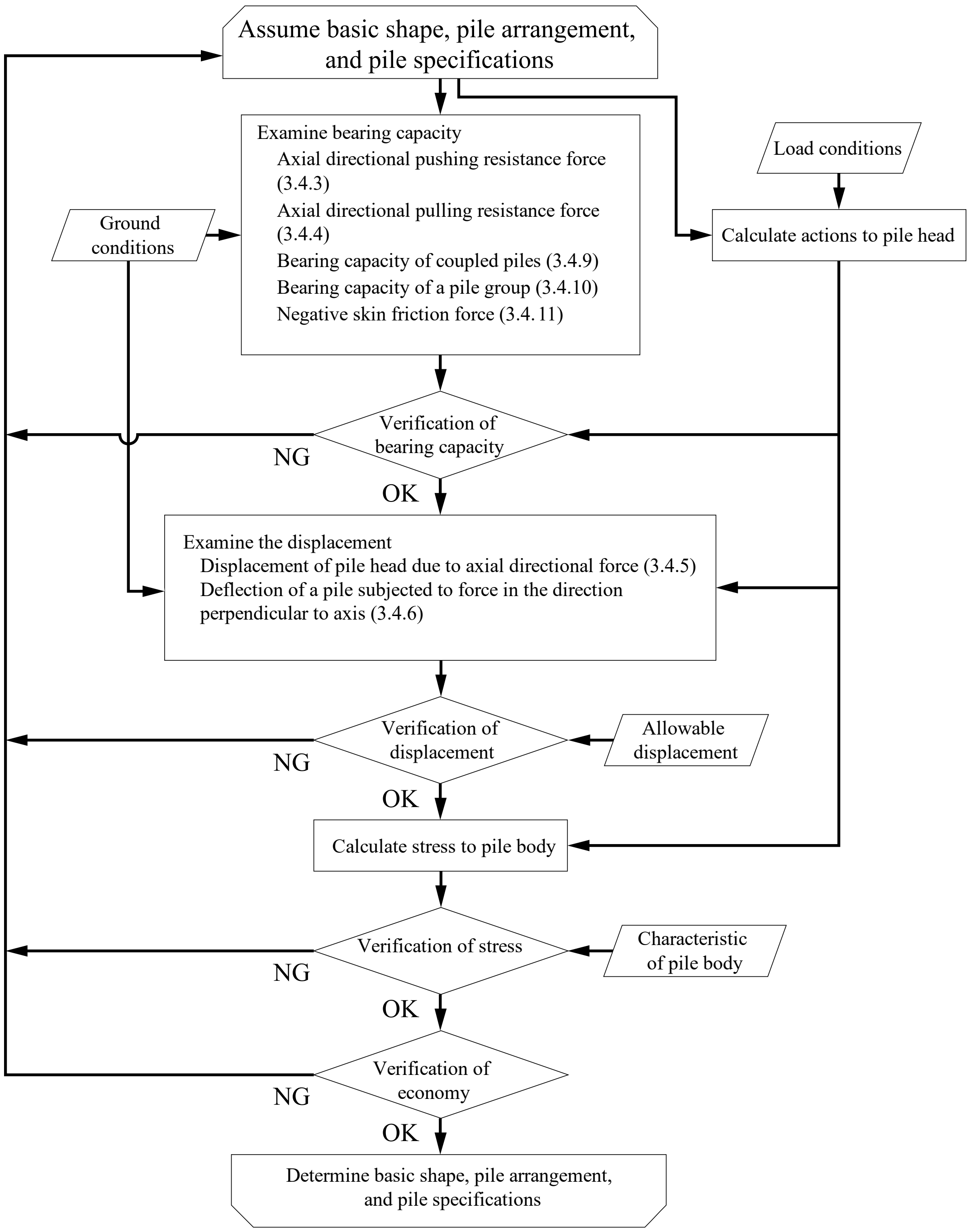


Fig. 3.4.1 Example of Performance Verification Sequence of a Pile Foundation

(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.

$$R_{tk} = R_{pk} + R_{fk} \qquad \textbf{(3.4.1)}$$

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.

$$R_{fk} = \sum_i \overline{r_f}_{ki} A_{si} \qquad \textbf{(3.4.2)}$$

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)**.

$$R_{pk} = 300 N A_p \tag{3.4.3}$$

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)**.

$$R_{pk} = 6c_p A_p \tag{3.4.4}$$

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**.

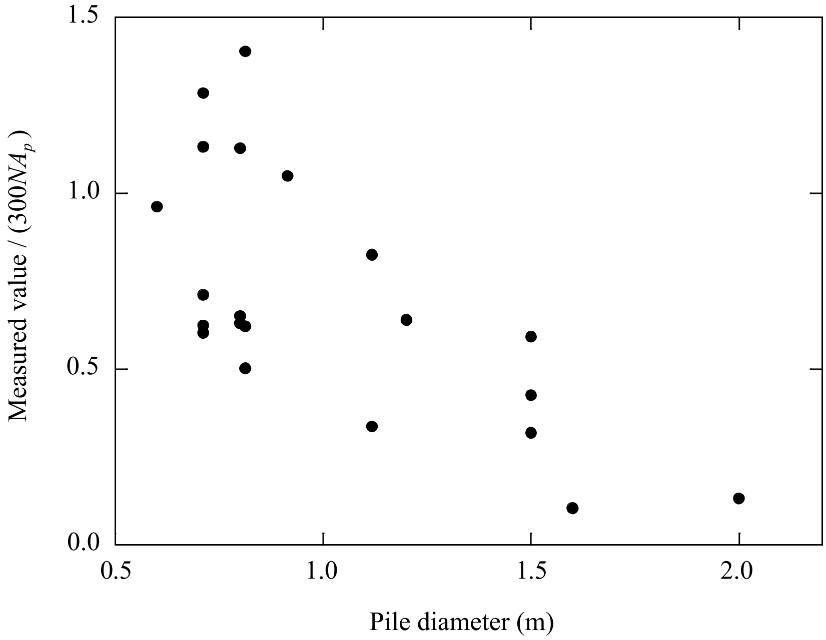


Fig. 3.4.2 Relation between Diameter of Open-ended Piles and Plugging Ratio (Kikuchi et al. [16], added and altered)

(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.

$$\alpha = \begin{cases} 0 & \dfrac{l}{d} \leq 120 \\ \dfrac{l}{2d} - 60 & \dfrac{l}{d} > 120 \end{cases} \tag{3.4.9}$$

$$\alpha = \begin{cases} 0 & \dfrac{l}{d} \leq 60 \\ \dfrac{l}{d} - 60 & \dfrac{l}{d} > 60 \end{cases} \tag{3.4.10}$$

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)**.

$$S_0 = S_p + S_S = S_{PE} + S_{PP} + S_{SE} + S_{SP} \quad \textbf{(3.4.11)}$$

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.

$$S_{0E} = S_{PE} + S_{SE} - S_f \quad \textbf{(3.4.12)}$$

where

S_{0E} : elastic return of a pile head (m)

S_f : restraint of elastic return due to skin friction (m)

$$S_{0P} = S_{PP} + S_{SP} + S_f \quad \textbf{(3.4.13)}$$

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.

$$p = \frac{E_s}{B}\, y = k_{CH}\, y \qquad \textbf{(3.4.15)}$$

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.

$$\left.\begin{array}{ll} \text{Above the ground level} & EI\,\dfrac{d^4 y_z}{dz^4} = 0 \\ \text{Below the ground level} & EI\,\dfrac{d^4 y_x}{dx^4} + Bk_{CH} y_x = 0 \end{array}\right\} \qquad \textbf{(3.4.16)}$$

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**).

$$\left.\begin{aligned} y_z &= y_t - \theta_t z + \frac{M_t}{2EI} z^2 + \frac{H_t}{6EI} z^3 \\ y_x &= \frac{H_t}{2EI\beta^3} e^{-\beta x} \left\{ (1+\beta h_0) \cos \beta x - \beta h_0 \sin \beta x \right\} \end{aligned}\right\} \tag{3.4.18}$$

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)

$$y_t = \frac{2(1+\beta h)^3 + 1}{6EI\beta^3} H_t + \frac{(1+\beta h)^2}{2EI\beta^2} M_t$$

θ_t : inclination of a pile (rad)

$$\theta_t = \frac{(1+\beta h)^2}{2EI\beta^2} H_t + \frac{1+\beta h}{EI\beta} M_t$$

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)**).

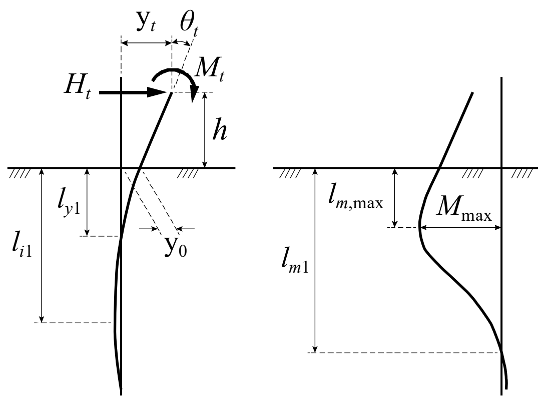


Fig. 3.4.3 Deflection Curve and Distribution of Bending Moment for Free Head Piles

(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**).

$$\left.\begin{aligned} y_z &= y_t - \frac{(1+\beta h)\,H_t}{4EI\beta} z^2 + \frac{H_t}{6EI} z^3 \\ y_x &= \frac{H_t}{4EI\beta^3} e^{-\beta x} \left\{(1+\beta h)\cos\beta x + (1-\beta h)\sin\beta x\right\} \end{aligned}\right\} \qquad \textbf{(3.4.26)}$$

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)**.

$$y_0 = \frac{1+\beta h}{4EI\beta^3} H_t \qquad \textbf{(3.4.27)}$$

The bending moment and shear caused in a pile body are expressed by **equations (3.4.28)** and **(3.4.29)**, respectively.

$$\left.\begin{aligned} M_z &= \frac{H_t}{2\beta}(1+\beta h - 2\beta z) \\ M_x &= \frac{H_t}{2\beta} e^{-\beta x} \left\{(1-\beta h)\cos\beta x - (1+\beta h)\sin\beta x\right\} \end{aligned}\right\} \qquad \textbf{(3.4.28)}$$

$$\left.\begin{aligned} S_z &= -H_t \\ S_x &= -H_t\, e^{-\beta x} (\cos\beta x - \beta h \sin\beta x) \end{aligned}\right\} \qquad \textbf{(3.4.29)}$$

The maximum bending moment caused in a pile body below the ground level is expressed by **equation (3.4.30)**.

$$M_{\max} = -\frac{H_t}{2\beta}\sqrt{1+(\beta h)^2}\; e^{-\beta l_{m,\max}} \qquad \textbf{(3.4.30)}$$

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)**).

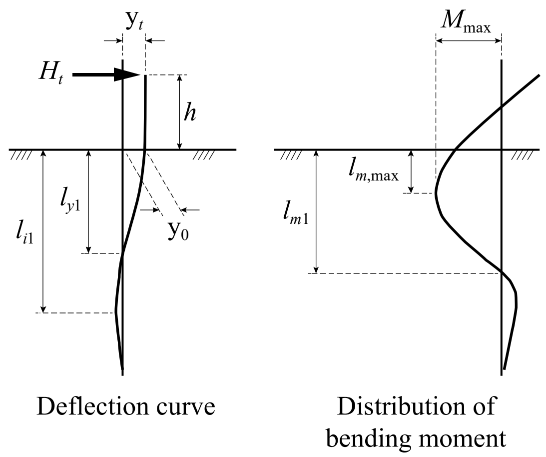


Fig. 3.4.4 Deflection Curve and Distribution of Bending Moment for Fixed Head Piles

(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.

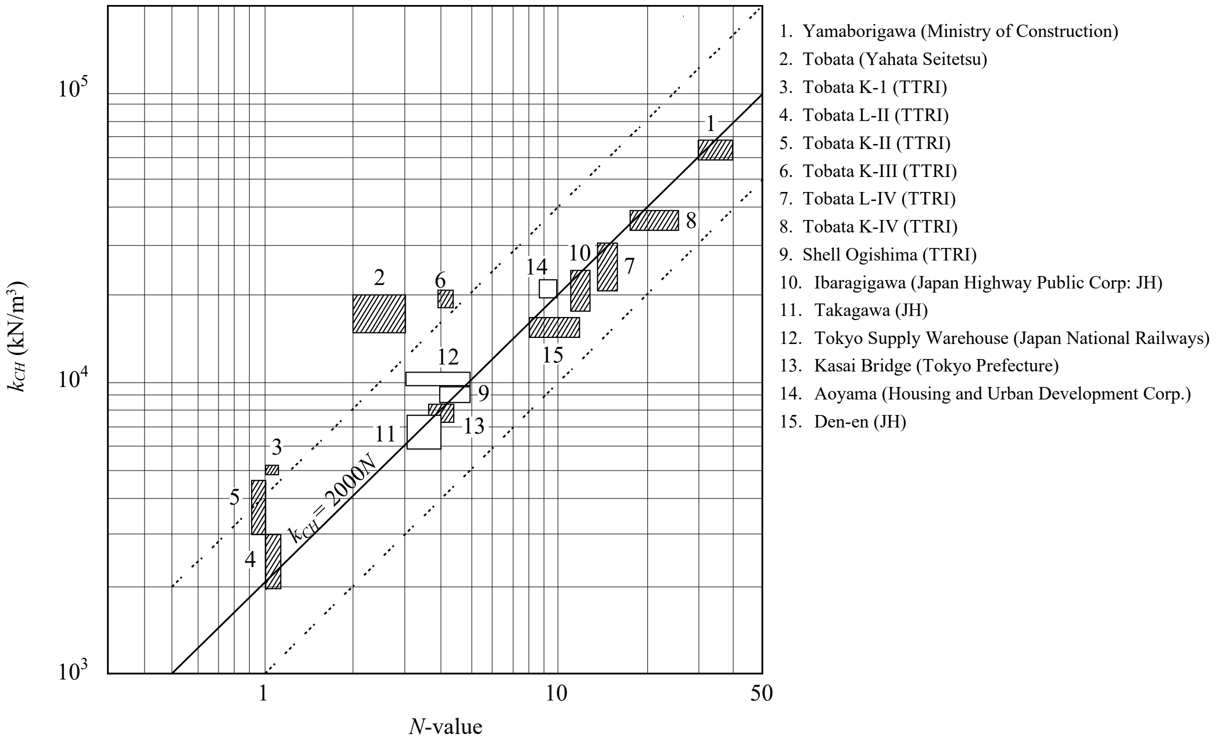


Fig. 3.4.5 Relation between the Coefficient of Lateral Subgrade Reaction Obtained from Horizontal Loading Test of a Pile and the SPT-N value of the Ground (Yokoyama [48]), added and altered)

(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]).

$$\left.\begin{array}{ll}\text{Free head pile, S-type ground} & k_{CH}=103\left(\dfrac{EI}{B}\right)^{0.207} y_0^{-0.398}\, h^{-0.035}\, \bar{N}^{0.519}\\ \text{Fixed head pile, S-type ground} & k_{CH}=114\left(\dfrac{EI}{B}\right)^{0.216} y_0^{-0.392}\, h^{-0.088}\, \bar{N}^{0.513}\\ \text{Free head pile, C-type ground} & k_{CH}=721\left(\dfrac{EI}{B}\right)^{-0.001} y_0^{-0.499}\, h^{0.009}\, N^{0.649}\\ \text{Fixed head pile, C-type ground} & k_{CH}=705\left(\dfrac{EI}{B}\right)^{-0.005} y_0^{-0.501}\, h^{0.028}\, N^{0.651}\end{array}\right\} \quad \textbf{(3.4.36)}$$

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.

$$p = k x^m y^{0.5} \tag{3.4.37}$$

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)**.

$$\left.\begin{array}{ll}\text{S-type ground} & p = k_s\, x\, y^{0.5} \\ \text{C-type ground} & p = k_c\, y^{0.5}\end{array}\right\} \tag{3.4.38}$$

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.

$$\log R_x = \log R_h \qquad \textbf{(3.4.42)}$$

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.

$$\log R_T = \log R_S \qquad \textbf{(3.4.43)}$$

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)

$\mathrm{M}_{\mathrm{max},S}$: 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

$\log T_s$	$\log y_{t,s}$	$\log M_{\mathrm{top},s}$	$\log l_{m1,s}$	$\log y_{0,s}$	$\log M_{\max,s}$	$\log i_{0,s}$
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})

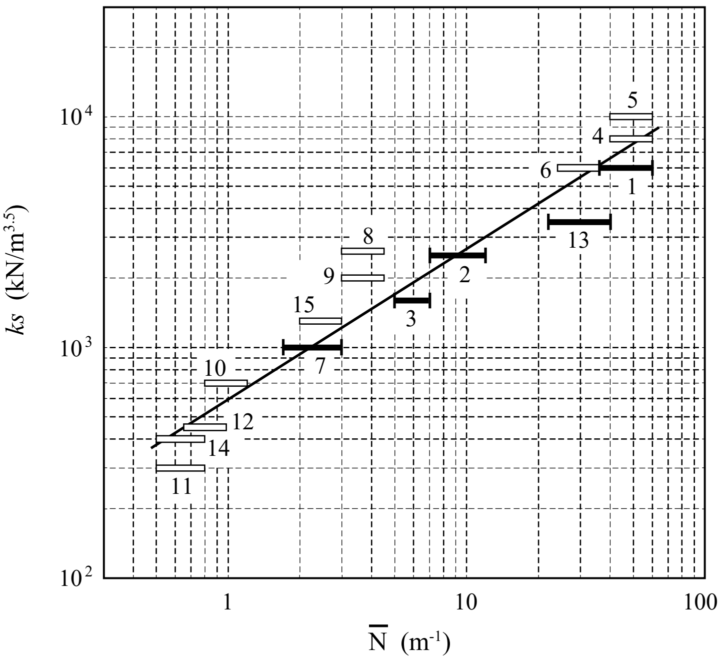


Fig. 3.4.6 Relation between lateral resistance coefficient in S-type ground and increment of SPT-N values $\bar{N}$ per Unit Depth (Kubo [54], added and altered)

(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.

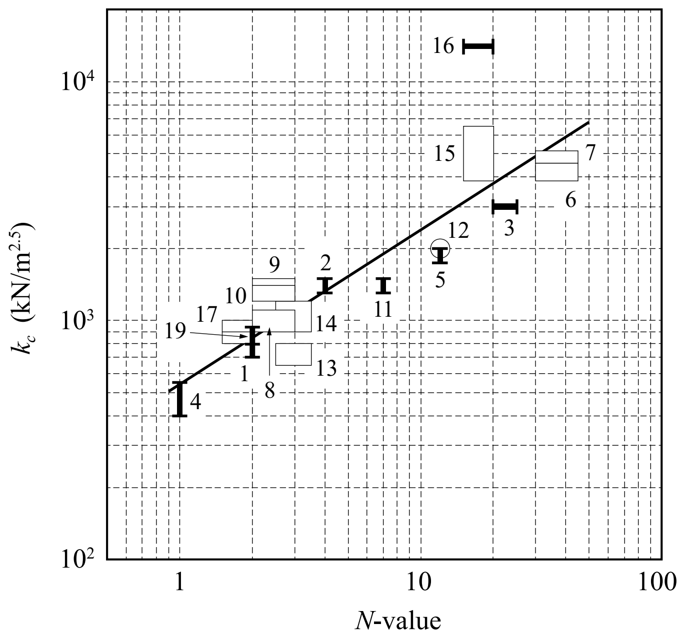
The line in **Fig. 3.4.7** is a regression line obtained by the least squares method and is expressed by **Equation (3.4.49)** [55].

$$k_C = 540\,N^{0.648} \qquad \textbf{(3.4.49)}$$

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 A_4
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]).

$$k_S = \alpha \sqrt{\frac{2\pi}{\varepsilon_{50} B}} \qquad \textbf{(3.4.50)}$$

$$k_C = c_u \sqrt{\frac{2\pi}{\varepsilon_{50} B}} \qquad \textbf{(3.4.51)}$$

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.

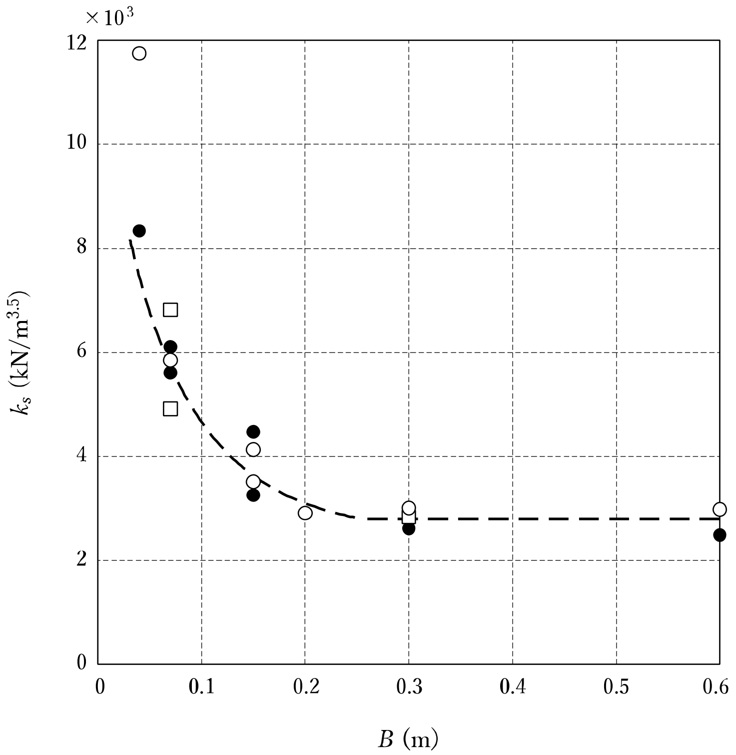


Fig. 3.4.8 Relation between lateral resistance coefficient in the S-type ground and the width of piles (Sawaguchi [60)], added and altered)

(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)**.

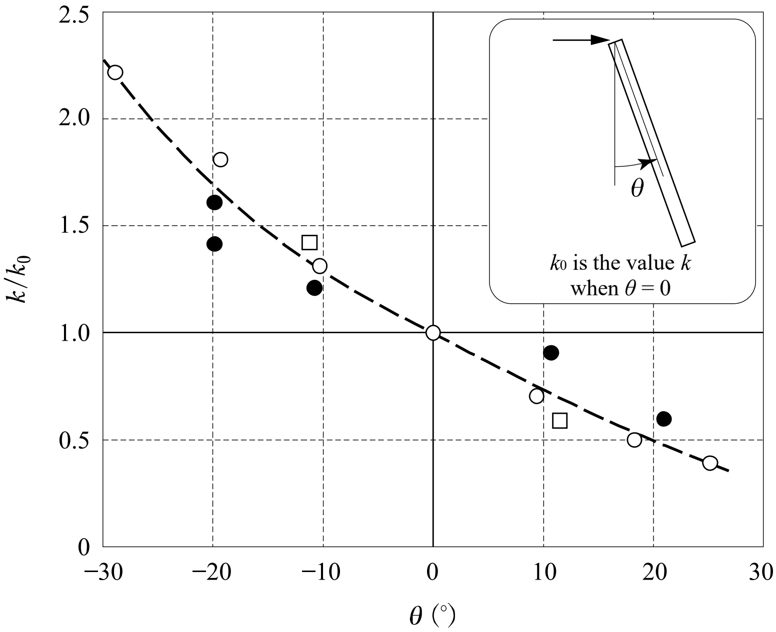


Fig. 3.4.9 Ratio of angle of pile's inclination to the lateral resistance coefficient of the ground (Kubo [61], added and altered)

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)

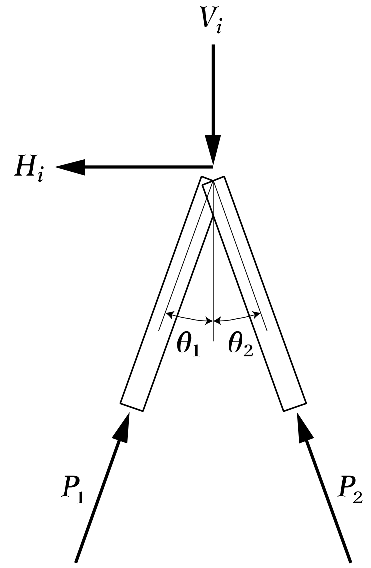


Fig. 3.4.10 Resolution of load acting on coupled piles into elements in the axial direction

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 Chapte**r**.

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.

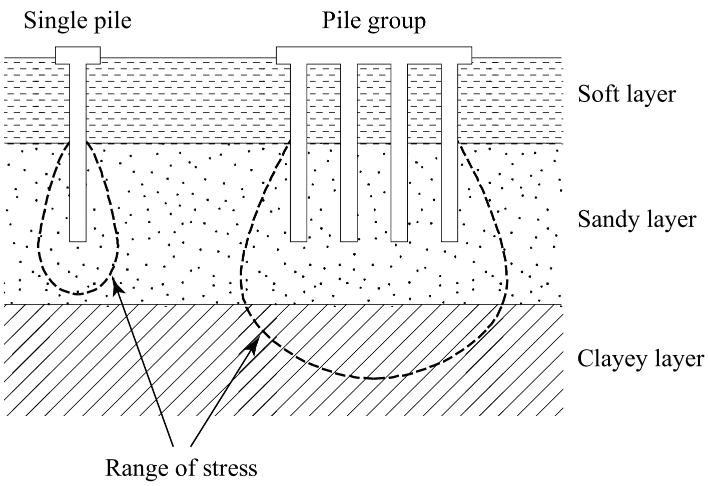


Fig. 3.4.11 Comparison of Ranges Where Stress in the Ground Is Caused in a Single Pile and a Pile Group

(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)**.

$$R_{gk} = q_{u1k} A_g + s_k U_g L - \gamma'_g A_g L \qquad \textbf{(3.4.53)}$$

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)** .

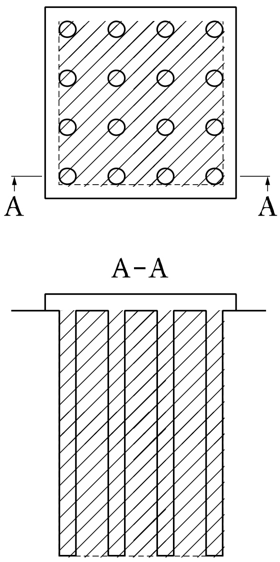


Fig. 3.4.12 A block Comprising a Group of Piles and Soil between the Piles (Shaded Part in the Figure)

(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)**.

$$R_{\text{pull},gk} = s_k U_g L + \gamma'_g A_g L \qquad \textbf{(3.4.54)}$$

where

$R_{\text{pull},gk}$: 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].

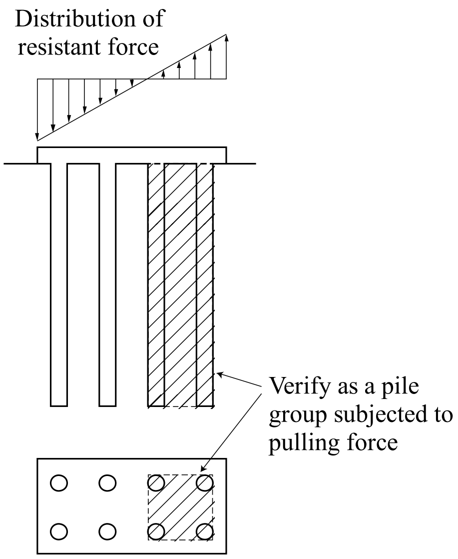


Fig. 3.4.13 Verification Method of Pulling Resistance Force on Foundation Subjected to Eccentric Moment

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.

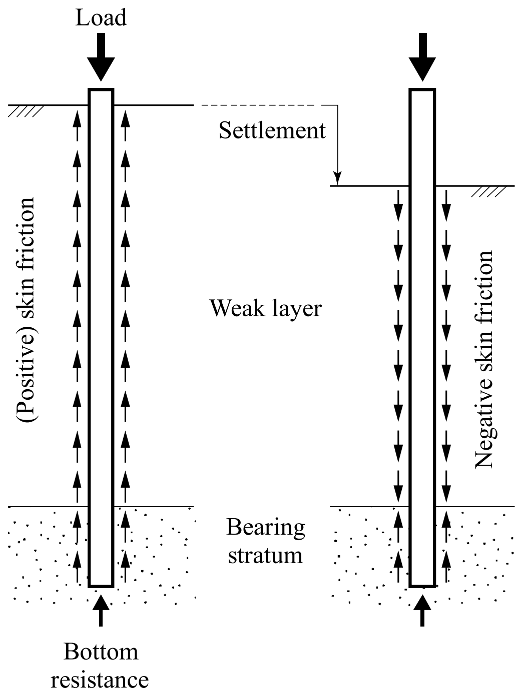


Fig. 3.4.14 Negative Skin Friction Force

(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].

$$P_k = \frac{E}{4\left(1-\nu^2\right)}\left(\frac{t}{r}\right)^3 \qquad \textbf{(3.4.57)}$$

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].

$$\frac{\sigma_{cr}}{\sigma_y} = 0.86 + 2.7\frac{t}{r} \quad \left(0.01 \le \frac{t}{r} \le 0.10\right) \qquad \textbf{(3.4.58)}$$

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/m²)

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**)

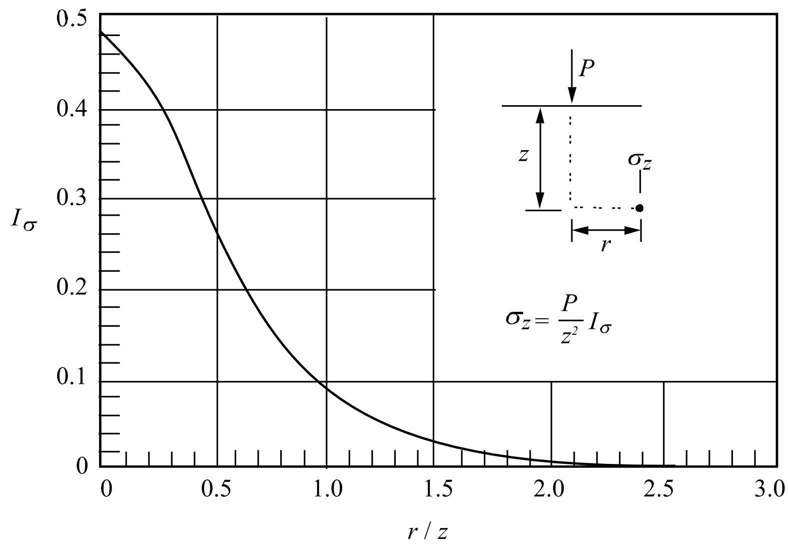


Fig. 3.5.1 Influence Value of Vertical Stress in the Ground due to Vertical Concentrated Load

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**)

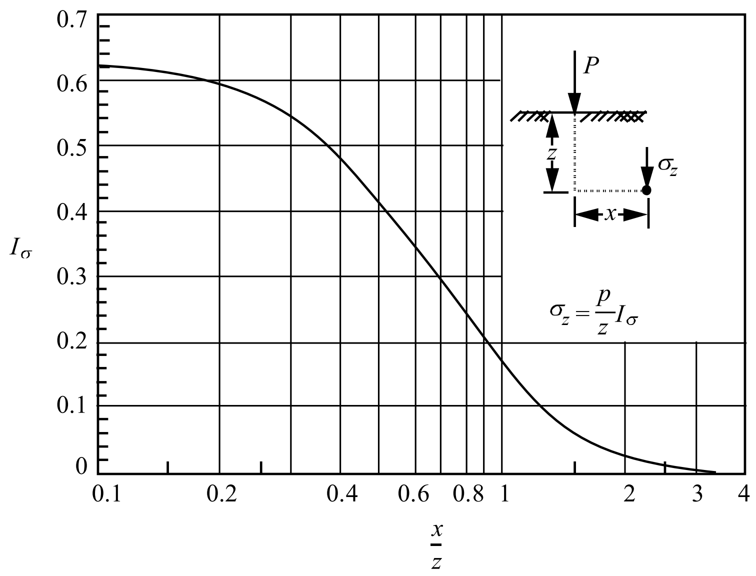


Fig. 3.5.2 Influence Value of Vertical Stress in the Ground due to Vertical Line Load

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**)

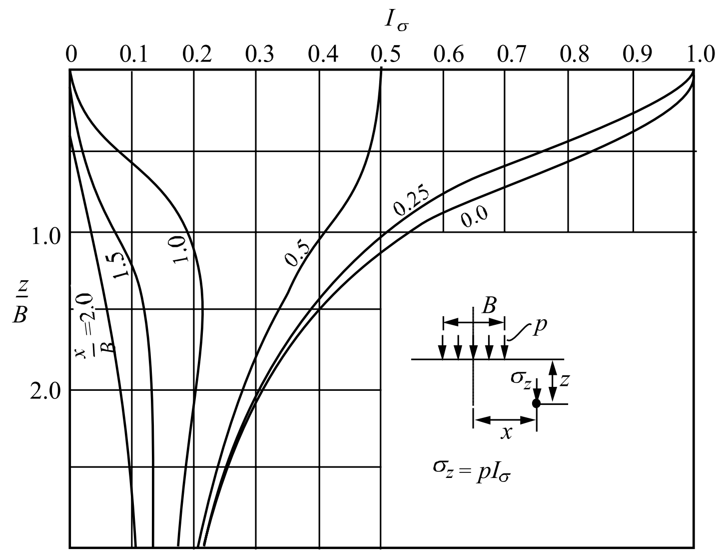


Fig. 3.5.3 Influence Value of Vertical Stress in the Ground due to Uniformly Distributed Strip Load

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)

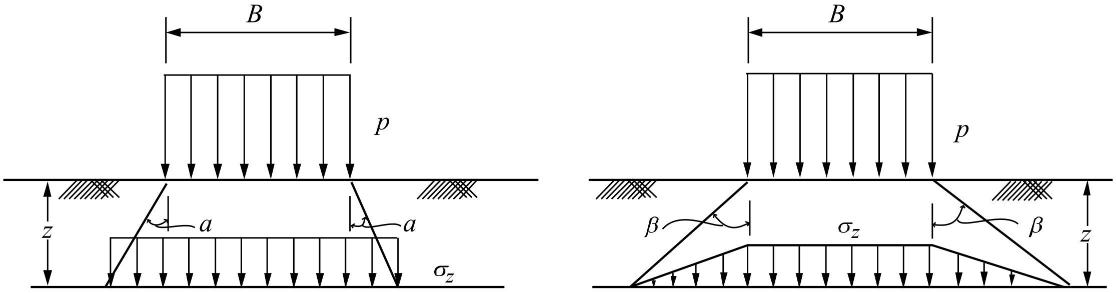
p : strength of the load (kN/m^2)

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

$$\sigma_z = \frac{p}{1+\left(\dfrac{z}{B}\right)\tan\beta} \tag{3.5.5}$$

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, β = 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**.

$$\sigma_z = pI_\sigma \tag{3.5.6}$$

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.

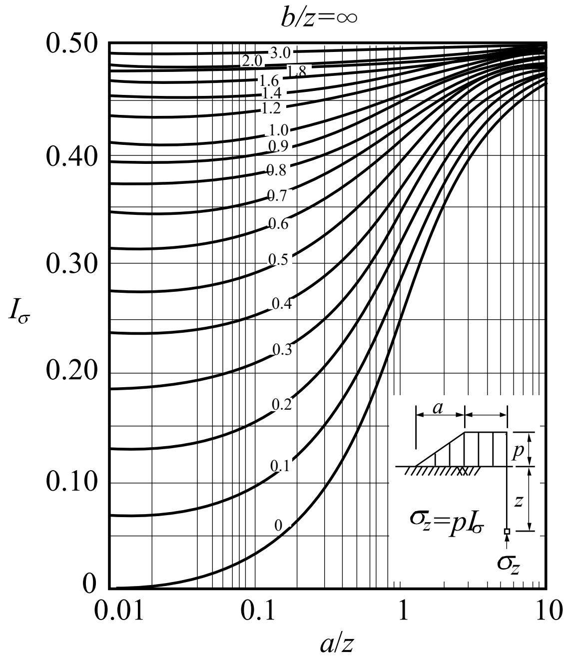


Fig. 3.5.5 Influence Value of Vertical Stress in the Ground due to Strip Load

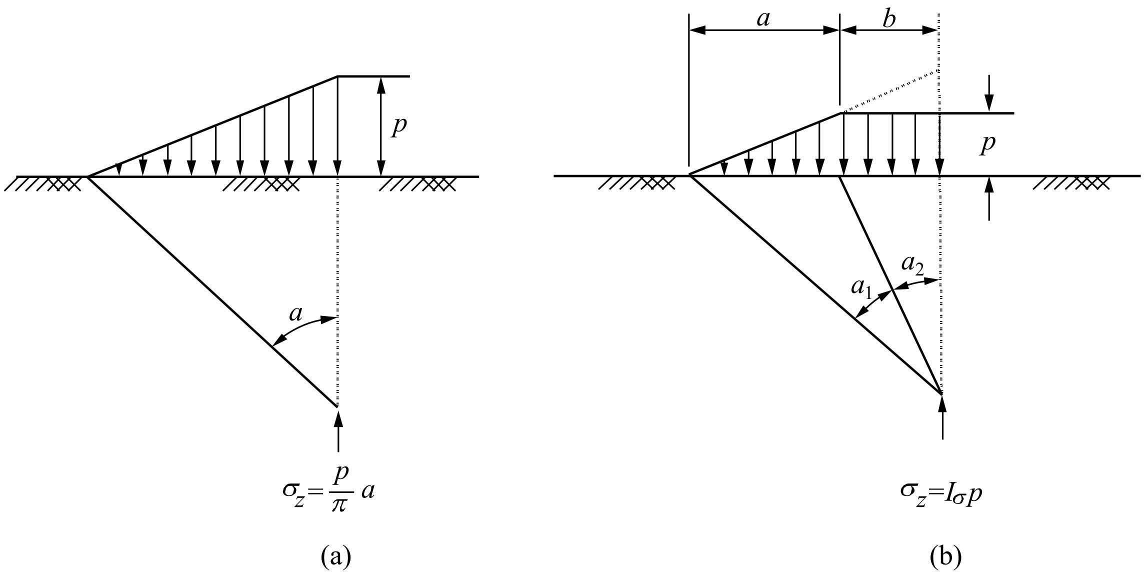


Fig. 3.5.6 Strip Load Consisting of Overlapped Triangular Loads

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**)

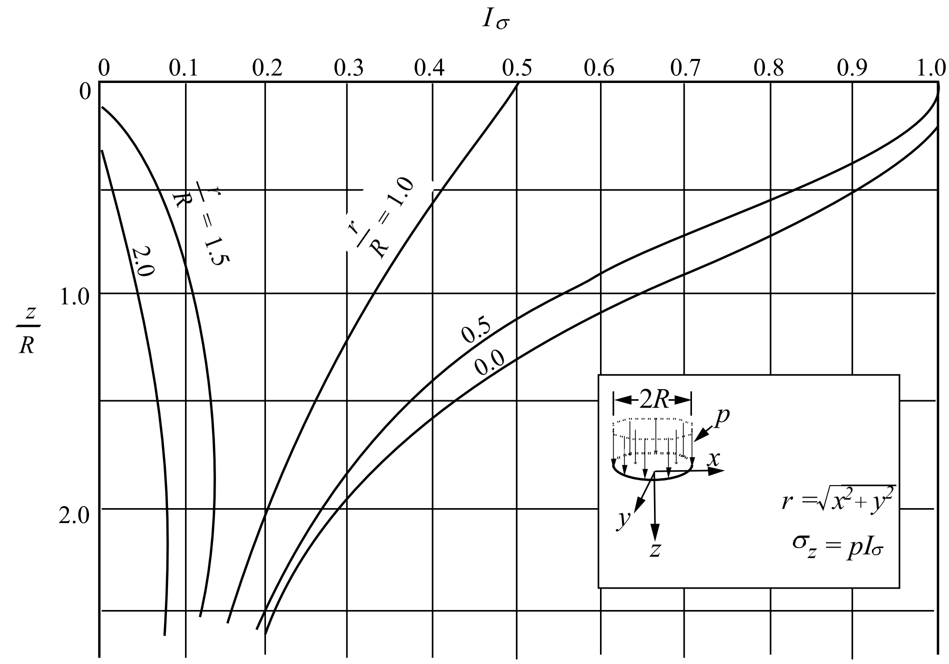


Fig. 3.5.7 Influence Value of Vertical Stress in the Ground due to Uniformly Distributed Load in a Circle Shape

(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**.

$$\sigma_z = pI_\sigma \tag{3.5.8}$$

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.

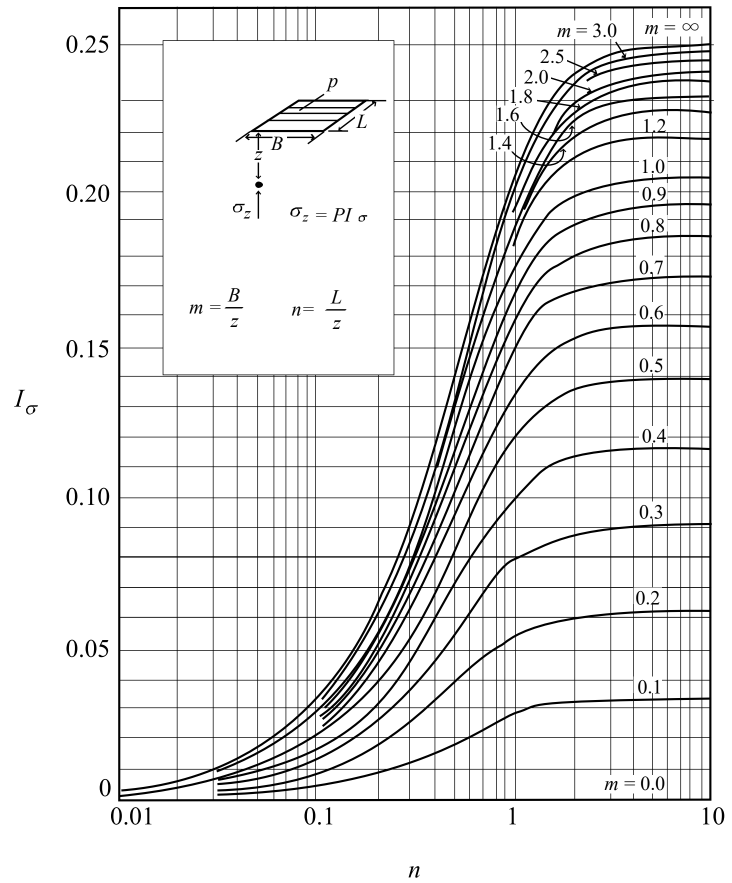


Fig. 3.5.8 Influence Value of Vertical Stress in the Ground due to Uniformly Distributed Load in a Rectangular Shape

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 ν 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)

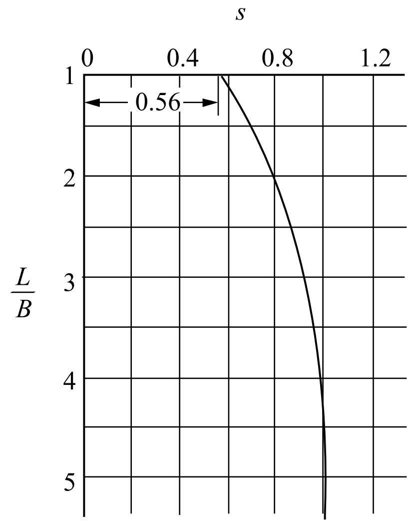


Fig. 3.5.9 Influence Value to Settlement of Corner Points due to Uniformly Distributed Load in a Rectangular Shape

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.

$$S_0 = \sum S \qquad \textbf{(3.5.17)}$$

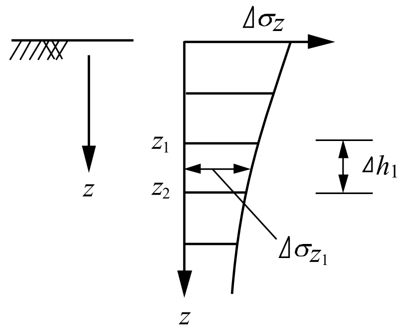


Fig. 3.5.10 Division of the Clayey Layer in Calculating the Consolidation Settlement

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.

$$S_s = \frac{C_\alpha}{1+e_0} h \log_{10}\left(\frac{t}{t_0}\right) \quad \textbf{(3.5.18)}$$

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].

$$C_\alpha = (0.03 \sim 0.05) C_c \quad \textbf{(3.5.19)}$$

(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)].

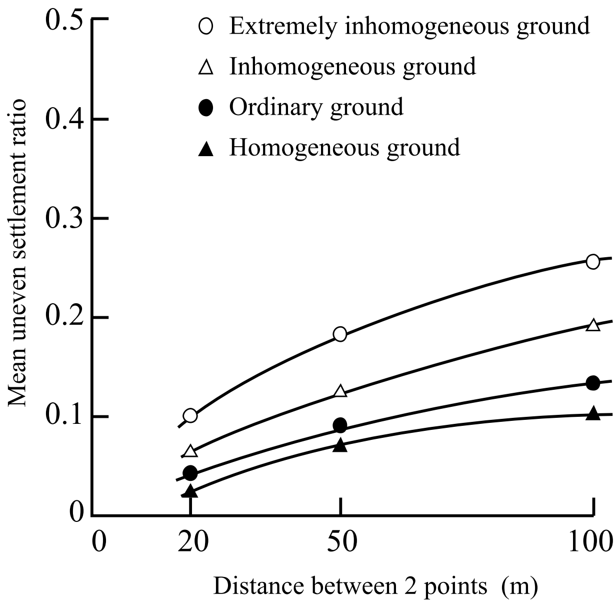


Fig. 3.5.11 Relationship between Distance and Differential Settlement Ratio in Reclaimed Land

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

[References]

1) Architectural Institute of Japan: Guideline for design of architectural foundation, p.108, 2001 (in Japanese)
2) Davis, E.H. and Booker: The effect of increasing strength with depth on the bearing capacity of clays, Geotechnique, Vol.23, No,4, 1973
3) Nakase, A.: Bearing capacity of rectangular footings on clay of strength increasing linearly with depth, Soil and Foundations, Vol. 21, No.4, pp.101-108, 1981
4) Yamaguchi, K.: Soil Mechanics (Fully revised Edition), Chapter 9 Bearing strength, Giho-do Publishing, pp.273-274, 1984 (in Japanese)
5) Kobayashi, M., M. Terashi, K. Takahashi and K. Nakajima: A New Method for Calculating the Bearing Capacity of Rubble Mounds, Rept. of PHRI Vol.26, No.2, 1987 (in Japanese)
6) Shoji, Y.: Study on shearing Properties of Rubbles with Large Scale Triaxial Compression Test, Rept. of PHRI Vol. 22, No,4,1983 (in Japanese)
7) Minakami, J. and M. Kobayashi: Soil Strength Characteristics of Rubble by Large Scale Triaxial Compression Test, Rept. of PHRI No.699, 1991 (in Japanese)
8) Japan Road Association: Specifications for highway bridges, IV - Substructures, pp.317-437, 2017. (in Japanese)
9) Ikehara, T. and Yokoyama, A.: The stability of the well-foundation under the lateral force, JSCE Magazine, Vol.38, No.12, pp.19-23, 1953. (in Japanese)
10) Takahashi, K. and Sawaguchi, M.: Experimental study on the lateral resistance of a well, Report. of PHRI, Vol.16, No.4, pp.3-34, 1977. (in Japanese)
11) Japanese Society of Soil Mechanics and Foundation Engineering: Design methods for soil mechanics and foundation engineering, pp.296-297, 1961. (in Japanese)
12) Japanese Geothechnical Society: Survey, design, construction and inspection of pile foundation, pp.3-4, 2004. (in Japanese)
13) Watabe, Y., Mizutani, T., Kaneko, T. and Masukado, K.: Applicability of piled foundation at confined disposal facilities in coastal area - in situ demonstration for pile-driving and impermeable performance at untreated wastes ground -, Technical Note of PARI, No.1321, 2016. (in Japanese)
14) Mizutani, T.: Estimation of axial capacity of piles utilizing load tests, Report of PARI, Vol.55, No.1, pp.3-23, 2016. (in Japanese)

15) G. G. Meyerhof: Penetration tests and bearing capacity of cohesionless soils, Proc. of ASCE, Vol.82, No.SM1, Paper 866, 1956.

16) Kikuchi, Y., Mizutani, T. and Morikawa, Y.: Systematization of design verification and installation of steel pipe piles, Technical Note of PARI, No.1202, p.31, 2009. (in Japanese)

17) Komatsu, M., Hijikuro, K. and Tominaga, M.: A few experiment on plugging effect of steel pipe piles with large diameter, JSSMFE magazine (Tsuchi-to-Kiso), Vol.17, No.5, pp.11-16, 1969. (in Japanese)

18) Yamahara, H.: Load transfer mechanism and applications of steel pipe piles, JSSMFE magazine (Tsuchi-to-Kiso), Vol.17, No.11, pp.19-27, 1969. (in Japanese)

19) Saito, R., Kusakabe, O., Kikuchi, Y., Fukui, J. Sasaki, H., Geshi, H. and Yoshizawa, Y: End bearing capacity of steel pipe piles with device at pile base in Tokyo port seaside road project, Proc. of 40th Japan National Conf. on Geotechnical Engineering, pp.1681-1682, 2005. (in Japanese)

20) Mizutani, T.: Variance of dynamic load test results of piles at a single construction site, Journal of JSCE, Ser.C (Geosphere Engineering), Vol.71, No.3, pp.228-240, 2015. (in Japanese)

21) Takahashi, K.: Behavior of Single Piles in Subsiding Ground, Technical Note of PHRI, No.533, p.17, 1985. (in Japanese)

22 M. J. Tomlinson and J. Woodward: Pile design and construction practice, 6th edition, pp.143-150, CRC Press, 2015.

23) Japan Road Association: Specifications for highway bridges, IV - Substructures, pp.237-250, 2017. (in Japanese)

24) Japanese Society of Soil Mechanics and Foundation Engineering: Design method and commentary for pile foundations, pp.257-260, 1985. (in Japanese)

25) Yasufuku, N., Ochiai, H. and Ohno, S.: Pile end bearing capacity of sandy ground with consideration of soil compressibility, JGS magazine (Tsuchi-to-Kiso), Vol.49, No.3, pp.12-15, 2001. (in Japanese)

26) Japan Road Association: Pile design handbook for highway bridge foundation, pp.433-440, 2015. (in Japanese)

27) Japan Road Association: Pile design handbook for highway bridge foundation, pp.445-458, 2015. (in Japanese)

28) Yokoyama, Y.: Design of steel piles and construction, Revised edition, pp.94-99, Sankai-do Publishing, 1966. (in Japanese)

29) Yokoyama, Y.: Calculation methods of pile structures and example, pp.147-152, Sankai-do Publishing, 1977. (in Jpanese)

30) Japan Road Association: Specifications for highway bridges, IV - Substructures, p.270, 2017. (in Japanese)

31) Nakase, A., Okumura, T. and Sawaguchi, M.: Easy-to-understand Foundation works, pp.52-54, Kajima Institute Publishing, 1995. (in Japanese)

32) Architectual Institute of Japan: Design recommendations for foundations of buildings, 2nd edition, pp.152-154, 2001. (in Japanese)

33) Kitazume, M. and Murakami, K.: Behaviour of sheet pile walls in the improved ground by sand compaction piles of low replacement area ratio, Report of PHRI, Vol.32, No.2, pp.183-211, 1993. (in Japanese)

34) Tanikawa, M., Sawaguchi, M. and Tanaka, M.: Horizontal subgrade reaction of pile in composite ground - Relation between coefficient of horizontal subgrade reaction and ratio of replacement of clay ground by sand piles -, Proc. of 28th Japan National Conf. of Soil Mechanics and Foundation Engineering, pp.1815-1816, 1993. (in Japanese)

35) Takahashi, K. and Ikki, Y.: Lateral resistance of a pile in rubble mound, Report of PHRI, Vol.30, No.2, pp.229-273, 1991. (in Japanese)

36) Kikuchi, Y., Ogura, T., Ishimaru, M. and Kondo, T.: Coefficient of subgrade lateral reaction in rubble ground, Proc. of 53rd JSCE Annual Meeting, Part 3(B), pp.52-53, 1998. (in Japanese)

37) Kikuchi, Y., Takahashi, K. and Suzuki, M.: Lateral resistance of single piles under large repeated loads, Report of PHRI, Vol.31, No.4, pp.33-60, 1992. (in Japanese)

38) Shinohara, T. and Kubo, K.: Experimental study on the lateral resistance of piles, Part 1 - Lateral resistance of single free head piles embedded in uniform sand layer -, Monthly Report of Transportation Technical Research Institute, Vol.11, No.6, pp.169-242, 1961. (in Japanese)

39) Kikuchi, Y., Takahashi, K. and Hirohashi, T.: Lateral load tests on piled slab structures, Technical Note of PHRI, No.773, pp.9-12, 1994. (in Japanese)

40) Kubo, K. and Saegusa, F.: Two-way cyclic loading tests of model piles, Proc. of 2nd Research Presentation Conference of PHRI, pp.64-73, 1964. (in Japanese)

41) Kikuchi, Y.: Lateral resistance of soft landing moundless structure with piles, Technical Note of PARI, No.1039, pp.59-71, 2003. (in Japanese)

42) Bureau of Port and Harbours, Ministry of Transport: Handbook of countermeasures for liquifaction in reclaimed land, Coastal Development Institute of Technology, pp.314-319, 1997. (in Japanese)

43) Yokoyama, Y.: Calculation methods of pile structures and example, Sankai-do Publishing, pp.33-38, 1977. (in Jpanese)

44) Yokoyama, Y.: Calculation methods of pile structures and example, Sankai-do Publishing, p.68, 1977. (in Jpanese)

45) Yokoyama, Y.: Calculation methods of pile structures and example, Sankai-do Publishing, pp.47-56, 1977. (in Jpanese)

46) K. Terzaghi: Evaluation of coefficients of subgrade reaction, Géotechnique, Vol.5, No.4, pp.297-326, 1995.

47) Yokoyama, Y.: Design of steel piles and construction, Revised edition, pp.139-141, Sankai-do Publishing, 1966. (in Japanese)

48) Yokoyama, Y.: Calculation methods of pile structures and example, Sankai-do Publishing, pp.71-73, 1977. (in Jpanese)

49) Kikuchi, Y.: Horizontal subgrade reaction model for estimation of lateral resistance of pile, Report of PARI, Vol.48, No.4, pp.2-22, 2009. (in Japanese)

50) Japan Road Association: Specifications for highway bridges, IV - Substructures, pp.259-260, 2017. (in Japanese)

51) Kubo, K.: A new method for the estimation of lateral resistance of piles, Report of PHRI, Vol.2, No.3, pp.1-37, 1964. (in Japanese)

52) Yamashita, I., Inatomi, T., Ogura, K. and Okuyama, Y.: New standard curves in the PHRI method, Report of PHRI, Vol.10, No.1, pp.107-168, 1971. (in Japanese)

53) Yamashita, I., Inatomi, T., Ogura, K. and Okuyama, Y.: New standard curves in the PHRI method, Report of PHRI, Vol.10, No.1, pp.136-145, 1971. (in Japanese)

54) Kubo, K.: A new method for the estimation of lateral resistance of piles, Report of PHRI, Vol.2, No.3, p.23, 1964. (in Japanese)

55) Kikuchi, Y.: Horizontal subgrade reaction model for estimation of lateral resistance of pile, Report of PARI, Vol.48, No.4, pp.9-11, 2009. (in Japanese)

56) Port and Harbour Technical Research Institute and Yawata Iron & Steel Co., Ltd.: Studies on the lateral resistance of H-piles, pp.345-353, 1963. (in Japanese)

57) Sawaguchi, M.: Soil constants for piles, Report of PHRI, Vol.7, No.2, pp.65-94, 1968. (in Japanese)

58) Kubo, K.: Lateral resistance of short piles, Report of PHRI, Vol.5, No.3, pp.20-25, 1966.

59) Shinohara, T. and Kubo, K.: Experimental study on the lateral resistance of piles, Part 1 - Lateral resistance of single free head piles embedded in uniform sand layer -, Monthly Reports of Transportation Technical Research Institute, Vol.11, No.6, p.229, 1961. (in Japanese)

60) Sawaguchi, M.: Soil constants for piles, Report of PHRI, Vol.7, No.2, p.83, 1968. (in Japanese)

61) Kubo. K.: Experimental study on the lateral resistance of piles, Part 3 - Lateral resistance of single free-head battered piles and single fixed-head vertical piles -, Monthly Reports of Transportation Technical Research Institute, Vol.12, No.2, pp.182-190, 1962. (in Japanese)

62) Sawaguchi, M.: Experimental investigation on the horizontal resistance of coupled piles, Report of PHRI, Vol.9, No.1, pp.43-45, 1970. (in Japanese)

63) Yokoyama, Y.: Design of steel piles and construction, Revised edition, pp.193-197, Sankai-do Publishing, 1966. (in Japanese)

64) Aoki, Y.: Design method for coupled pile under lateral loading, JSSMFE magazine (Tsuchi-to-Kiso), Vol.18, No.8, pp.27-32, 1970. (in Japanese)

65) Segawa, M., Uchida, T. and Katayama, T.: Design method for coupled pile (Part2), Technical Note of PHRI, No.110, 1970. (in Japanese)

57) Sawaguchi, M.: Experimental investigarion on the horizontal resistance of coupled piles, Report of PHRI, Vol.9, No.1, pp.3-69, 1970. (in Japanese)

67) Matsumura, S., Matsubara, T., Fujii, N., Mizutani, T., Morikawa, Y. and Sato, M.: Lateral resistance of coupled piles with ist intermediate soil stabilized by cement treating method,Report of PARI, Vol.56, No.3, pp.3-27, 2017. (in Japanese)

68 Á, Kézdi: Bearing capacity of piles and pile groups, Proc. of 4th ICSMFE, pp.46～51, 1957.

69) K.Terzaghi, R. B. Peck, and G. Mesri: Soil mechanics in engineering practice, 3rd edition, pp.435～436, 1996.

70) Architectual Institute of Japan: Technical standards and commentaries for steel-pipe-pile foundations of buildings, p.55, 1963. (in Japanese)

71) Architectual Institute of Japan: Design recommendations for foundations of buildings, 2nd edition, pp.254-256, 2001. (in Japanese)

72) Sawaguchi, M.: Approximate calculation of negative skin friction of a pile, Report of PHRI, Vol.10, No.3, pp.67-87, 1971. (in Japanese)

73) Takahashi, K.: Behavior of single piles in subsiding ground, Technical Note of PHRI, No.533, pp.41-50, 1985. (in Japanese)

74) Takahashi, K.: Behavior of single piles in subsiding ground, Technical Note of PHRI, No.533, pp.8-11, 1985. (in Japanese)

75) Takahashi, K.: Behavior of single piles in subsiding ground, Technical Note of PHRI, No.533, pp.92-168, 1985. (in Japanese)

76) Yokoyama, Y.: Design of steel piles and construction, Revised edition, pp.230-235, Sankai-do Publishing, 1966. (in Japanese)

77) Kishida, H. and Takano, A.: Buckling of steel pipe piles and reinforcement at pile end, Transactions of AIJ, Vol.213, pp.29-38, 1973. (in Japanese)

78) Japan Road Association: Specifications for highway bridges, IV - Substructures, 2017. (in Japanese)

79) Architectual Institute of Japan: Design recommendations for foundations of buildings, 2nd edition, 2001. (in Japanese)

80) Japan Road Association: Pile design handbook for highway bridge foundation, 2015. (in Japanese)

81) Japan Road Association: Pile construction handbook for highway bridge foundation, 2015. (in Japanese)

82) Akai, K.: Bearing Capacity and settlement of soil, Sankai-do Publishing, 1964 (in Japanese)

83) Ishii, Y.: Tschbotarioff Soil Mechanics, (Vil. 1) Gihoi-do Publishing, p.212,1957 (in Japanese)

84) J. O. Osterburg: Influence values for vertical stresses in a semi-infinite mass due to an embankment loading, Proc. 4th. Int. Conf. S.M.F.E., Vol.2, 1957

85) Kobayashi, M., J. Minakami and T. Tsuchida: Determination of the Horizontal Coefficient of Consolidation cohesive soil, Rept. of PHRI Vol.29, No.2, 1990 (in Japanese)

86) Nakase, A., M. Kobayashi and A. Kanechika: Consolidation Parameters of Over consolidated Clays, Rept. of PHRI Vol. 12, No. 1, pp. 123-139, 1973 (in Japanese)

87) L.A. Palmer and P.P. Brown: Settlement analysis for areas of continuing subsidence, Proc. 4th. Int. Conf. S.M.F.E, Vol.1, pp.395-398,1957

88) R.L. Schifflnan and R.E. Gibson: Consolidation of nonhomogeneous clay layers, Journal of S.M.F.E., ASCE, Vol.90, No. SM 5, pp.1-30,1964

89) Kobayashi, M.: Numerical Analysis of One-Dimensional Consolidation Problems, Rept. of PHRI Vol. 21, No.1, 1982 (in Japanese)

90) Kobayashi, M.: Study on the application of Finite Element Method to settlement analysis, Tokyo Institute of Technology Dissertation, Technical Note of Soil Mechanics Laboratory, No.1,1990 (in Japanese)

91) Kobayashi, M.: Finite Element Analysis of the Effectiveness of Sand Drains, Rept. of PHRI Vol. 30, No.2, 1991 (in Japanese)

92) Mesri, G.: Coefficient of secondary compression, Proc. A.S.C.E, Vol.99, SM1, pp.123-137, 1973

93) WATABE, Y., KANEKO, T.: Interpretation of long-term consolidation behavior of worldwide clays on the basis of the isotache concept, Rept. of PHRI Vol. 54, No.1, pp.3-30, 2015 (in Japanese)

94) Šuklje, L.: The analysis of the consolidation process by the isotache method, Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, London, Vol.1, 200‑206, 1957.

95) Kasugai, Y., K. Minami and H. Tanaka: The Prediction of the Lateral Flow of Port and Harbour Structures, Technical Note of PHRI No. 726, 1992 (in Japanese)

96) Okumura, T. and T. Tsuchida: Prediction of Differential Settlement with Special Reference to Variability of Soil Parameters, Rept. of PHRI Vol. 20, No. 3, 1981 (in Japanese)

97) Tsuchida, T. and K. Ono: Evaluation of Differential Settlements with Numerical Simulation and Its Application to Airport Pavement Design, Rept. of PHRI Vol.27, No.4, 1988 (in Japanese)

98) Tsuchida, T.: Estimation of differential settlement in reclaimed land, Proceedings of Annual Conference of PHRI, 1989 (in Japanese)

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.

$$m \cdot \frac{\gamma_S \cdot \left[\sum\left\{x\left(W_k + q_k\right) + aP_{Hk}\right\}\right]}{\gamma_R \cdot R\sum\left\{c_k \ell + \left(W'_k + q_k\right)\cos\theta \tan\phi_k\right\}} \leq 1 \qquad \textbf{(4.2.1a)}$$

$$m \cdot \frac{\gamma_S \cdot \sum \left\{ (W_k + q_k)\sin\theta + \frac{1}{R} a P_{Hk} \right\}}{\gamma_R \cdot \sum \left\{ c_k s + (W'_k + q_k)\cos^2\theta \tan\phi_k \right\} \sec\theta} \leq 1 \qquad \textbf{(4.2.1b)}$$

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.

$$m \cdot \frac{S_d}{R_d} \leq 1$$

$$S_d = \gamma_S S_k = \gamma_S \cdot \sum \left\{ (W_k + q_k)\sin\theta + \frac{1}{R} a P_{Hk} \right\} \tag{4.2.3}$$

$$R_d = \gamma_R R_k = \gamma_R \cdot \sum \left\{ c_k s + (W'_k + q_k)\cos^2\theta \tan\phi_k \right\} \sec\theta$$

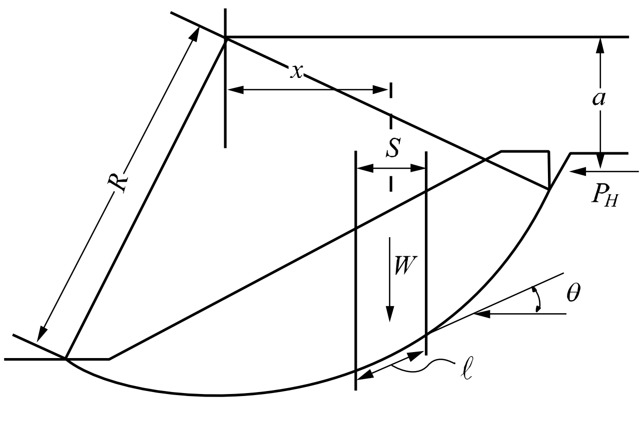


Fig. 4.2.1 Circular Slip Failure Analysis using Modified Fellenius Method

(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.

$$\frac{m}{\gamma_R}\cdot\frac{\gamma_S\cdot\sum\left\{(W_k+q_k)\sin\theta+aP_{Hk}/R\right\}}{\sum\left[\left\{c_k s+(W'_k+q_k)\tan\phi_k\right\}\dfrac{\sec\theta}{1+\tan\theta\tan\phi_k/(m/\gamma_R)}\right]}\leq 1 \tag{4.2.4}$$

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.

$$\frac{m}{\gamma_R}\cdot\frac{\gamma_S\cdot\sum\{(W_k+q_k)\sin\theta+aP_{Hk}/R\}}{\sum\left[\{nc_k s+(W'_k+q_k)\tan\phi_k\}\times\dfrac{\sec\theta}{n+\{\tan\theta-\tan(\beta\theta)\}\tan\phi_k/(m/\gamma_R)}\right]}\leq 1 \tag{4.2.5}$$

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.

$$m\cdot\frac{\gamma_S\cdot\sum\{(W_k+q_k)\sin\theta+P_{Hk}\cos\theta\}}{\gamma_R\cdot\sum[c_k\ell+\{(W'_k+q_k)\cos\theta-P_{Hk}\sin\theta\}\tan\phi_k]}\leq 1 \tag{4.2.6}$$

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.

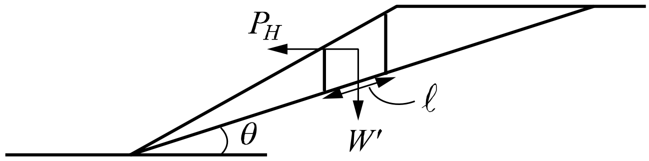


Fig. 4.2.2 Examination of Slope Stability Analysis using Linear Sliding Surface

[References]

1) Tsuchida, T.and TANG Yi Xin: The Optimum Safety Factor for Stability Analyses of Harbour Structures by Use of the Circular Arc Slip Method, Rept. of PHRI Vol. 5、No. 1、pp. 117-146, 1996 (in Japanese)

1) R.F, Scott: Principle of Soil mechanics, Addison Wesley, p.431, 1972

3) Yamaguchi, K.: Soil Mechanics (Fully Revised Edition) Chapter 7, Stability analysis of earth structure, Giho-do Publishing, pp.197-223, 1969 (in Japanese)

4) Akio Nakase: The ϕ=0 analysis of stability and unconfined compression strength, Siol and Foundation, Vol. 7, No. 2, pp.33- 50, 1967 (in Japanese)

5) A.W. Bishop: The use of the slip circle in the stability analysis of slopes, Geotechnique, Vol. 5, No. 1, pp.7-17. 1955

6) Nomura, K., T. Hayafuji and F. Nagatomo: Comparison between Bishop's method and Tschebotarioff's method in slope stability analysis, Rept. of PHRI Vol. 7 No. 4, pp.133-175, 1968 (in Japanese)

7) Kobayashi, M.: Outstanding issues in stability analysis of ground, Proceedings of Annual Conference of PHRI 1976, pp.73-93, 1976 (in Japanese)

8) Tsuchida, T., M. Kobayashi and T. Fukuhara: Calculation method for bearing capacity by circular slip analysis utilizing slice method, Proceedings of 33rd Conference on Geotechnical Engineering, pp.1371-1372, 1998 (in Japanese)

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.

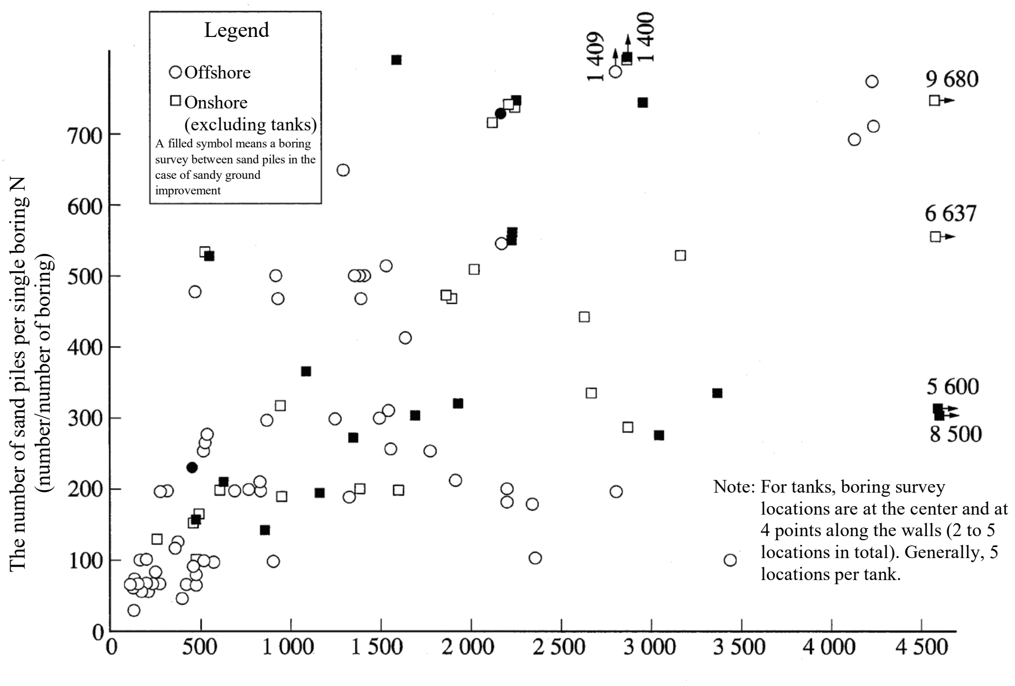


Fig. 5.2.1 Examples of the Relationship between the Total Number of Sand Piles and the Number of Sand Piles per Single Boring[71)]

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

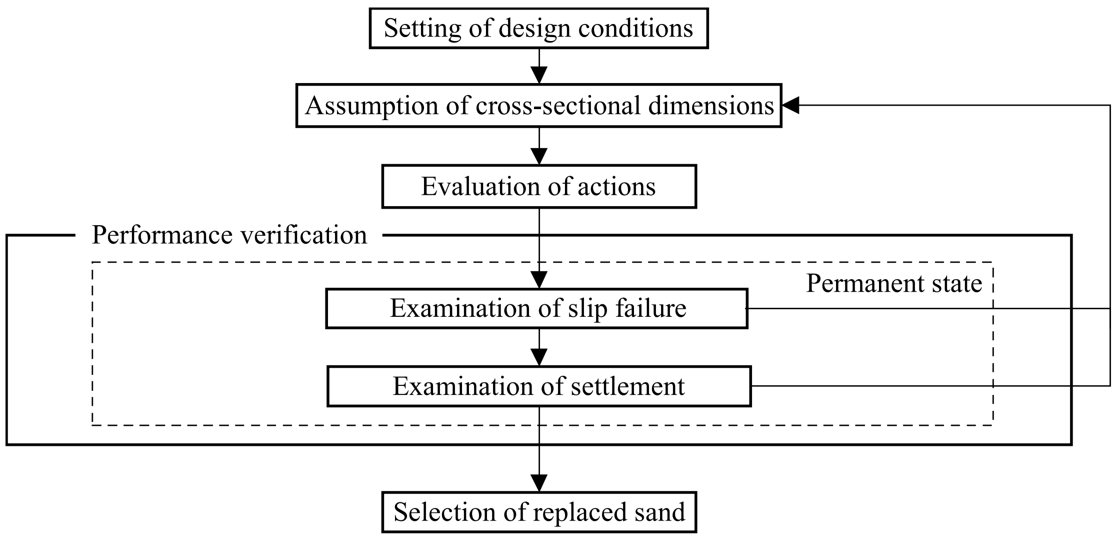
(1) The replacement methods can be separated into the replacement of ground by excavation (foundation replacement by excavation) and forced replacement. The replacement of ground by excavation is widely used in offshore work and implemented in a manner that removes soft soil by excavating it with pump 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)].

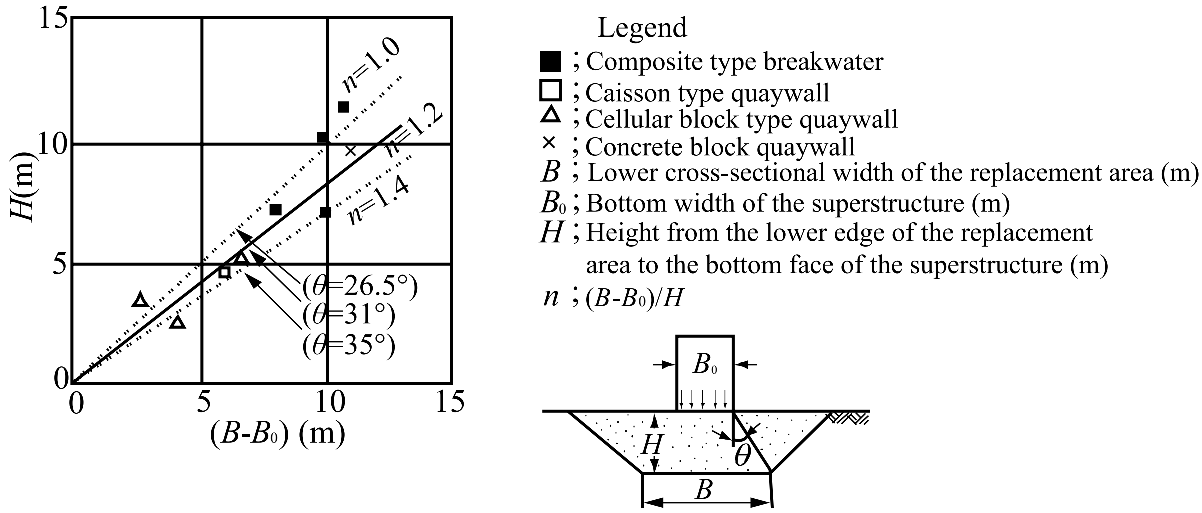


Fig. 5.3.2 Relationship between Replacement Widths and Depths

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.

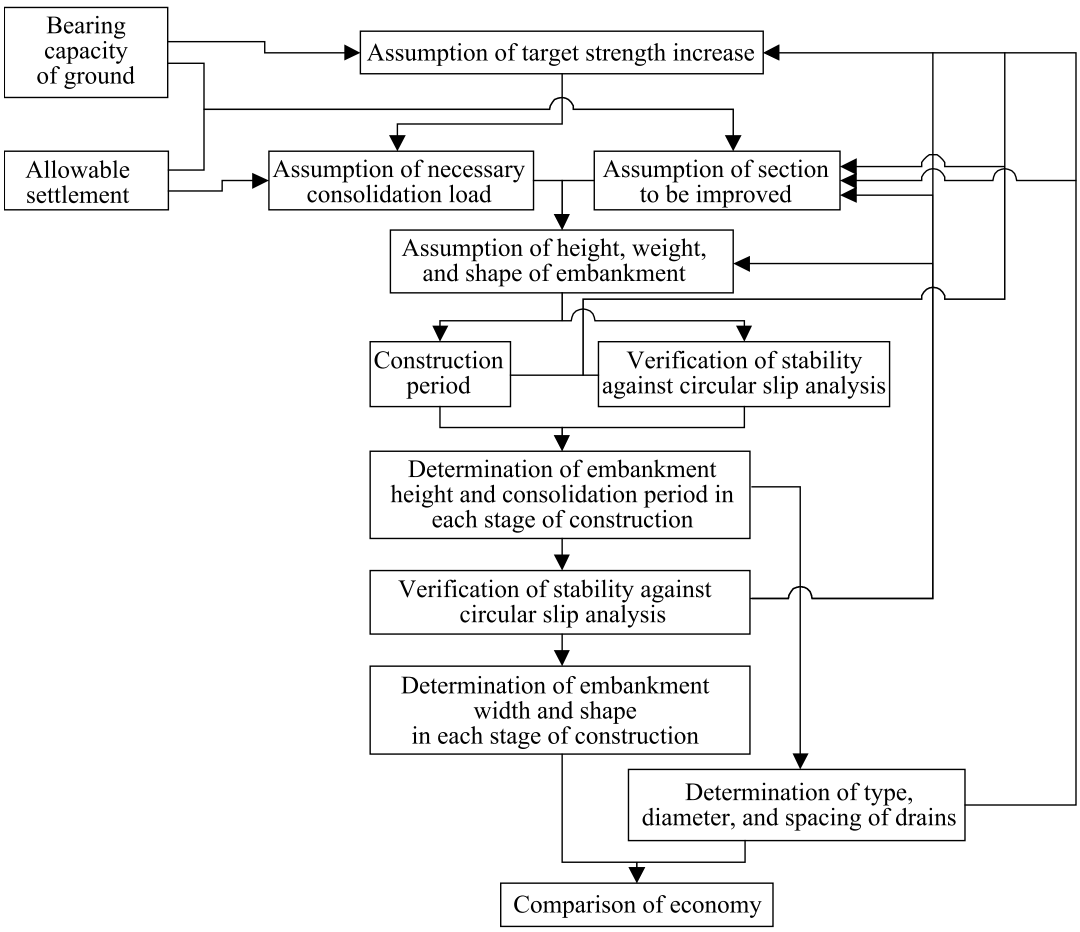


Fig. 5.4.1 Example of Performance Verification Procedure for the Vertical Drain Method

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**).

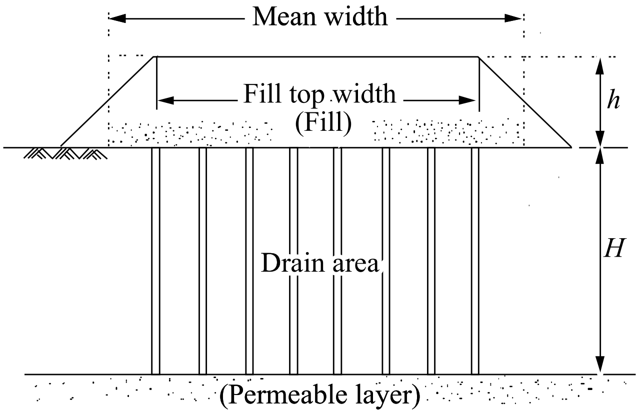


Fig. 5.4.2 Width of Fill for the Vertical Drain Method

(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.

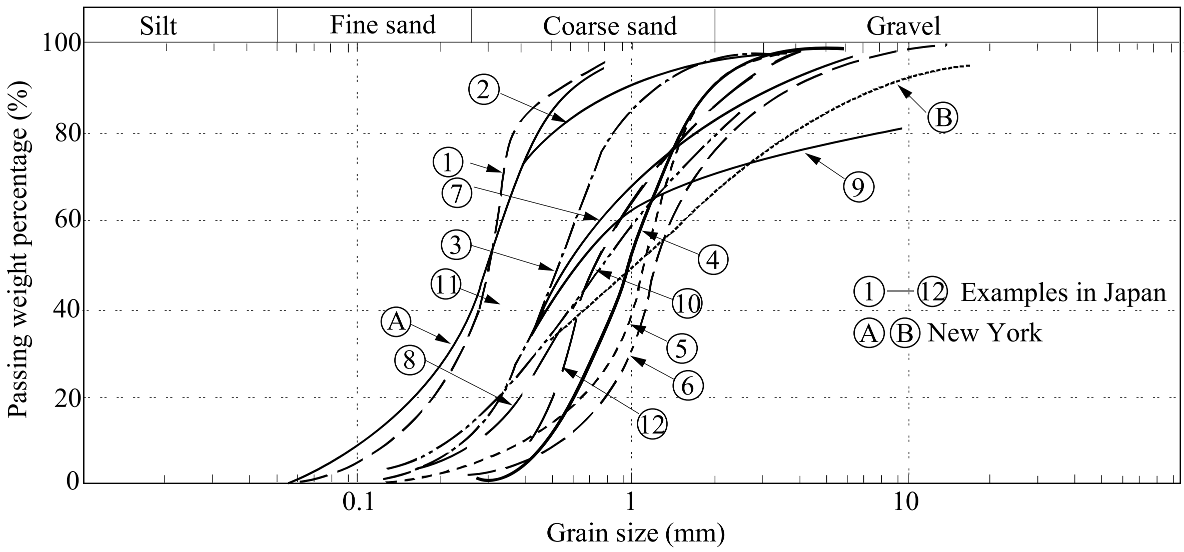


Fig. 5.4.3 Examples of Sand used in Sand Piles

(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)].

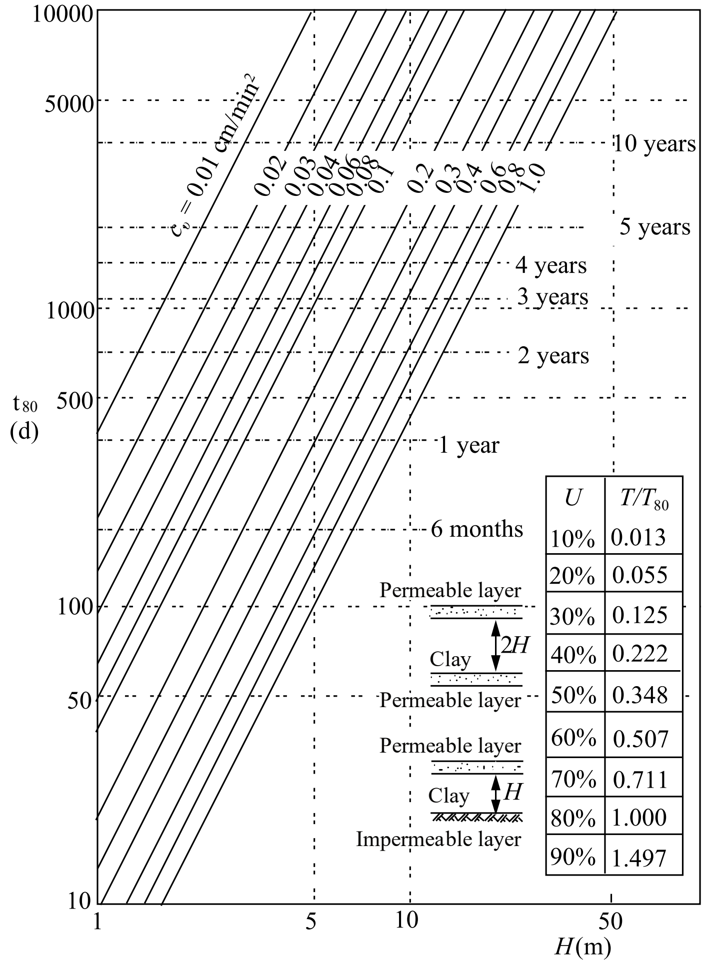
② **Drain intervals**

(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 (cm²/min), respectively.



U	T/T_{80}
10%	0.013
20%	0.055
30%	0.125
40%	0.222
50%	0.348
60%	0.507
70%	0.711
80%	1.000
90%	1.497

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).

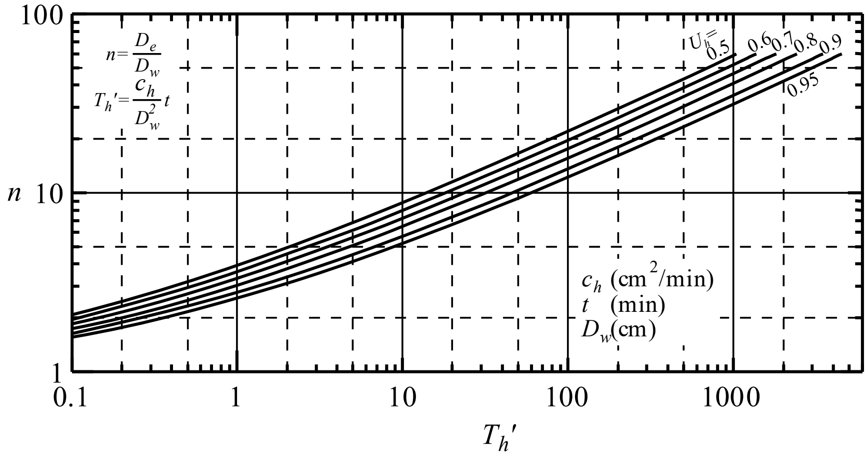


Fig. 5.4.5 Calculation Chart for the *N*-value

(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

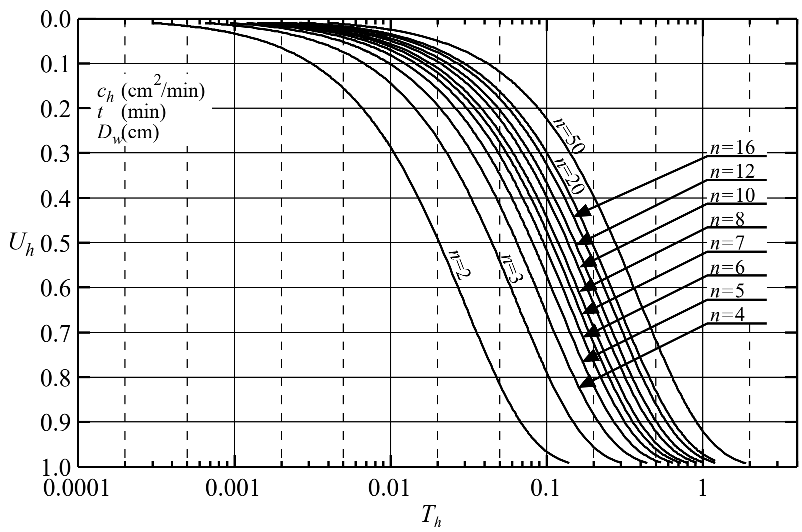
T_h : time factor for consolidation due to water flowing in a horizontal direction

c_h : coefficient of consolidation due to water flowing in a horizontal direction (cm²/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.

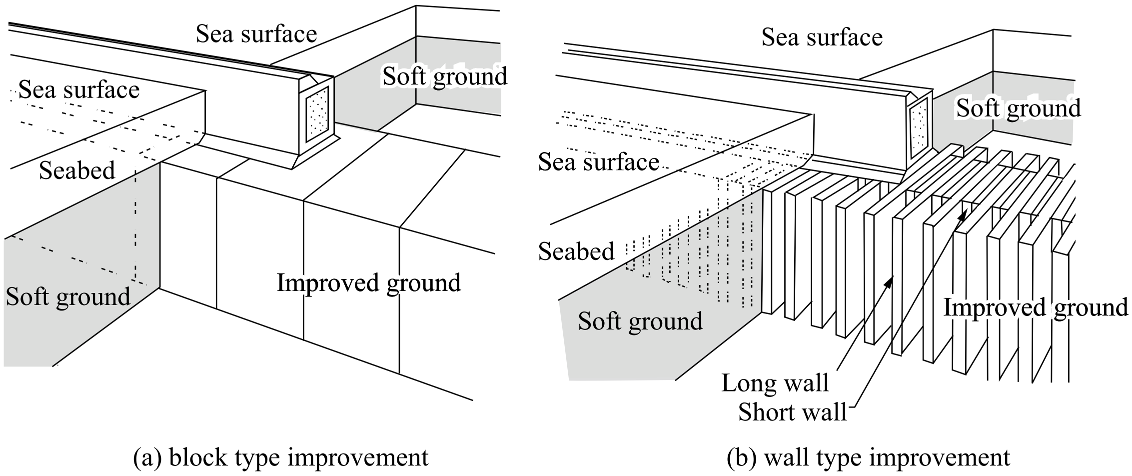


Fig. 5.5.1 Typical Improvement Patterns in the Deep Mixing Method

(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.

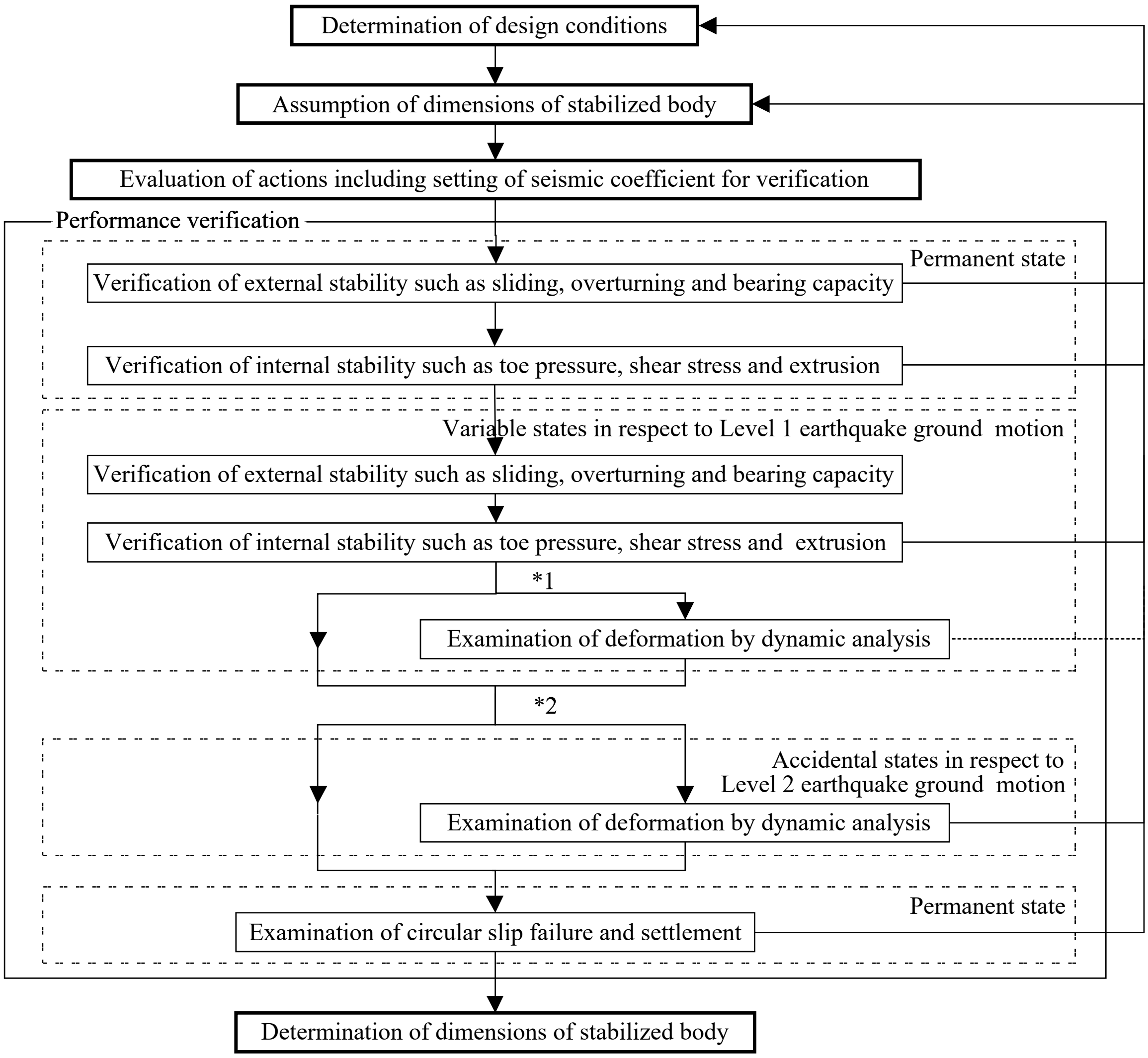
(g) Fixed type: A type of improvemed ground constructed by improving soft ground all the way through the bearing layer so that a stabilized body is seated on the bearing layer and transfers actions of the superstructure to the bearing layer.

(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

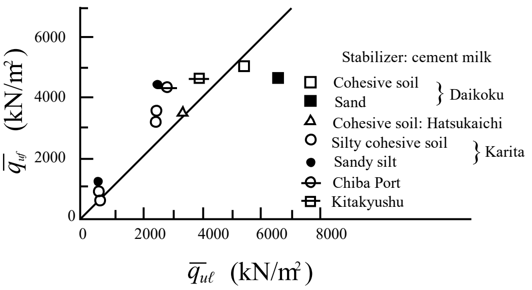
① The strength of stabilized soil depends largely on the physical characteristics and chemical properties of the object soil for improvement, the characteristics of the binders, and the mixing and curing conditions[116)] . In addition, the specifications of the construction machines widely vary, and 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.

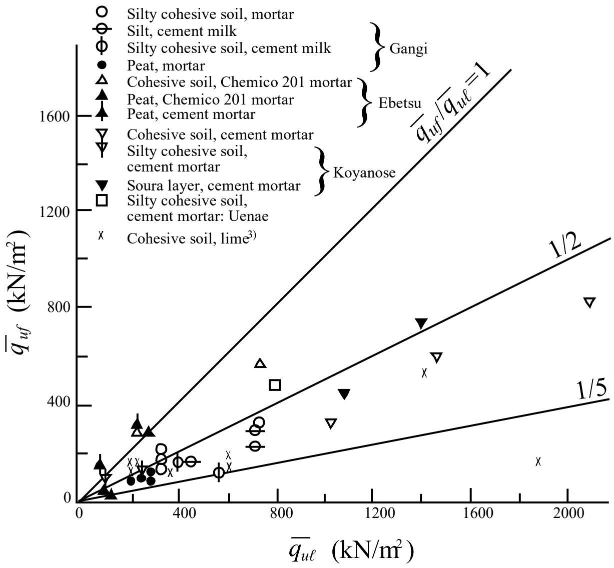
③ Because laboratory mix tests are for obtaining the strength of object soil for improvement under standard test conditions, they cannot be used for directly obtaining the strength of object soil for improvement under actual conditions. When predicting field strength from laboratory mix test results, 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.

$$f_{sh_k} = \frac{1}{2} f_{c_k} \tag{5.5.2}$$

$$f_{t_k} = 0.15 f_{c_k} \leq 200\text{kN/m}^2 \tag{5.5.3}$$

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.

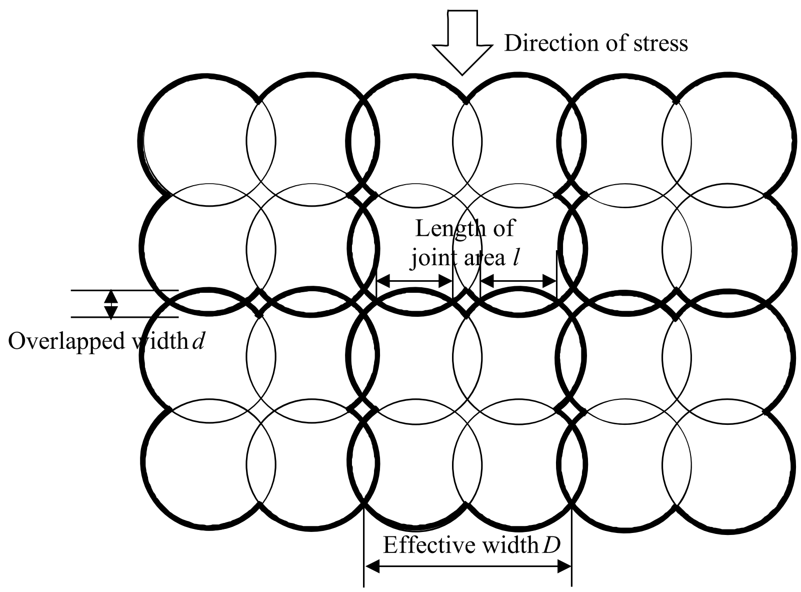


Fig. 5.5.4 Concept of the Factor for Effective Cross-Sectional Area α (When Using Four-Axis Construction Machines)

$$\alpha = \frac{N \cdot l}{D} \qquad \textbf{(5.5.4)}$$

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.

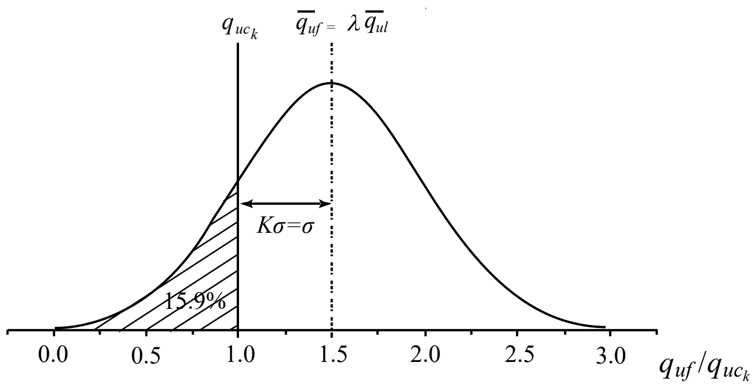


Fig. 5.5.5 Relationships of q_{uc_k} with $\overline{q_{uf}}$ and $\overline{q_{ul}}$ (Schematic Diagram)

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)] .

$$k_{h1_k} = 1.78\left(\frac{D_a}{D_r}\right)^{-0.55}\frac{\alpha_c \times 0.64}{g} + 0.04 \tag{5.5.7}$$

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).**

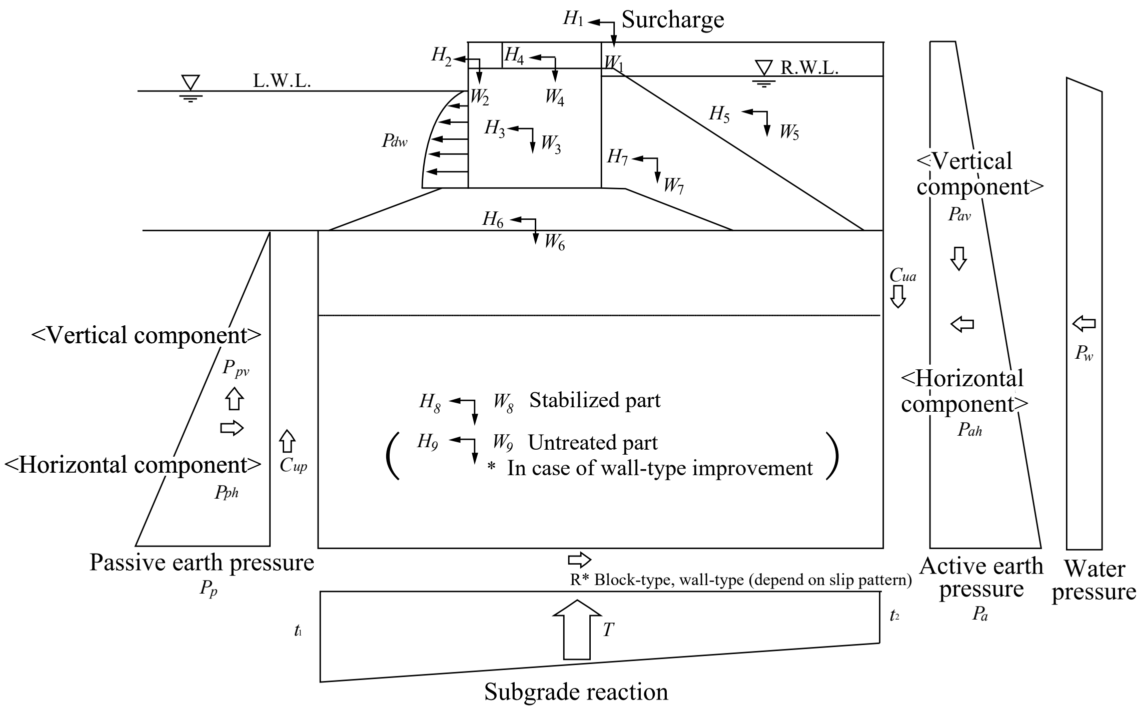


Fig. 5.5.6 Loads Acting on Stabilized Bodies

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.

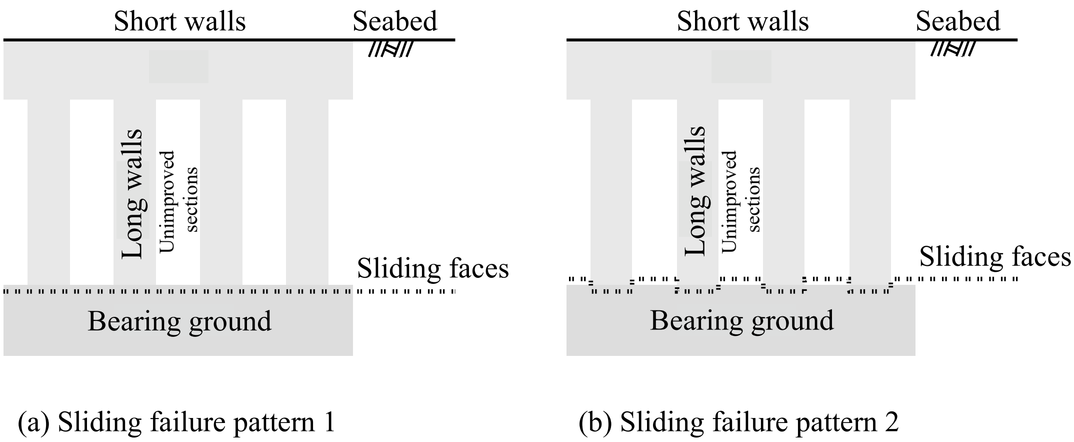


Fig 5.5.7 Sliding Failure Patterns

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k$$

$$R_k = P_{ph_k} + R_{1_k} + R_{2_k} \quad \text{(Sliding failure pattern 1)} \qquad R_k = P_{ph_k} + R_{1_k} + R_{3_k} \quad \text{(Sliding failure pattern 2)} \tag{5.5.8}$$

$$S_k = P_{ah_k} + P_{w_k} + P_{dw_k} + H_{i_k}$$

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.

$$
\left.
\begin{array}{ll}
m \cdot \dfrac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k & \\
R_k = P_{ph_k} y_p + \sum (W_{i_k} x_i) + W_{8_k} x_8 + W_{9_k} x_9 + P_{av_k} x_{av} + C_{ua_k} x_{C_{ua}} & \\
S_k = P_{ah_k} y_a + P_{w_k} y_w & \text{(Permanent situation)} \\
S_k = P_{ah_k} y_a + P_{w_k} y_w + P_{dw_k} y_{dw} + \sum H_{i_k} y_i & \text{(Variable situation in respect to Level 1 earthquake ground motions)}
\end{array}
\right\} \quad \textbf{(5.5.9)}
$$

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.

$$\left.\begin{array}{l} m \cdot \dfrac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\ \\ R_k = q_{ap_k} + q_{ar1_k} \ (\text{In the case of } \dfrac{1}{\eta} \geq 3),\ R_k = q_{ap_k} + q_{ar_k} \ (\text{In the case of } 1 \leq \dfrac{1}{\eta} < 3),\ S_k = t_{1_k}, t_{2_k} \end{array}\right\} \quad \textbf{(5.5.10)}$$

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

$$\left.\begin{array}{l} m \cdot \dfrac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\ \\ R_k = \dfrac{1}{2}\alpha\beta q_{uc_k} 、 \; S_k = \left(T_{\ell_k} - W_{\ell_k}\right)/A \end{array}\right\} \qquad \textbf{(5.5.12)}$$

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).

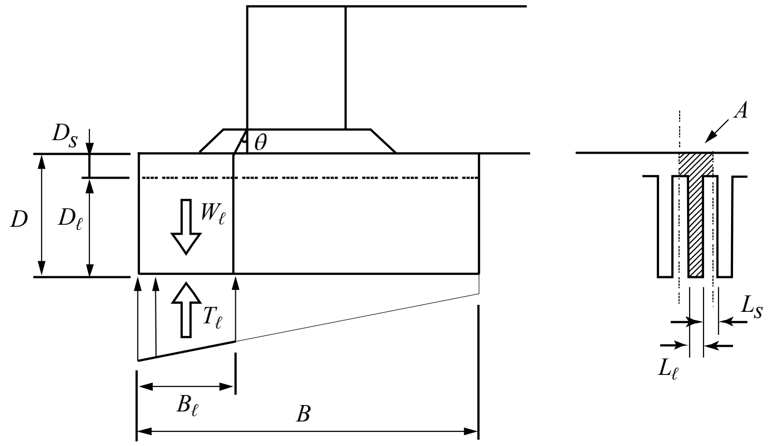


Fig. 5.5.8 Schematic Calculation Diagram of Vertical Shear Stress (Long Wall)

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

$$
\left.
\begin{array}{l}
m \cdot \dfrac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\[2ex]
R_k = \dfrac{1}{2}\alpha\beta q_{uc_k} \text{、} \quad S_k = \left(T'_{1_k} + w_{m_k} h_m + w_{i_k} D_s\right) L_s / \left(2D_s\right)
\end{array}
\right\} \qquad \textbf{(5.5.13)}
$$

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.

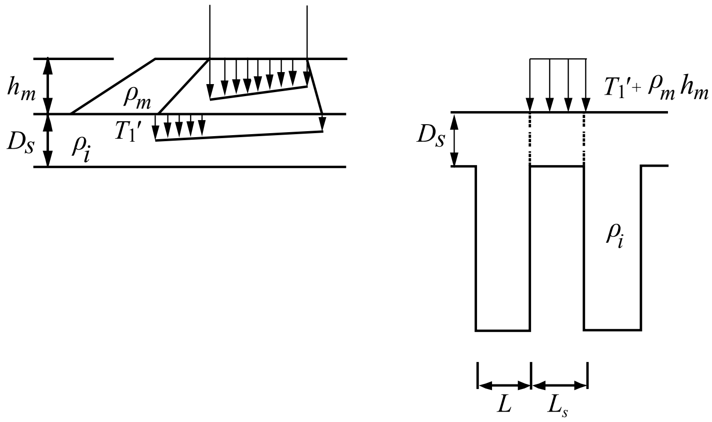


Fig. 5.5.9 Schematic Calculation Diagram of Vertical Shear Stress (Short Wall)

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.

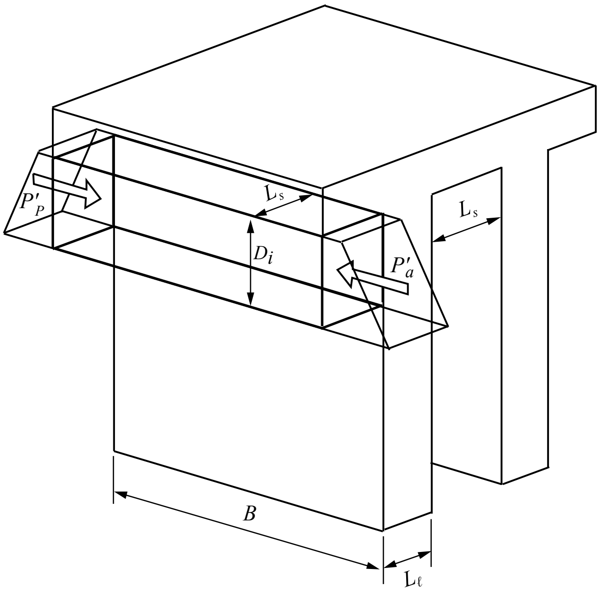


Fig. 5.5.10 Schematic Diagram of the Extrusion of Untreated Soil

(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.

$$\left.\begin{aligned} & m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\ & R_k = 2(L_s + D_i) C_{u_k} B + P_{ph}'{}_k \\ & S_k = P_{ah}'{}_k + k_{h2_k} w_{i_k} B D_i L_s + h_{wd} \rho_w g D_i L_s \end{aligned}\right\} \quad \textbf{(5.5.14)}$$

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**.

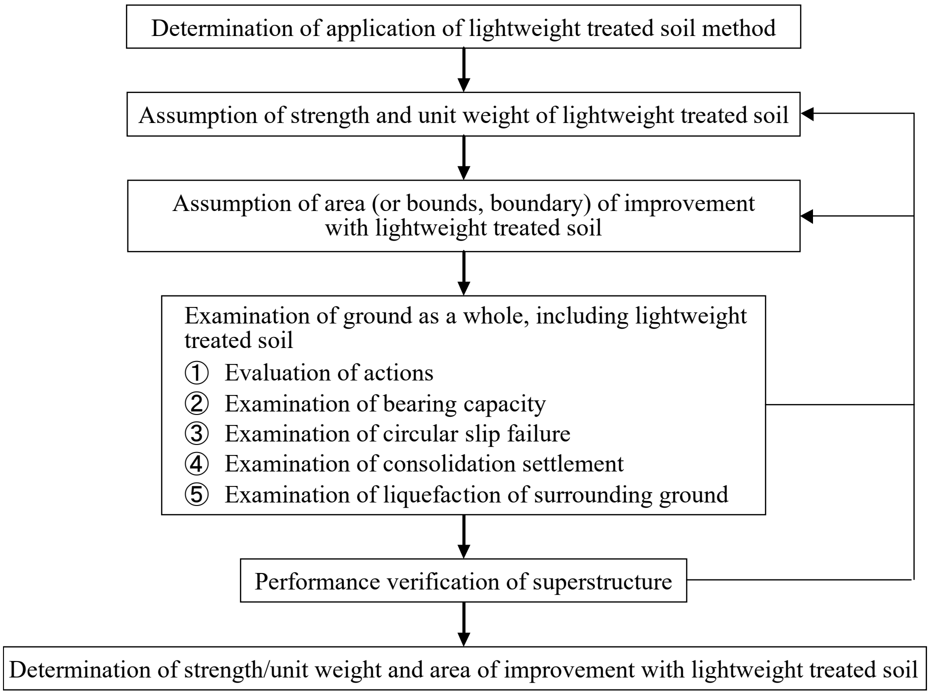


Fig. 5.6.1 Example of the Performance Verification Procedure for the Lightweight Treated Soil Method

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)**.

$$c_u = q_u/2 \tag{5.6.1}$$

(c) Consolidation yield stress p_y

The consolidation yield stress p_y can be calculated by the **equation (5.6.2)**.

$$p_y = 1.4q_u \tag{5.6.2}$$

(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)**.

$$E_{50} = 100\sim 200q_u \tag{5.6.3}$$

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: $\nu = 0.10$

Lightweight treated soil with expanded beads: $\nu = 0.15$

(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 ν 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**).

$$\tau_f = c + \sigma' \tan\phi \qquad \textbf{(5.8.1)}$$

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**.

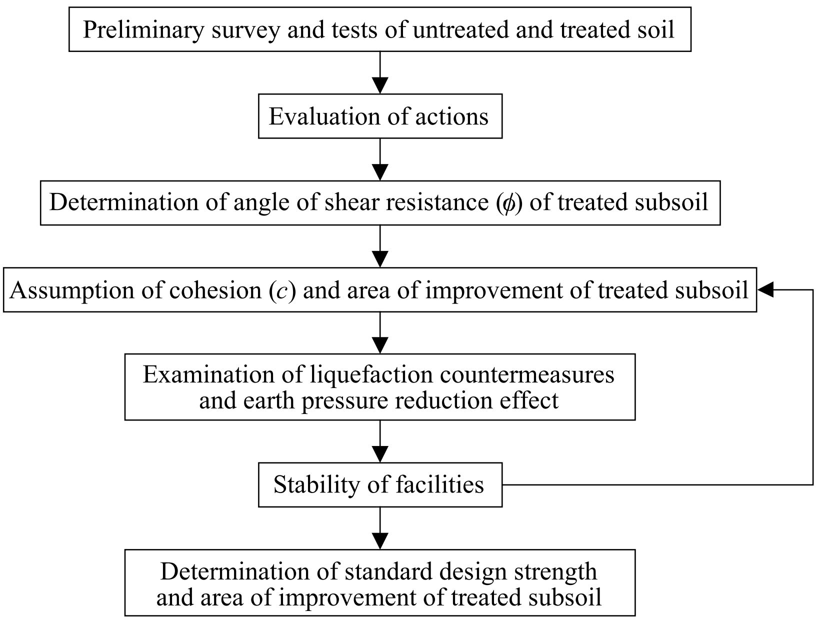


Fig. 5.8.1 Example of the Performance Verification Procedure for the Premix Method

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.

$$\left.\begin{array}{l} m\cdot\dfrac{S_d}{R_d}\leq 1.0 \qquad R_d=\gamma_R R_k \qquad S_d=\gamma_S S_k \\ R_k=R_{1_k}+R_{2_k}+P_{w1_k} \\ S_k=H_{1_k}+H_{2_k}+P_{h_k}+P_{w2_k}+P_{w3_k} \end{array}\right\} \quad \textbf{(5.8.2)}$$

The characteristic values in the above equation can be calculated as follows.

$$\left.\begin{array}{l} R_{1_k}=f_{1_k}W_1{'}_k \\ R_{2_k}=f_{2_k}(W_2{'}_k - P_{v_k}) \text{ (when the original ground below the treated ground is sandy soil)} \\ R_{2_k}=c_k l_{bc} \quad \text{(when the original ground below the treated ground is cohesive soil)} \\ P_{w1_k}=\dfrac{1}{2}\rho w g h_1{}^2 \\ P_{w2_k}=\dfrac{7}{12}k_{h_k}\rho_w g h_1{}^2 \\ P_{w3_k}=\dfrac{1}{2}\rho_w g h_2{}^2 \\ H_{1_k}=k_{h_k}W_{1_k} \\ H_{2_k}=k_{h_k}W_{2_k} \\ P_{h_k}=\dfrac{1}{2}K_a w'_k\, h_{2_k}{}^2\dfrac{\cos(\delta_k+\phi)}{\cos\phi} \\ P_{v_k}=P_{h_k}\tan(\delta_k+\phi) \end{array}\right\} \quad \textbf{(5.8.3)}$$

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.

$$\left.\begin{aligned} & m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \\ & R_k = R_{1_k} + R_{2_k} + P_{w1_k} \\ & S_k = H_{1_k} + H_{2_k} + P_{h_k} + P_{w2_k} \end{aligned}\right\} \qquad \textbf{(5.8.4)}$$

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.

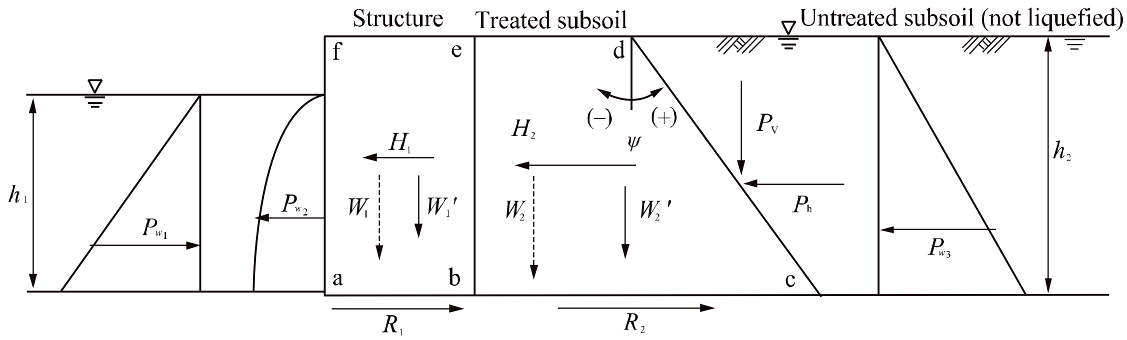


Fig. 5.8.2 Diagram of Actions to be Considered When Implementing the Premix Method for Reducing Earth Pressure

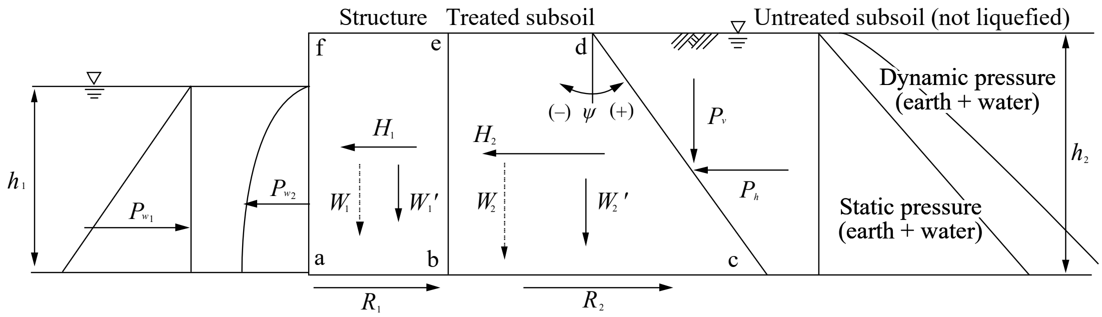


Fig. 5.8.3 Diagram of Actions to be Considered When Implementing the Premix Method for Liquefaction Countermeasures

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.

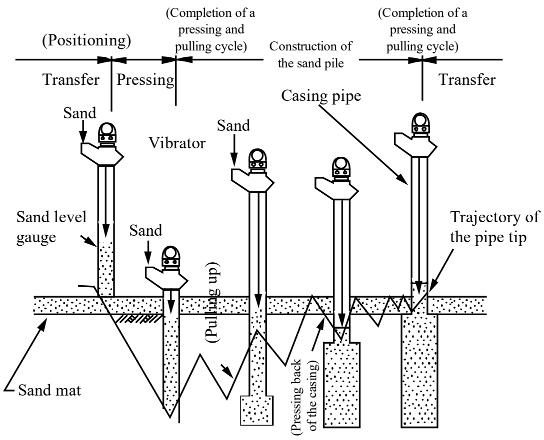
① The properties of the object soil for improvement (grain size distribution and fine particle contents (grain diameters less than 75 μm))
② The degrees of saturation and the positions of groundwater levels
③ The relative density of the object soil before improvement
④ The initial stresses in the object 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

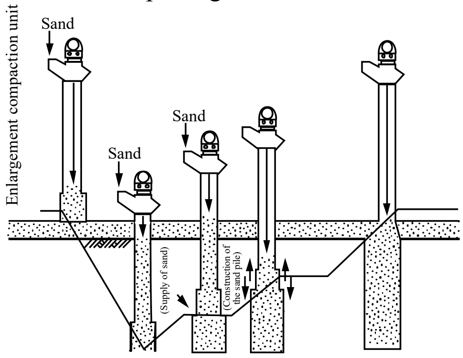
The variations of the sand compaction pile method are largely classified into: (a) sand pile formation by vibro-driving and vibro-removal; (b) expanding bottom diameter type; and (c) bottom vibration type. The characteristics of the respective variations are shown in **Table 5.9.1** and **Fig. 5.9.1**. 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

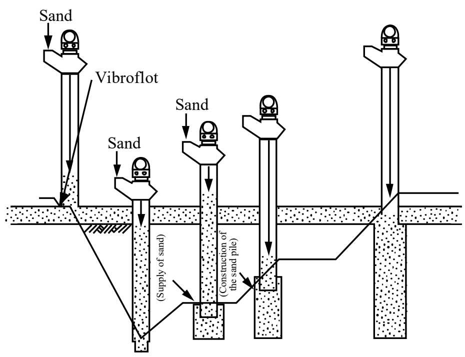
Type	Characteristics
(a) Sand pile formation by vibro-driving and vibro-removal	Typical and most frequently used sand compaction pile method for constructing sand piles by repeatedly pressing and pulling vibrated casing pipes in the ground.
(b) Expanding bottom diameter type	A method that constructs 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

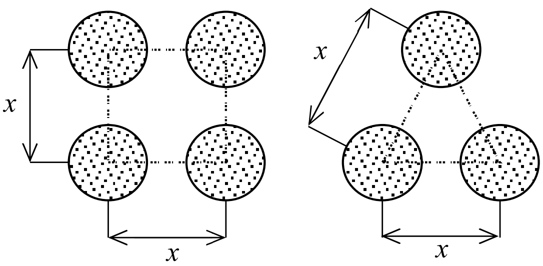
F_V : sand supply ratio

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 $\kappa = 5 \times 10^{-0.01Fc}$

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 (%)

γ_i^* : coefficient which can be calculated by the **equation** (**5.9.4**).

$$\gamma_i^* = \frac{c\sqrt{N_0/(AC_M)}}{1-\sqrt{N_0/(AC_M)}} \tag{5.9.4}$$

where

N_0 : N-value of the original ground

A : coefficient which can be calculated by the **equation** (**5.9.5**).

$$A = \frac{69 + \sigma_v'}{167} \tag{5.9.5}$$

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.

$$F_V = \frac{(c+\gamma_i^*)\sqrt{N_1/(AC_M)} - \gamma_i^*}{\kappa\left\{1-\sqrt{N_1/(AC_M)}\right\}} \tag{5.9.6}$$

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.

$$N_1 = C_M\left(\frac{\kappa F_V + \gamma_i^*}{c + \kappa F_V + \gamma_i^*}\right)^2 A_{K_1} \tag{5.9.7}$$

where

C_M : coefficient which can be calculated by $C_M = (1/0.16)^2$

κ : coefficient which can be calculated by $\kappa = 4 \times 10^{-0.01Fc}$

c : coefficient which can be calculated by $c = \dfrac{0.02F_c + 0.4}{0.02F_c + 2.0}$

γ_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)**.

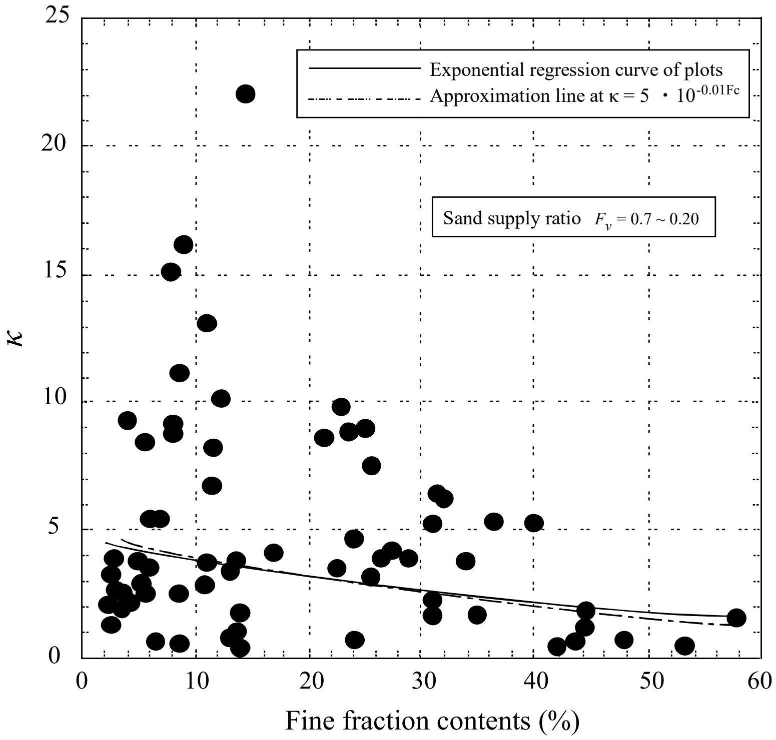


Fig. 5.9.3 Relationship between κ and Fine Particle Contents

(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.

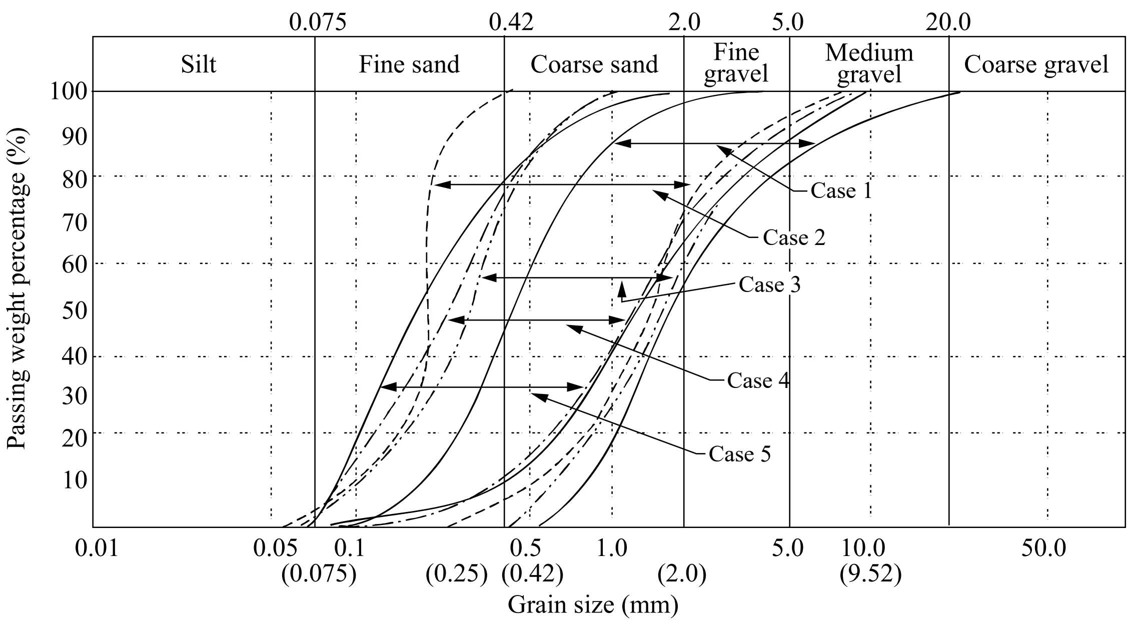


Fig. 5.10.1 Examples of the Grain Size Distribution Ranges of Sand used in the Actual Implementation of the SCP method

(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

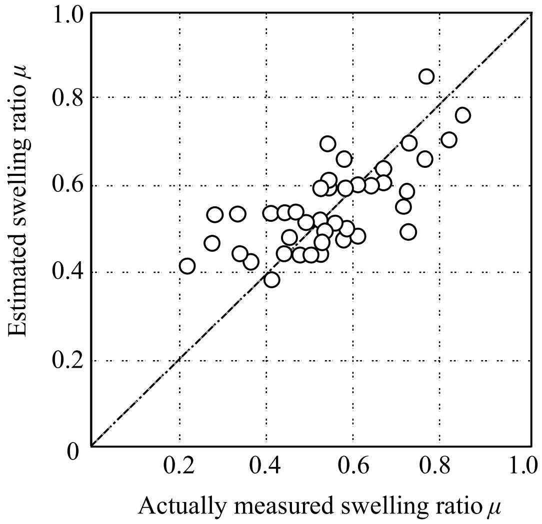


Fig. 5.10.2 Comparison of Estimated Heaving Ratios by the **equation (5.10.1)** and Actually Measured Heaving Ratios[184)]

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.

$$\mu = 0.718 a_s + 2.117 L^{-1} + 0.056 \tag{5.10.2}$$

where

a_s : replacement area ratio

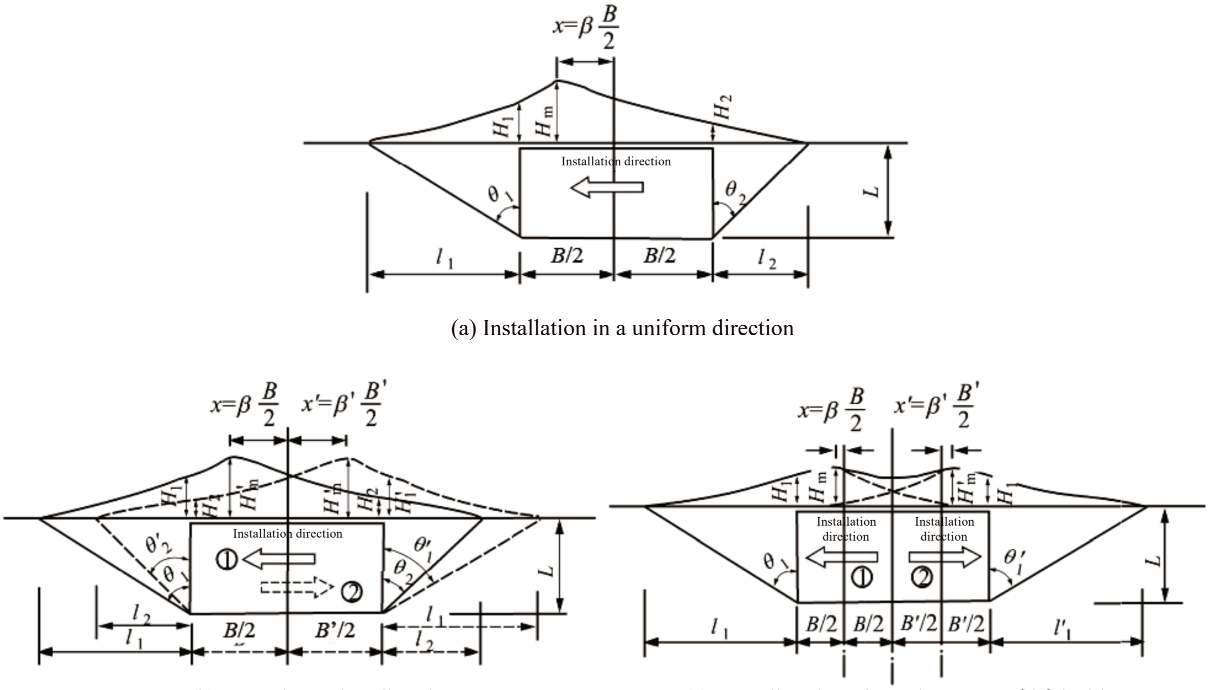
L : average length of the sand piles (m)

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.

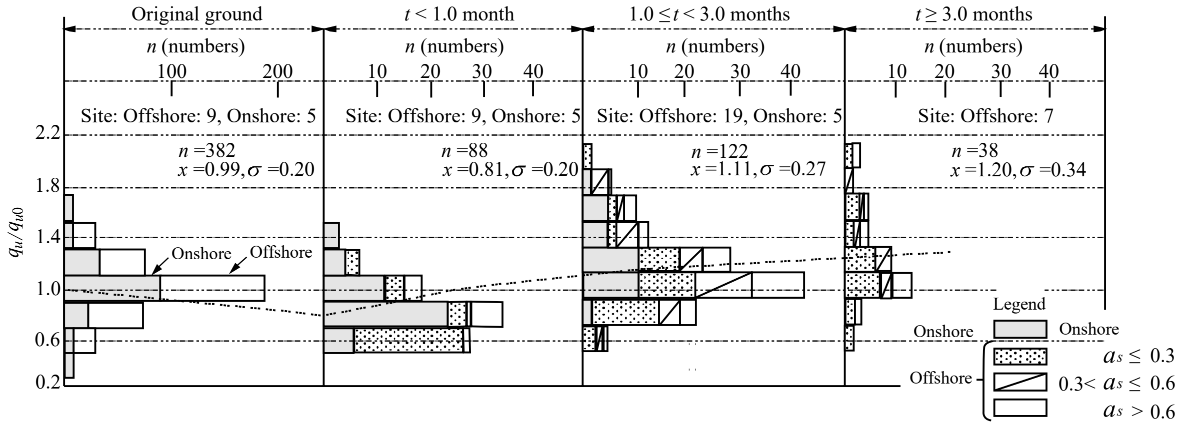


Fig. 5.10.4 Disturbance and Restoration of Cohesive Soil in Areas of Improvement (between Sand Piles)[188)]

(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**).

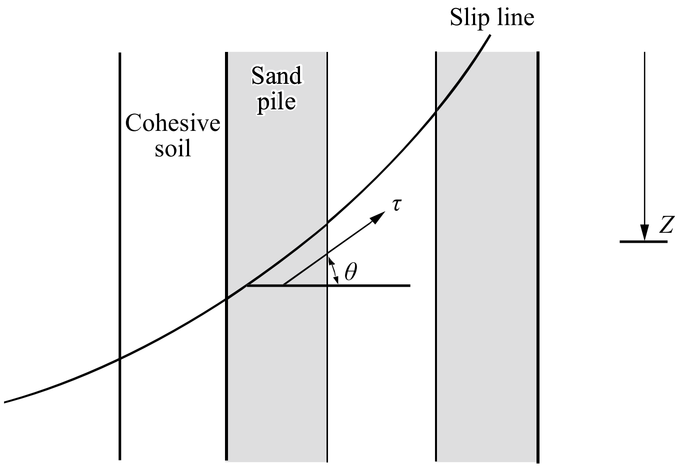


Fig. 5.10.5 Shear Strength of Composite Ground

$$\tau = (1 - a_s)(c_0 + kz + \Delta\sigma_z \mu_c (\Delta c/\Delta p)U) + (w_s z + \mu_s \Delta\sigma_z) a_s \tan\phi_s \cos^2\theta \quad \textbf{(5.10.3)}$$

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.

$$\tau = (1 - a_s)(c_0 + kz) + (w_m z + \Delta\sigma_z)\mu_s a_s \tan\phi_s \cos^2\theta \tag{5.10.4}$$

$$\tau = (w_m z + \Delta\sigma_z)\tan\phi_m \cos^2\theta \tag{5.10.5}$$

$$\tau = (w_m z + \Delta\sigma_z)\mu_s a_s \tan\phi_s \cos^2\theta \tag{5.10.6}$$

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)

$$\sum_i M_{R_i} = \sum r s \bar{\tau}_k \sec\theta$$

r : radius of a slip circle (m)

s : width of a sliced piece (m)

θ : angle between a slip surface and a horizontal plane (°)

$\overline{\tau}$: 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**).

$$\overline{\tau}_k = \left(1 - a_s\right)\left\{c'_k + kz + \Delta\sigma_Z \mu_c (\Delta c / \Delta p) U\right\} + \left(w_{s_k} z + \mu_s \Delta\sigma_z\right) a_s \tan\phi_{s_k} \cos^2\theta \tag{5.10.9}$$

③ **Fig. 5.10.6** shows a schematic diagram of a slip circle failure.

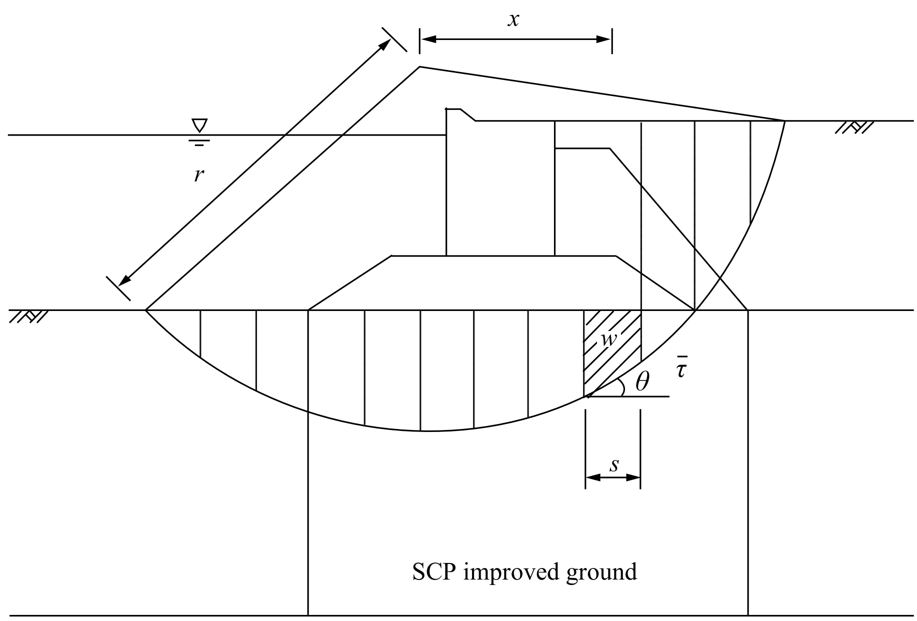


Fig. 5.10.6 Schematic Diagram of a Slip circle Failure

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.

$$\begin{aligned}
&S_a \geq S_f \\
&S_f = \beta S_{f_0} \\
&\quad S_{f_0} = m_v(p_0' + \alpha\gamma' h - p_c')H(1-U) \\
&\quad S_{f_0} = \frac{\Delta e}{1+e_0}H(1-U) \\
&\quad S_{f_0} = \frac{C_c}{1+e_0}H\left(\log_{10}\frac{p_0' + p'}{p_0'}\right)(1-U)
\end{aligned} \qquad \textbf{(5.10.10)}$$

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.

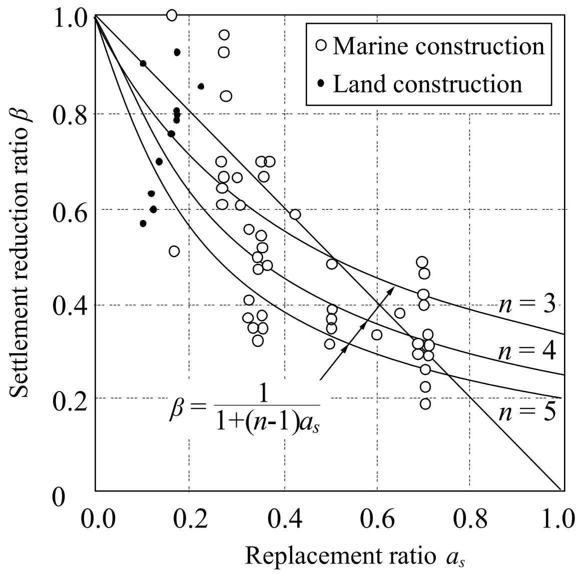


Fig. 5.10.7 Relationship between Settlement Reduction Ratios and Replacement area ratios[189)]

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.

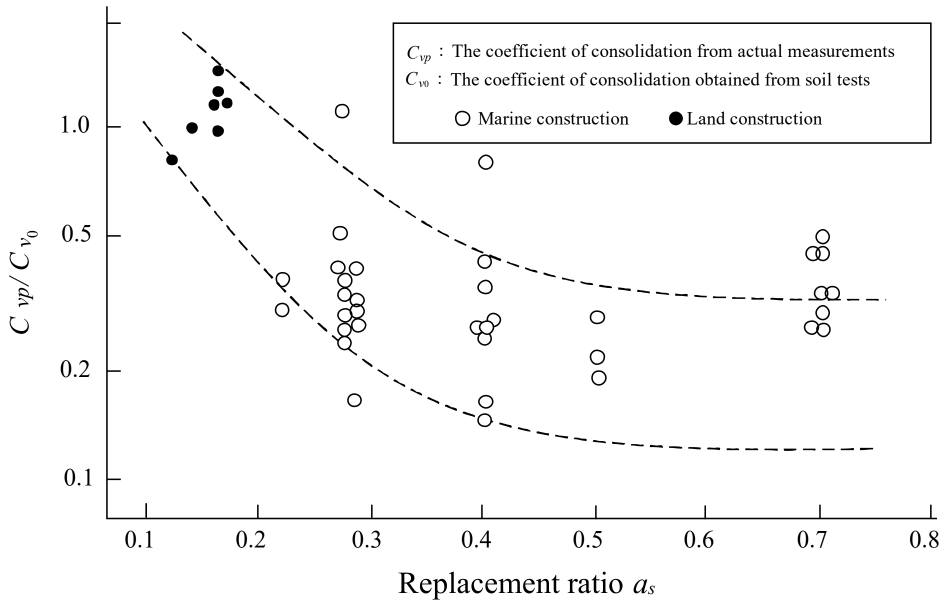


Fig. 5.10.8 Low Consolidation Degrees of Ground Improved through the SCP Method[195)]

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

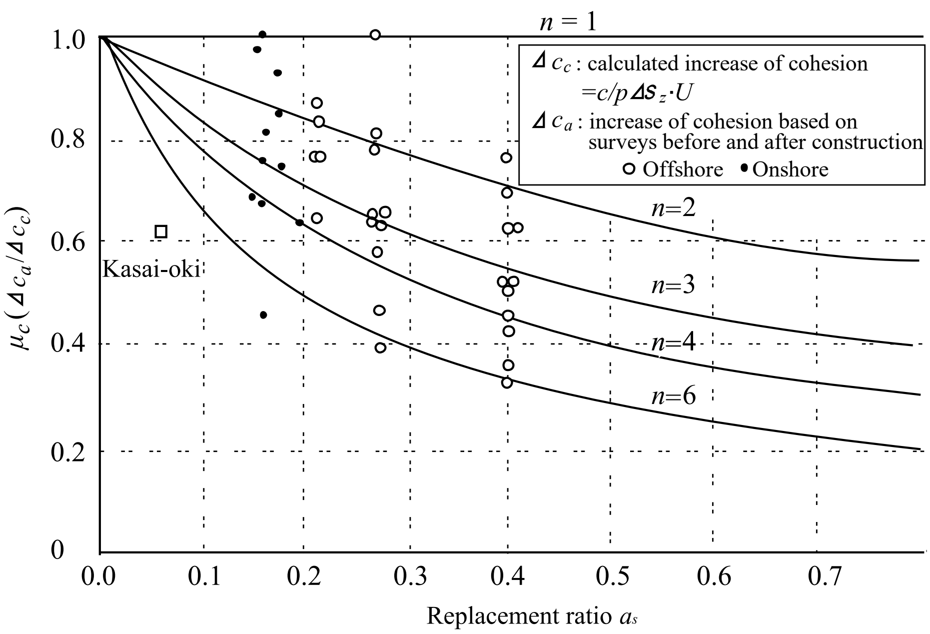


Fig. 5.10.9 Increases in the Strength of Cohesive Soil between Piles in Improved Ground[189]

(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.

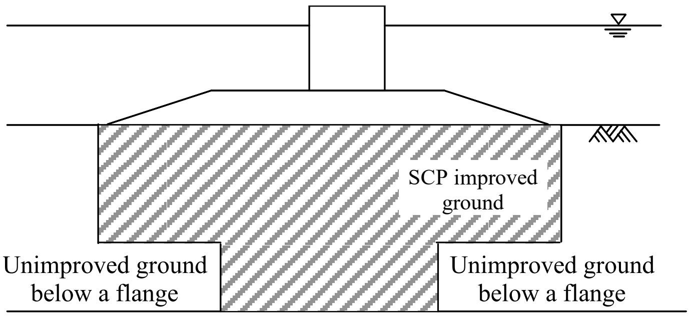


Fig. 5.10.10 T-shaped SCP Method

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.

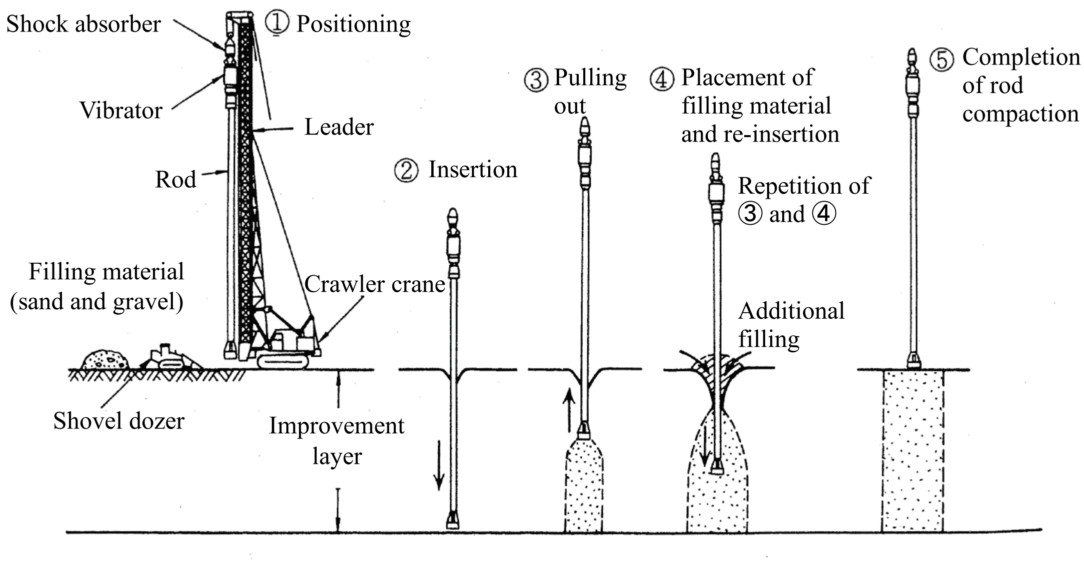


Fig. 5.11.1 Schematic Drawing of the Implementation Procedure of the Rod Compaction Method[201]

(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)] .

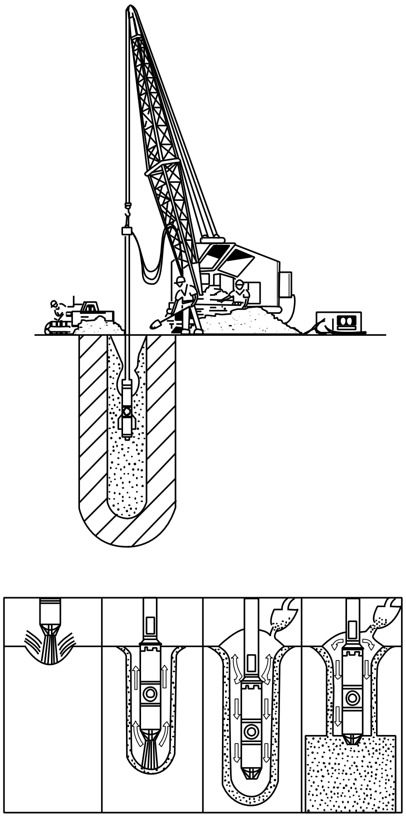


Fig. 5.12.1 Schematic Drawing of the Implementation Procedure of the Vibro-Flotation Method Alteration of 204)

(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.

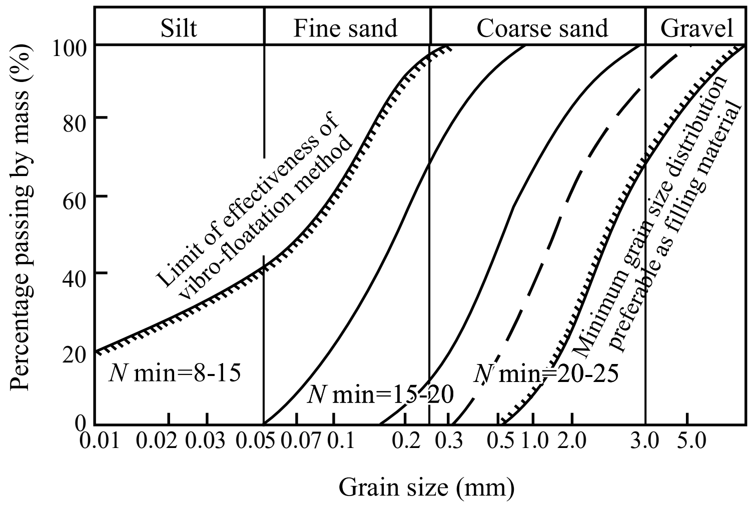


Fig. 5.12.2 Relationship between the Grain Sizes of Original Ground and the Minimum *N*-Values[204)] (in the Case of Sandy Ground Improvement)

(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)**.

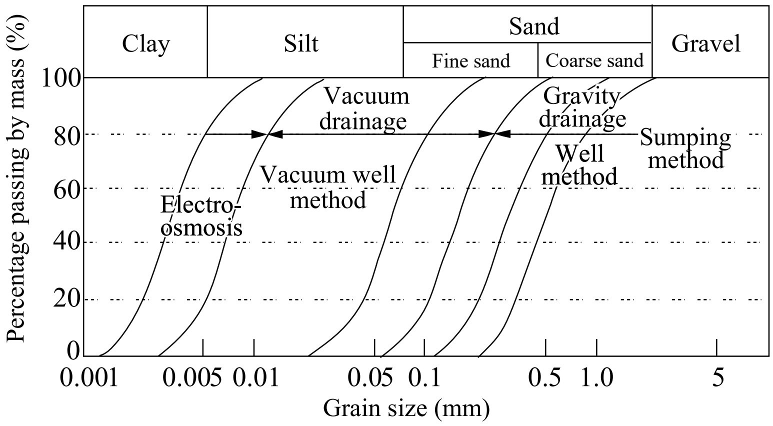


Fig. 5.14.1 Applicability of the Well Point Method by Soil Grain Size[211)]

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**.

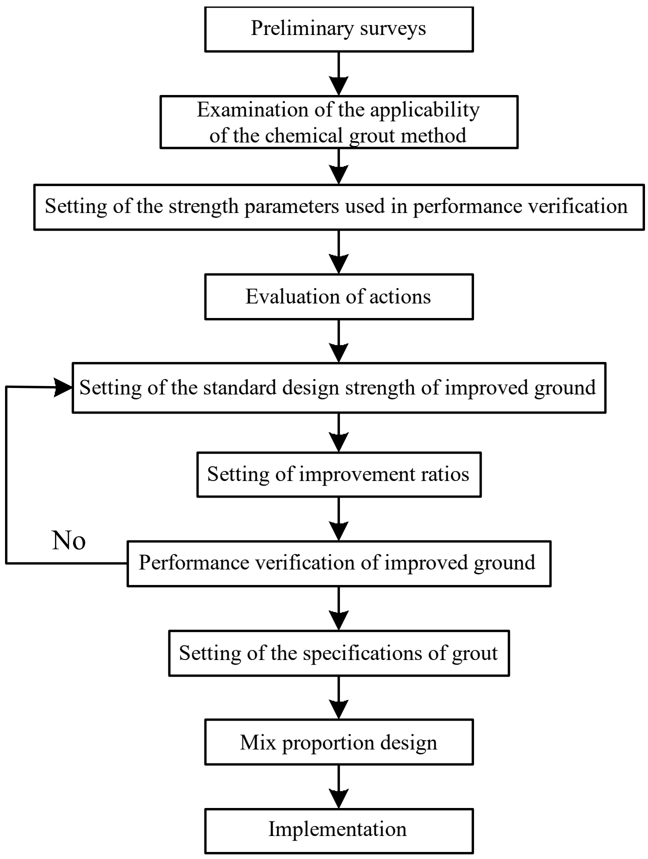


Fig. 5.16.1 Procedures of Performance Verification

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**).

$$p_{a_i}=\left\{\frac{\left(\sum\gamma_i h_i\right)\cos(\psi-\beta)}{\cos\psi}+\omega\right\}\frac{\sin(\zeta_i-\phi_i+\theta)\cos(\psi-\zeta_i)}{\cos\theta\cos(\psi-\zeta_i+\phi_i+\delta)\sin(\zeta_i-\beta)}-\frac{c_i\cos(\psi-\beta)\cos\phi_i}{\cos(\psi-\zeta_i+\phi_i+\delta)\sin(\zeta_i-\beta)}$$

$$2\zeta_i=\psi+\phi_i-\mu_i+90°$$

$$\mu_i=\tan^{-1}\frac{B_iC_i+A_i\sqrt{B_i^{\,2}-A_i^{\,2}+C_i^{\,2}}}{B_i^{\,2}-A_i^{\,2}}$$

$$A_i=\sin(\delta+\beta+\theta) \qquad \textbf{(5.18.1)}$$

$$B_i=\sin(\psi+\phi_i+\delta-\beta)\cos\theta-\sin(\psi-\phi_i+\theta)\cos(\delta+\beta)+\frac{2c_i\cos(\psi-\beta)\cos\phi_i\cos(\delta+\beta)\cos\theta}{\dfrac{\left(\sum\gamma_i h_i\right)\cos(\psi-\beta)}{2\cos\psi}+\omega}$$

$$C=\sin(\psi+\phi+\delta-\beta)\sin\theta+\sin(\psi-\phi+\theta)\sin(\delta+\beta)-\frac{2c\cos(\psi-\beta)\cos\phi\sin(\delta+\beta)\cos\theta}{\dfrac{\left(\sum\gamma h\right)\cos(\psi-\beta)}{2\cos\psi}+\omega}$$

where

p_{ai} : 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

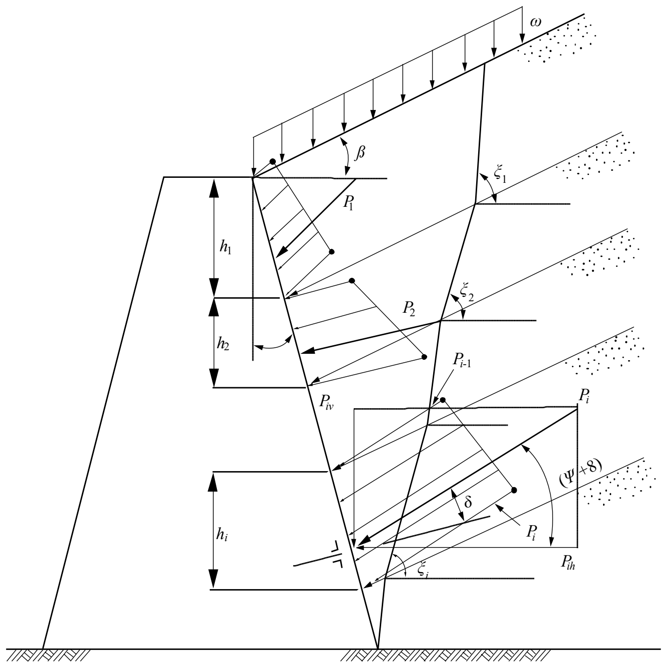


Fig. 5.18.1 Active Earth Pressure Acting on a Structure

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.

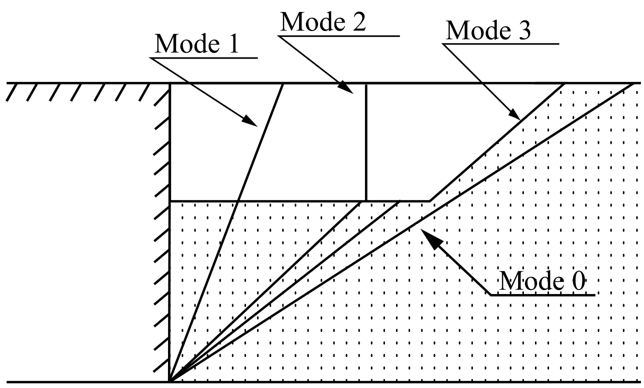


Fig 5.18.2 Three Failure Modes Considered in the Slice Method

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.

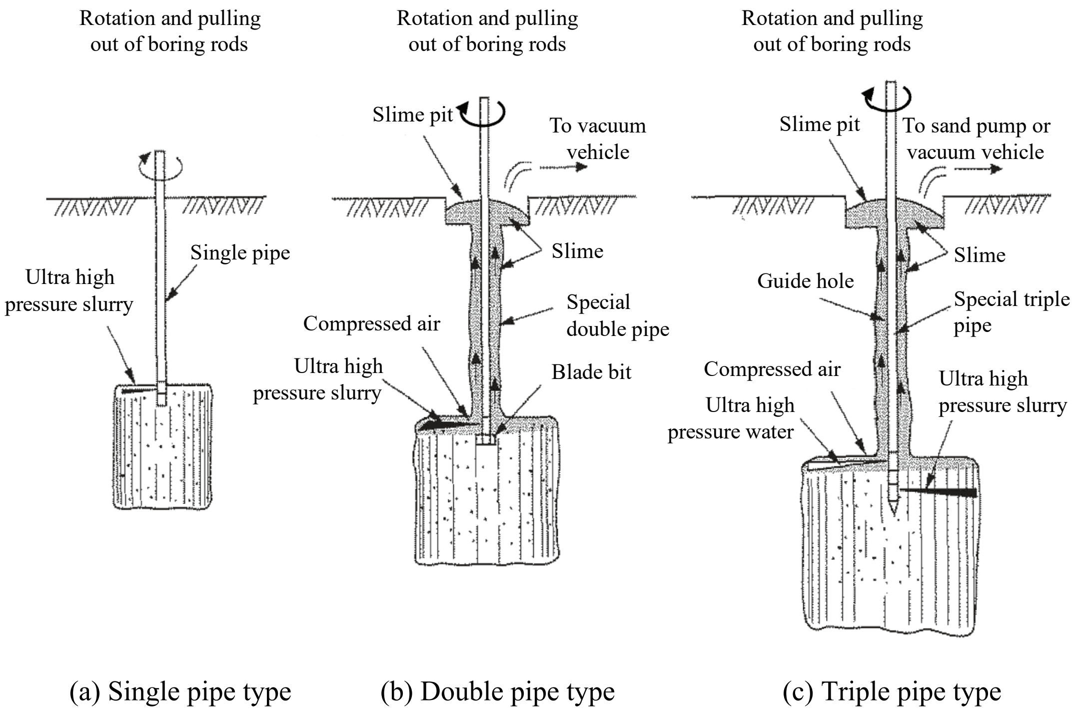


Fig. 5.19.1 Classification of the Types of Jet Grouting Method [221)]

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.

[References]

1) Japanese Geotechnical Society: Handbook of Geotechnical Engineering, pp.1197-1262, 1999 (in Japanese)

2) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.13, 2013 (in Japanese)

3) Industrial Technology Service Center: Compendium of practical measures for soft ground, pp.439-454, 1993 (in Japanese)

4) Coastal Development Institute of Technology: Technical Manual of deep mixing method for ports and airports, p.116, 2014 (in Japanese)

5) Ministry of Land, Infrastructure, Transport and Tourism: Recycling Guidelines for Port and Airport Development (revised version), pp.1-10 - 1-29, 2015 (in Japanese)

6) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.47, 2013 (in Japanese)

7) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.57, 2013 (in Japanese)

8) Japanese Geotechnical Society: Geotechnical Engineering Handbook, Japanese Geotechnical Society, p.1210, 1999 (in Japanese)

9) Matsuo, M., Tsukada, J., Kanaya, Y. and Syouno, H.: Trial execution of ground improvement to construct Yokkaichi LNG terminal, JSCE Magazine Civil Engineering, Vol.69, No.4, pp.9-15, 1984 (in Japanese)

10) Kitazume, M., Terashi, M., Aihara, N. and Katayama, T.: Applicability of fabric-packed sand drain for extremely soft clay ground, Report of the Port and Harbour Research Institute, Vol.32, No.1, pp.101-123, 1993 (in Japanese)

11) Japanese Geotechnical Society: Research committee of ground investigation for ultra-soft clay ground and ground improvement, pp.268-271, 2002 (in Japanese)

12) Japanese Geotechnical Society: Geotechnical Engineering Handbook, Japanese Geotechnical Society, p.1212, 1999 (in Japanese)

13) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.90, 2013 (in Japanese)

14) Kobayashi, M. and Tsuchida, T.: Field test of the vacuum consolidation in Kinkai Bay, Technical Note of Port and Harbour Research Institute, No.476, 28p., 1984 (in Japanese)

15) Soil Stabilization Committee, Japan Lime Association: Soft soil stabilization method by lime, pp.92-93, 1983 (in Japanese)

16) Japanese Society of Soil Mechanics and Foundation Engineering: Countermeasure works for soft ground- survey, design and construction-, pp.329-330, 1990 (in Japanese)

17) Kitazume, M.: the Sand Compaction Pile Method, A. A. Balkema Publishers, p.1, 2005

18) Japanese Geotechnical Society: Design and Execution of Sand Compaction Pile Method using Vibro-driving and Vibro-removal, p.1, 2009 (in Japanese)

19) Kitazume, M. and Terashi, M.: The Deep Mixing Method, CRC Press, pp.17-19, 2012

20) Coastal Development Institute of Technology: Technical manual of deep mixing method for ports and airports, p.2, 2014 (in Japanese)

21) Public Works Research Center: Design and construction manual of deep mixing method on land (Revised Version), p.2, 2004 (in Japanese)

22) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.145, 2013 (in Japanese)

23) Coastal Development Institute of Technology: Technical manual of light weight treated soil method for ports and airports (Revised Edition), p.3, 2014 (in Japanese)

24) Coastal Development Institute of Technology: Technical manual for pneumatic flow mixing method (Revised Edition), p.2, 2008 (in Japanese)

25) Sato, T.: Development and application of pneumatic flow mixing method to reclamation for offshore airport, Technical Note of Port and Harbour Research Institute, No.1076, 81p., 2004 (in Japanese)

26) Japanese Geotechnical Society: Geotechnical Engineering Handbook, Japanese Geotechnical Society, p.1186, 1999 (in Japanese)

27) Japanese Geotechnical Society: Geotechnical Engineering Handbook, Japanese Geotechnical Society, p.1223, 1999 (in Japanese)

28) Coastal Development Institute of Technology (CDIT): Handbook for countermeasure works for reclaimed land (Revised Edition), pp.170-192, 1997 (in Japanese)

29) Yamamoto, M. and Nozu, M.: Quiet compaction of sandy round- no vibration and low noise static compaction method for sand pile, JSCE Magazine Civil Engineering, Vol.83, No.8, pp.19-21, 1998 (in Japanese)

30) Kakehashi, T., Umeki, Y., Ookori, K., Goami, Y., Makibe, Y. Sato, Y.: Introduction of “KS-EGG Method”: Low vibration and low noise soil improvement method, Proceedings of 52nd Conference of JSCE, Section III, pp.151-152, 2002 (in Japanese)

31) Kato, M., Tanaka, Y., Ichikawa, H. and Mishiro, N.: “Geo-KONG Method”; Low vibration and low noise ground compaction method, Foundation Engineering & Equipment, Vol.31, No.12, pp38-41, 2003 (in Japanese)

32) Japanese Geotechnical Society: Design and Execution of Sand Compaction Pile Method using Vibro-driving and Vibro-removal, pp.122-130, 2009 (in Japanese)

33) Soft ground handbook editing committee: New soft ground Handbook for Civil and architectural engineers, p.387, 1989 (in Japanese)

34) Ishiguro, K. and Shimizu, H.: Examination of better countermeasure for liquefaction, JSCE Magazine Civil Engineering, VoL83, No.5, pp,17-19, 1998 (in Japanese)

35) Japanese Geotechnical Society: Geotechnical Engineering Handbook, Japanese Geotechnical Society, pp.1228-1229, 1999 (in Japanese)

36) Brown, R. E.: Vibroflotation compaction of cohesionless soils, Proc. ASCE, Vol.103, No.GT12, pp.1437-1451, 1977

37) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.113, 2013 (in Japanese)

38) Tsuchiya, H.: Compaction by Heavy Tamping, Soil mechanics and foundation engineering, Vol.39, No.2, pp.62-66, 1991 (in Japanese)

39) Narumi, N., Nomura, A. and Ikeda, M.: Improvement of solid waste ground by heavy tamping method, Soil mechanics and foundation engineering, Vol.40, No.6, pp.49-52, 1992 (in Japanese)

40) Suzuki, Y., Saitou, S., Onimaru, S., Kimura, T., Uchida, A. and Okumura, R.: Countermeasure work for liquefaction by grid soil improvement by deep mixing method, Soil mechanics and foundation engineering, Vol.44, No.3, pp.46-48, 1996 (in Japanese)

41) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.148, 2013 (in Japanese)

42) Japanese Society of Soil Mechanics and Foundation Engineering: Survey, design and construction of Chemical Grouting Method, p.1, 1985 (in Japanese)

43) Coastal Development Institute of Technology (CDIT): Technical Manual for osmotic solidification method, pp.1-2, 2010 (in Japanese)

44) Public Works Research Institute, Ministry of Construction: Design and Construction Manual for countermeasure work for liquefaction (draft), pp.364-374, 1999 (in Japanese)

45) Japanese Geotechnical Society: Countermeasure works for liquefaction, Geotechnical Engineering, Practical Business Series, pp.326, 2004 (in Japanese)

46) Coastal Development Institute of Technology: Technical Manual for premixing-type stabilization method (Revised Edition), p.1, 2008 (in Japanese)

47) Public Works Research Center: Design and construction Manual for Reinforces Soil “Terre Armee” wall method, pp.3-5, 2003 (in Japanese)

48) Coastal Development Institute of Technology (CDIT): Handbook for countermeasure works for reclaimed land (Revised Edition), pp. 230-237, 1997 (in Japanese)

49) Coastal Development Institute of Technology (CDIT): Handbook for countermeasure works for reclaimed land (Revised Edition), pp.285-294, 1997 (in Japanese)

50) Japanese Geotechnical Society: Geotechnical Engineering Handbook, Japanese Geotechnical Society, p.1220, 1999 (in Japanese)

51) Japanese Society of Soil Mechanics and Foundation Engineering: Case history collection of soil engineering, Vol.1, pp.379-480, 1978 (in Japanese)

52) Japanese Society of Soil Mechanics and Foundation Engineering: Causes and countermeasure for problems of ground improvement, 253p., 1993 (in Japanese)

53) Noda, S. and Uwabe, T.: Seismic disasters of gravity quaywalls, Technical Note of the Port and Harbour Research Institute Ministry of Transport, No.227, 159p., 1975 (in Japanese)

54) Tsuchida, H., Inatomi, T., Noda, S., Yagyu, T., Tabata, T., Tokunaga, S., Otsuki, Y. and Hirano, T.: The damage to port structures by the 1978 Miyagi-Ken-oki Earthquake, Technical Note of the Port and Harbour Research Institute Ministry of Transport, No.325, 175p., 1979 (in Japanese)

55) Tsuchida, H., Inatomi, T. and Ueda, H.: The damage to port facilities by the 1982 Urakawa-Oki Earthquake, Technical Note of the Port and Harbour Research Institute Ministry of Transport, No.472, 47p., 1983 (in Japanese)

56) Tsuchida, H., Noda, S., Inatomi, T., Uwabe, T., Iai,S., Ohneda, H. and Toyama, S.: Damage to structures by the 1983 Nipponkai-Chubu Earthquake, Technical Note of the Port and Harbour Research Institute Ministry of Transport, No.511, 447p., 1985 (in Japanese)

57) Ueda, S., Inatomi, T., Uwabe, T., Iai, S., Kazama, M., Nagamatsu, Y., Hujimoto, T., Kikuchi, Y., Miyai, S., Sekiguchi, S. and Hujimoto, Y.: Damage to port structures by the 1993 Kusiro-Oki Earthquake, Technical Note of the Port and Harbour Research Institute Ministry of Transport, No.766, 454p., 1993 (in Japanese)

58) Inatomi, T., Uwabe, T., Iai, S., Kazama, M., Yamazaki, H., Nagamatsu, Y., Sekiguchi, S., Mizuno, Y. and Hujimoto, Y.: Damage to port facilities by the 1993 Hokkaido-nansei-oki Earthquake, Technical Note of the Port and Harbour Research Institute Ministry of Transport, No.791, 449p., 1994 (in Japanese)

59) Yamazaki, H., Zen, K., Sado, A. and Tachishita, T.: Mechanism of damage to port facilities during 1995 Hyogo-Ken Nanbu Earthquake. (Part 5) Liquefaction Potential of Reclaimed Land, Technical Note of the Port and Harbour Research Institute, Ministry of Transport, No.813, pp.167-205, 1995 (in Japanese)

60) Inatomi, T., Uwabe, T., Iai, S., Tanaka, S., Yamazaki, H., Miyai, S., Nozu, A., Miyata, M. and Fujita, Y.: Damage to port structures by the 1994 east off Hokkaido Earthquake, Technical Note of the Port and Harbour Research Institute Ministry of Transport, No.856, 583p., 1997 (in Japanese)

61) Sugano, T., Nozue, Y., Tanaka, T., Nozu, A., Kohama, E., Hazarika, H. and Motono, I.: Damage to port facilities by the 2005 west off Fukuoka Prefecture, Technical Note of the Port and Airport Research Institute, No.1165, 127p., 2007 (in Japanese)

62) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.78-79, 2013 (in Japanese)

63) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.79, 2013 (in Japanese)

64) Japanese Geotechnical Society: Geotechnical and Geoenvironmental Investigation Methods, pp823-996, 2013 (in Japanese)

65) Japanese Society of Soil Mechanics and Foundation Engineering: Countermeasure works for soft ground- survey, design and construction-, pp.351-383, 1990 (in Japanese)

66) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, pp.37-44, 2013 (in Japanese)

67) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, pp.65-70, 2013 (in Japanese)

68) Yoshikuni, H., Inoue, T., Sumioka, N. and Hara, H.: On the characteristics of settlement prediction methods by monitoring, Soil mechanics and foundation engineering, Vol.29, No.8, pp.7-13, 1981 (in Japanese)

69) Japanese Geotechnical Society: Design and Execution of Sand Compaction Pile Method using Vibro-driving and Vibro-removal, p.120, 2009 (in Japanese)

70) Japanese Geotechnical Society: Design and Execution of Sand Compaction Pile Method using Vibro-driving and Vibro-removal, pp.30-37, 2009 (in Japanese)

71) Japanese Geotechnical Society: Design and Execution of Sand Compaction Pile Method using Vibro-driving and Vibro-removal, p,57, 2009 (in Japanese)

72) Japan Cement Association, Manual of ground improvement using cement-based stabilizer, pp.4-6, 2012 (in Japanese)

73) Public Works Research Center: Design and construction manual of deep mixing method on land, p.37, 2004 (in Japanese)

74) Terashi, M., Fuseya, H. and Noto, S.: Soil mechanics and foundation engineering, Vol.31, No.6, pp.57-64, 1983 (in Japanese)

75) Japanese Geotechnical Society: Laboratory Testing Standards of Geomaterials, pp.426-434, 2009 (in Japanese)

76) Japanese Geotechnical Society: Laboratory Testing Standards of Geomaterials, pp.541-551, 2009 (in Japanese)

77) Coastal Development Institute of Technology: Technical manual of deep mixing method for ports and airports, p.123, 2014 (in Japanese)

78) Coastal Development Institute of Technology: Technical manual of light weight treated soil method for ports and airports, pp.180-182, 2014 (in Japanese)

79) Coastal Development Institute of Technology: Technical Manual of deep mixing method for ports and airports, pp.147-153, 2014 (in Japanese)

80) Morita, T., Iai, S., H. Liu., Ichii, K. and Sato, Y.: Simplified method to determine parameter of FLIP, Technical Note of the Port and Harbour Research Institute Ministry of Transport, No.869, 36p., 1997 (in Japanese)

81) Japanese Geotechnical Society: Design and Execution of Sand Compaction Pile Method using Vibro-driving and Vibro-removal, p.104, 2009 (in Japanese)

82) Coastal Development Institute of Technology: Technical Manual of light weight treated soil method for ports and airports (Revised Edition), pp.346-361, 2014 (in Japanese)

83) Coastal Development Institute of Technology: Technical Manual for pneumatic flow mixing method (Revised Edition), pp.128-138, 2008 (in Japanese)

84) Coastal Development Institute of Technology: Technical Manual for premixing-type stabilization method, pp.170-171, 2008 (in Japanese)

85) Coastal Development Institute of Technology: Technical Manual of light weight treated soil method for ports and airports (Revised Edition), p.102, 2014 (in Japanese)

86) Coastal Development Institute of Technology: Technical Manual for pneumatic flow mixing method, pp.42, 2008 (in Japanese)

87) Coastal Development Institute of Technology: Technical Manual for premixing-type stabilization method, pp.52-56, 2008 (in Japanese)

88) Coastal Development Institute of Technology: Technical Manual of light weight treated soil method for ports and airports (Revised Edition), pp.151-156, 2014 (in Japanese)

89) East Nippon Expressway, Central Nippon Expressway, West Nippon Expressway: NEXCO test method, PART 3 Test methods for concrete, pp.8-12, 2016 (in Japanese)

90) Coastal Development Institute of Technology: Technical Manual for premixing-type stabilization method (Revised Edition), p.51, 2008 (in Japanese)

91) Coastal Development Institute of Technology: Technical Manual of light weight treated soil method for ports and airports, pp.76-97, 2014 (in Japanese)

92) Coastal Development Institute of Technology: Technical Manual for pneumatic flow mixing method (Revised Edition), pp.57-67, 2008 (in Japanese)

93) Coastal Development Institute of Technology: Technical Manual for premixing-type stabilization method (Revised Edition), pp.67-70, 2008 (in Japanese)

94) Japanese Geotechnical Society: Geotechnical Engineering Handbook, Japanese Geotechnical Society, p.1208, 1999 (in Japanese)

95) Eide, O. and Bjerrum, L.: The slide at Bekkelaget, Geotechnique, Vol.5, No.1, pp.88-100, 1955

96) Akai, K.: Stability analyses of embankment on soft ground by composite slip surface method, Soil mechanics and foundation engineering, Special Issue, No.2, pp.9-13, 1960 (in Japanese)

97) Coastal Development Institute of Technology (CDIT): Handbook for countermeasure works for reclaimed land (Revised Edition), pp.114, 1997 (in Japanese)

98) Katayama, T., Yahiro, A., Kitazume, M. and Nakanodo, H.: Analysis and experimental study on fabric-packed sand-drain's stability in Tokyo International Airport extension protect, Jour. Of JSCE, No.486/IV-22, pp.19-25, 1994 (in Japanese)

99) Terzaghi, K., Peck, R.B. and Mesri, G.: Soil mechanics in engineering practice (Third Edition), John Wiley & Sons, p.81, 1996

100) Aboshi, H and Yoshikuni, H.: A study on the consolidation process affected by well resistance in the vertical drain method, Soils and Foundations, Vol.7, No.4, pp.38-58, 1967

101) Japanese Geotechnical Society: Geotechnical Engineering Handbook, Japanese Geotechnical Society, p.1215, 1999 (in Japanese)

102) Yoshikuni, H.: Design and construction supervision of vertical drain method, Giho-do Publishing, pp.29-76, 1979 (in Japanese)

103) Yoshikuni, H.: Design and construction supervision of vertical drain method, Giho-do Publishing, pp.175-192, 1979 (in Japanese)

104) Maeda, K., Takahashi, H., Ishiguro, T. and Mimura, M.: Development of network drain method, Geosynthetics Engineering Journal, Vol.20, pp.89-96, 2005 (in Japanese)

105) Nakase, A.: Design diagram for sand drain, Soil mechanics and foundation engineering, Vol.12 ,No.6, pp.35-38, 1964 (in Japanese)

106) Amihoshi, H., Koba, Z., Inoue, T., Niki, M. and Murase, H.: Change of consolidation coefficient due to the construction of sand drain, Proceedings of the 19th Japan National Conference on Soil and Mechanics and Foundation Engineering, pp. 1573-1574, 1984 (in Japanese)

107) Tanaka, H., Oota, K. and Maruyama, T.: Performance of vertical drains for soft and ununiform soils, Report of Port and Harbour Research Institute, Vol.30, No.2, pp,211-227, 1999 (in Japanese)

108) Onoue, A.: Consolidation of multi-layered anisotropic soils by vertical drains with well resistance. Soils and Foundations, Vol.28, No.3, pp.75-90, 1988

109) Kobayashi, M., Mizukami, J. and Tsuchida, T.: Determination of the horizontal coefficient of consolidation cohesive soil, Report of Port and Harbour Research Institute, Vol.29, No,2, pp.63-83, 1990 (in Japanese)

110) Hitachi, S., Yamamoto, H., Ikeda, N., Oikawa, K. and Nakanodou, H.: Consolidation of clay layer underlying vertical drain improved ground, Proceedings of the 29th Japan National Conference on Soil and Mechanics and Foundation Engineering, pp.2107-2110, 1994 (in Japanese)

111) Kobayashi, M.: Numerical analysis of one-dimensional consolidation problems, Report of Port and Harbour Research Institute, Vol.21, No.1, pp.57-79, 1982 (in Japanese)

112) Mikasa, M.: Consolidation of soft clay, Kajima Publishing, 160p., 1963 (in Japanese)

113) Terashi, M., Tanaka, H., Mitsumoto, T., Niidome, Y. and Honma, J.: Fundamental Properties of Lime- and Cement-Treated Soils (2nd Report), Report of Port and Harbour Research Institute, Vol.19, No,1, pp.33-62, 1980 (in Japanese)

114) Coastal Development Institute of Technology: Technical Manual of deep mixing method for ports and airports, pp.32-102, 2014 (in Japanese)

115) Coastal Development Institute of Technology: Technical Manual for FGC deep mixing method- soft soil improvement method utilizing fly ash, 2004 (in Japanese)

116) Babasaki, R., Terashi, M., Suzuki, T., Maekawa, J., Kawamura, M. and Fukazawa, E.: Influence factors on the strength of stabilized soil, Japanese Geotechnical Society, Proceedings of Symposium on Cement-Treated Soils, pp.20-41, 1996 (in Japanese)

117) Noto, S., Taguchi, N. and Terashi, M.: Practice of deep ground Improvement and problems 11, Practice and problems of deep mixing method- Examples of deep mixing method-, Soil and Foundation, Vol.31, No.7, pp.73-80, 1983 (in Japanese)

118) Coastal Development Institute of Technology: Technical Manual of deep mixing method for ports and airports, p.42, 2014 (in Japanese)

119) Terashi, M., Kitazume, M. and Nakamura, T.: External Forces Acting on a Stiff Soil Mass Improved by DMM, Report of Port and Harbour Research Institute, Vol. 27, No.2, pp.147-184, 1988 (in Japanse)

120) Kitazume, M.: Model and Analytical Studies on Stability of improved ground by Deep Mixing Method, Technical Note of Port and Harbour Research Institute, No.774, 73p., 1994 (in Japanese)

121) Coastal Development Institute of Technology: Technical Manual of deep mixing method for ports and airports, pp.47-53, 2014 (in Japanese)

122) Terashi, M. and Kitazume, M.: Interference Effect on Bearing Capacity of Foundations on Sand, Report of Port and Harbour Research Institute, Vol.26, No,2, pp. 413-436, 1987 (in Japanese)

123) Terashi, M., Tanaka, H. and Kitazume, M.: Dislodging failure of wall-type improved soil by deep mixing method, Proceedings of the 18th Japan National Conference on Soil and Mechanics and Foundation Engineering, pp.1553-1556, 1983 (in Japanese)

124) Coastal Development Institute of Technology: Technical Manual of deep mixing method for ports and airports, pp.306-307, 2014 (in Japanese)

125) Takahashi, H., Morikawa, Y., Tsukuni, S., Yoshida, M. and Fukada, H.: Approach to reducing improvement depth of lattice-shaped cement treatment method for liquefaction countermeasure, Report of the Port and Airport Research Institute, Vol.51, No.2, pp.3-39, 2012 (in Japanese)

126) Koga, Y., Mastuo, O., Enokida, M., Ito, K. and Suzuki, Y.: Shaking table tests on DMM method as a countermeasure against liquefaction of sandy ground (Part 2) -effects of improved in grid configuration against

liquefaction-, Proceedings of the 23th Japan National Conference on Soil and Mechanics and Foundation Engineering, pp.1019-1020, 1988 (in Japanese)

127) Babasaki, R., Suzuki, Y., Suzuki, Y. and Fujii, N.: Cell type foundation improved by deep cement mixing method against soil liquefaction (Part 2) centrifugal vibration tests on cell type foundation, Proceedings of the 26th Japan National Conference on Soil and Mechanics and Foundation Engineering, pp.1007-1008, 1991 (in Japanese)

128) Namikawa, T., Koseki, J. and Suzuki, Y.: Finite Element Analysis of Lattice-shaped Ground Improvement by Cement-mixing for Liquefaction Mitigation, Soils and Foundations, Vol.47, No.3, pp.559-576, 2007

129) Suzuki, Y., Saito, S., Onimaru, S., Kimura, T., Uchida, A. and Okumura,R.: Grid-shaped stabilized ground improved by deep cement mixing method against liquefaction for a building foundation, Soil and Foundation, Vol.44, No.3, pp.46-48, 1995 (in Japanese)

130) Higashi, S., Harada, K., Nitao, H., Hashimoto, N., Suzuki, A., Hatsuyama, Y., Tateshita, K., Sugano, T. and Nakazawa, H.: Study on liquefaction countermeasure effectiveness of lattice-type deep mixing based on full-scale field test, Journal of the Society of Materials Science, Japan, Vol.59, No.1, pp.14-19, 2010 (in Japanese)

131) Coastal Development Institute of Technology: Technical Manual of deep mixing method for ports and airports, pp.86-93, 2014 (in Japanese)

132) Cement Deep Mixing Method Association: Technical Manual of Floating-type Lattice-shaped Liquefaction Countermeasure Method (FULAT method) (Draft), 147p., 2014 (in Japanese)

133) Japan Cement Association: Ground improvement manual using cement-based solidified material (Vol.3), pp.281-286, 2003 (in Japanese)

134) Coastal Development Institute of Technology: Technical Manual of deep mixing method for ports and airports, pp.99-100, 2014 (in Japanse)

135) Coastal Development Institute of Technology: Technical Manual for FGC deep mixing method- soft soil improvement method utilizing fly ash 1, p.3-14, 2002 (in Japanese)

136) Sunami, S., Ohishi, K., Ohono, M., Katagiri, M., Katoh, T. and Sonoi, K.: Centrifuge model tests for steel pipe sheet-pile revetment with soil improvement, Proceedings of the 41th Japan National Conference on Geotechnical Engineering, pp.1683-1684, 2006 (in Japanese)

137) Katoh, T., Sonoi, K., Sunami, S., Ohishi, K., Ohono, M. and Katagiri, M.: Design method of steel pipe sheet-pile revetment with soil improvement, Proceedings of the 41th Japan National Conference on Geotechnical Engineering, pp.1685-1686, 2006 (in Japanese)

138) Khan, M.R.A., Hayano, K. and Kitazume, M.: Behavior of sheet pile quay wall stabilized with sea-side ground improvement, Report of the Port and Horbour Research Institute, Vol.46, No.4, pp.3-39, 2007

139) Khan, M.R.A., Hayano, K. and Kitazume, M.: Investigation on static stability of sheet pile quay wall improved by cement treated sea-side ground from centrifuge model test, Soils and Foundations, Vol.48, No.4, pp.563-575, 2008

140) Khan, M.R.A., Hayano, K. and Kitazume, M.: Behavior of sheet pile quay wall stabilized with sea-side ground improvement in dynamic centrifuge test, Soils and Foundations, Vol.49, No.2, pp.193-206, 2009

141) Coastal Development Institute of Technology (CDIT): Technical Manual of light weight treated soil method for ports and airports, 371p., 2014 (in Japanese)

142) Tsuchida, T., Yokoyama, Y., Mizukami, J., Shimizu, K. and Kasai, J.: Field test of light-weight geomaterials for harbor structures, Technical Note of Port and Harbour Research Institute, No.833, 30p., 1996 (in Japanese)

143) Tsuchida, T.: Research and development of light weight treated soil method in coastal areas and examples of practice, Proceedings of Annual Meeting of Port and Harbour Research Institute, 1998 (in Japanese)

144) Tsuchida, T., Kasai, J., Mizukami, J., Yokoyama, Y. and Tsuchida, K.: Effect of curing condition on mechanical properties of light-weight soils, Technical Note of Port and Harbour Research Institute, No.834, 24p., 1996 (in Japanese)

145) Tsuchida, T., Nagai, K., Yukawa, M., Kishida, T. and Yamamoto, M.: Properties of light-weight soil used for backfill of pier, Technical Note of Port and Harbour Research Institute, No.835, 15p., 1996 (in Japanese)

146) Tang, Y., Tsuchida, T., Takeuchi, D., Kagamida, M. and Nishida, N.: Mechanical properties of light weight cement treated soil using triaxial Apparatus, Technical Note of Port and Harbour Research Institute, No.845, 29p., 1996 (in Japanese)

147) Tsuchida, T., Fuzisaki, H., Nakamura, H., Makibuchi, M., Shinsha, H., Nagasaka, Y. and Hikosaka, Y.: Use of light-weight treated soils made of waste soil in airport extension project, Technical Note of Port and Harbour Research Institute, No.923, 24p., 1999 (in Japanese)

148) Tsuchida, T., Kikuchi, Y., Fukuhara, T., Wako, T. and Yamamoto, K.: Slice method for earth pressure analysis and its application to light-weight fill, Technical Note of Port and Harbour Research Institute, No,924, 28p., 1999 (in Japanese)

149) Tsuchida, T., Wako, T., Kikuchi, Y., Azuma, T. and Sinsya, H.: Fluidity and material properties of light-weight treated soil casted underwater, Technical Note of Port and Harbour Research Institute, No.865, 25p., 1997 (in Japanese)

150) Tsuchida, T., Wako, T., Matsushita, H. and Yoshiwara, M.: Evaluation of washout resistance of light-weight treated soil casted underwater, Technical Note of Port and Harbour Research Institute, No.884, 24p., 1997 (in Japanese)

151) Wako, T., Tsuchida, T., Matsunaga, Y., Hamamoto, K., Kishida, T and Fukasawa, T.: Use of artificial light weight materials (Treated soil with air form)for port facilities, Jour. JSCE, No,602/VI-40, pp.35-52, 1998 (in Japanese)

152) Kikuchi, Y., Ikegami, M. and Yamazaki, H.: Field investigation on the solidification of granulated blast furnace slag used for backfill of quay wall, Jour. Of JSCE, No.799/III-72, pp.171-182, 2005 (in Japanese)

153) Ministry of Land, Infrastructure, Transport and Tourism: Recycling Guidelines for Port and Airport Development (revised version), pp.3-7-1 - 3-7-38, 2015 (in Japanese)

154) Coastal Development Institute of Technology: Handbook of utilization of granulated blast furnace slag for port construction work, 120p., 2007 (in Japanese)

155) Japanese Geotechnical Society: Countermeasure works for liquefaction, Geotechnical Engineering, Practical Business Series, p.317, 2004 (in Japanese)

156) Coastal Development Institute of Technology (CDIT): Handbook for countermeasure works for reclaimed land (Revised Edition), p.195,1997 (in Japanese)

157) Coastal Development Institute of Technology: Technical Manual for premixing-type stabilization method (Revised Edition), 215p., 2008 (in Japanese)

158) Yamazaki, H., Morikawa, Y. and Koike, F.: Study on effect on fines content and drainage characteristics of sandy deposit on sand compaction pile method, Jour. Of JSCE, No.722/III-61, pp.303-314, 2002 (in Japanese)

159) Yamazaki, H., Morikawa, Y. and Koike, F.: Study on effect of K0-value prediction after densification by sand compaction pile method, Jour of JSCE No.750/III-65, pp.231-236, 2003 (in Japanese)

160) Japanese Geotechnical Society: Countermeasure works for liquefaction, Geotechnical Engineering, Practical Business Series, ,pp.233-242, 2004 (in Japanese)

161) Mizuno, T., Suematsu, N. and Okuyama, K.: Design method of sand compaction pile in sandy ground containing fine fraction and evaluation of improvement effect, Soil and Foundation Vol.35, No5, pp.21-26, 1987 (in Japanese)

162) Oe, M., Suzuki, Y. and Ohwaki,T.: Coefficient of lateral subgrade reaction of soft clayey seabed foundation improved by SCP in the amagasaki lock improvement work, Soil mechanics and foundation engineering, Vol.42, No.6, pp.47-50, 1994 (in Japanese)

163) Kitazume, M. and Murakami, K.: Behaviour of sheet pile walls in the improved ground by sand compaction piles of low replacement area ratio, Report of Port and Harbour Research Institute, Vol.32, No.2, pp.183-211, 1993 (in Japanese)

164) Kitazume, M., Takahashi, H. and Takemura, S.: Experimental and analytical studies on horizontal resistance of sand compaction pile improved ground, Report of Port and Harbour Research Institute, Vol.42, No.2, pp.47-71, 2003 (in Japanese)

165) Japanese Geotechnical Society: Estimation of effectiveness of soil improvement and practice, pp.163-165, 2000 (in Japanese)

166) Sugiyama, H., Iai, S., Kotsutsumi, O. and Mori, H.: Analysis of effective stress of gravity-type wharf on a clayey ground improved by SCP during an earthquake- (First Rept; Modeling of high replacement rate SCP improved soil.), Proceedings of 35th Conference on Geotechnical Engineering, ,pp.2055-2056, 2000 (in Japanese)

167) Sato, N., Yoshida, A., Iida, N., Tange, H., Iai, S. and Mori, H.: Analysis of effective stress of gravity-type wharf on a clayey ground improved by SCP during an earthquake- (Second Rept; Case Study), Proceedings of 35th Conference on Geotechnical Engineering, „pp.2057-2058, 2000 (in Japanese)

168) Kitazume, M., Sugano, T., Kawamata, Y., Nishida, N., Ishimaru, K. and Nakayama, Y.: Centrifuge model tests on dynamic properties of sand compaction pile improved ground, Technical Note of Port and Airport Research Institute, No.1029, p.17, 2003 (in Japanese)

169) Sugano, T., Kitazume, M., Nakayama, Y., Kawamata, Y., Obayashi, J., Nishida, N. and Ishimaru, K.: A Study on dynamic properties of sand compaction pile improved ground, Technical Note of Port and Airport Research Institute, No.1047, p.32, 2003 (in Japanese)

170) Nozu, A. and Kohama, E.: Damage to port structures during the 2016 kumamoto earthquake sequence, Geotechnical Engineering magazine, Vol.65, No.4, pp.36-39, 2017 (in Japanese)

171) Kitazume, M.: the Sand Compaction Pile Method, A. A. Balkema Publishers, pp.156-163, 2005

172) Takahashi, H.: Fundamental study on the failure process of ground composed of sand piles and cohesive soil, Technical Note of the Port and Airport Research Institute, No.1181, 155p., 2008 (in Japanese)

173) Ministry of Land, Infrastructure, Transport and Tourism: Recycling Guidelines for Port and Airport Development (revised version), pp.3-8-1 - 3-8-28, 2015 (in Japanese)

174) Coastal Development Institute of Technology: Technical Manual for Using Steelmaking Slag in Ports, Airports and Seashores, pp.21-24, 2015 (in Japanese)

175) Ministry of Land, Infrastructure, Transport and Tourism: Recycling Guidelines for Port and Airport Development (revised version), pp.3-14-1 - 3-14-16, 2015 (in Japanese)

176) Coastal Development Institute of Technology: Technical Manual for Using Non-ferrous Slag in Port and Airport Constructions, pp.88-91, 2015 (in Japanese)

177) Ministry of Land, Infrastructure, Transport and Tourism: Recycling Guidelines for Port and Airport Development (revised version), pp.3-15-1 - 3-15-30, 2015 (in Japanese)

178) Coastal Development Institute of Technology: Technical Manual for Using Non-ferrous Slag in Port and Airport Constructions, pp.94-97, 2015 (in Japanese)

179) Hashidate, Y., Fukuda, S., Okumura, T. and Kobayashi, M.: Engineering characteristics of sand containing oyster shells and utilization for sand compaction piles, Proceedings of the 29th Japan National Conference on Soil and Mechanics and Foundation Engineering, pp.717-720, 1994 (in Japanese)

180) Coastal Development Institute of Technology: Technical Manual for Using Steelmaking Slag in Ports, Airports and Seashores, pp.14-18, 2015 (in Japanese)

181) Coastal Development Institute of Technology: Technical Manual for Using Non-ferrous Slag in Port and Airport Constructions, pp.22-23, 2015 (in Japanese)

182) Coastal Development Institute of Technology: Technical Manual for Using Steelmaking Slag in Ports, Airports and Seashores, p.10, 2015 (in Japanese)

183) Ministry of Land, Infrastructure, Transport and Tourism: Recycling Guidelines for Port and Airport Development (revised version), p.3-17-12, 2015 (in Japanese)

184) Shiomi, M. and Kawamoto, K.: Estimation of rise of ground due to SPC driving, Proceedings of the 21st Conference of Soil Mechanics, Proceedings of the 29th Japan National Conference on Soil and Mechanics and Foundation Engineering, pp. 1861-1862, 1986 (in Japanese)

185) Hirao, H, Tsuboi, H., Matsuo, M. and Taga, H.: Profile forecast of emergence of sea bed ground due to compaction of sand piles, Proceedings of the 8th Symposium on Geotechnical Engineering, pp.55-60, 1996 (in Japanese)

186) Fukute, T., Higuchi, Y., Furuichi, M. and Tsuboi, H.: Profile forecast of emergence of sea bed due to large scale sand compaction piles, Proceedings of the 33rd Symposium on Soil Mechanics, pp,23-28, 1988 (in Japanese)

187) Ogata, F., Chinen, M., Konishi, A. and Kuno, A.: Efficient implementation procedure of off-shore SCP method on shallow sea, Proceedings of the 50th Japan National Conference on Geotechnical Engineering, pp.853-854, 2015 (in Japanese)

188) Aboshi, H., Nakamura, R., Okumura, T., Sogabe, T. and Ichimoto, E.: Problems of sand compaction method and deep mixing method for soft ground, JSCE Magazine Civil Engineering, Vol.67, No.4, pp.22-31, 1982 (in Japanese)

189) Ichimoto, E. and Suematsu, N.: Practice of deep ground improvement and problems, 9, Practice and problems of deep mixing method- Outline of deep mixing (3) -Summary-, Soil and Foundation, Vol.31, No.5, pp.83-90, 1983 (in Japanese)

190) Matsuo, M., Kimura, M., Nishio, R. and Andou, H.: Study on development of soil improvement method using construction waste soil, Jour. JSCE, No. 547/III-36, pp.199-210, 1996 (in Japanese)

191) Nozu, M. and Suzuki, A.: Effect of sand compaction piles on the consolidation of surrounding clayey ground and its utilization, Symposium on Recent Research and Practice on Clayey Ground- from observation of microscopic structure to countermeasure technology for extremely soft reclaimed land-, pp. 327-332, 2002 (in Japanese)

192) Hirao, H. and Matsuo, M.: Study on characteristics of upheaval part of cohesive ground caused by soil improvement, Jour. JSCE, Vol.376/III-6, pp.277-285, 1986 (in Japanese)

193) Ichimoto, E.: Practical design of sand compaction pile method and examples of construction, Proceedings of Annual Technical Conference, pp.51-55, 1981 (in Japanese)

194) Kanda, K. and Terashi, M.: Practical Formula for the Composite Ground Improved by Sand Compaction Pile Method, Technical Note of Port and Harbour Research Institute No. 669, pp.1-52, 1990 (in Japanese)

195) Japanese Geotechnical Society: Design and Execution of Sand Compaction Pile Method using Vibro-driving and Vibro-removal, pp.42-43, 2009 (in Japanese)

196) Tanaka, Y., Nakamichi, M., Nakai, A., Fujii, Y., Shiraishi, S. and Umeki, Y.: Design and construction of breakwater with economical soil improvement (T-shape type sand compaction pili method), Proceedings of the 39th Japan National Conference on Geotechnical Engineering pp.989-990, 2004 (in Japanese)

197) Kudou, K. and Fujii, Y.: Verification of economical ground improvement method (T-shaped SCP), Foundation Engineering & Equipment, Vol.34, No,7, pp.42-44, 2006 (in Japanese)

198) Coastal Development Institute of Technology (CDIT): Handbook of Countermeasure against Liquefaction of Reclaimed Land (Revised Edition), CDIT, pp.158-161, 1997 (in Japanese)

199) Ishiguro, T., Iijima, T., Shimizu, H. and Shimada, S.: Investigation about the vibration compaction work of saturated sand layers with elimination of excess pore-water pressure, Jour. JSCE, No.505/III-29, pp.105-114, 1994 (in Japanese)

200) Japanese Geotechnical Society: Countermeasure works for liquefaction, Geotechnical Engineering, pp.398-401, 2004 (in Japanese)

201) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, p.112, 2013 (in Japanese)

202) Nakada, K. and Terauchi, K.: Quaywall's destruction and repair works of the Akita port caused by the Nihonkai-Chubu Earthquake, Soil mechanics and foundation engineering, Vol.32, No.9, pp.27-37, 1984 (in Japanese)

203) Nakada, K., Terauchi, K. and Sendai, Y.: An experimental sand liquefaction prevention method, Soil mechanics and foundation engineering, Vol.32, No.12, pp.29-37, 1984 (in Japanese)

204) Coastal Development Institute of Technology (CDIT): Handbook of Countermeasure against Liquefaction of Reclaimed Land (Revised Edition), CDIT, pp.161-167, 1997 (in Japanese)

205) Industrial Technology Service Center: Compendium of practical measures for soft ground, Part 2, Chapter 1 and 2, pp.726-732, 1993 (in Japanese)

206) Coastal Development Institute of Technology (CDIT): Handbook of Countermeasure against Liquefaction of Reclaimed Land (Revised Edition), CDIT, pp.152-157, 1997 (in Japanese)

207) Kohama, E., Sugano, T. and Ohya, Y.: Effect of tsunami in steel sheet pile type quaywall damaged by 2011 Great East Japan Earthquake, Japanese Geotechnical Society Special Symposium -After Overcoming Great East Japan Earthquake-, pp.312-316, 2014 (in Japanese)

208) Nakagaki, N., Kohama, E., Kusunoki, K. and Murakami, K.: Numerical simulations on cause of damage to a sheet pile type quay wall in soma port during the 2011 off the Pacific Coast of Tohoku Earthquake, Jour of JSCE, Vol.72, No.4, pp.I_856-I_870, 2016 (in Japanese)

209) Hayashi, K., Kohama, E., Yamazaki, H. and Sassa, S.: Design of outlet for countermeasure for liquefaction by drain method, Proceedings of the 52th Japan National Conference on Geotechnical Engineering, pp.1365-1366, 2017 (in Japanese)

210) Coastal Development Institute of Technology (CDIT): Handbook of Countermeasure against Liquefaction of Reclaimed Land (Revised Edition), CDIT, pp.170-188, 1997 (in Japanese)

211) Industrial Technology Service Center: Handbook of practical Technology for countermeasure works of soft ground for construction engineers, Part 3, Chapter 6, pp.676-689,1993 (in Japanese)

212) Industrial Technology Service Center: Handbook of practical Technology for countermeasure works of soft ground for construction engineers, Part 3, Chapter 1, pp.619-631, 1993 (in Japanese)

213) Japanese Geotechnical Society: Countermeasure works for liquefaction, Geotechnical Engineering, Practical Business Series, pp.326-335, 2004 (in Japanese)

214) Coastal Development Institute of Technology: Technical Manual for osmotic solidification method, 172p., 2010 (in Japanese)

215) Coastal Development Institute of Technology: Coastal Development Technology Library No. 11, Technical Manual for pneumatic flow mixing method, pp.188, 2008 (in Japanese)

216) Taguchi, H., Yamane, N., Hashimoto, F. and Sakamoto, A.: Strength characteristics of stabilized ground by plug-flow mixing method, Proceedings of IS-YOKOHAMA, pp. 719-724, 2000

217) Coastal Development Institute of Technology: Technical Manual for pneumatic flow mixing method (Revised Edition), pp.116-117, 2008 (in Japanese)

218) Okabe, S.: General Theory on Earth Pressure and Seismic Stability of Retaining Wall and Dam, JSCE Magazine Civil Engineering, Vol.10, No.6, pp.1277-1323, 1924

219) Matsunami, H.: Studies on the design method of flexible anchorages of the quaywall, Report of the Port and Harbour Research Institute Ministry of Transport, Vol.19, No.3, pp.191-273, 1980 (in Japanese)

220) Coastal Development Institute of Technology: Technical Manual of light weight treated soil method for ports and airports (Revised Edition), pp.30-34, 2014 (in Japanese)

221) Yoshida, H.: Technical issues and prospects of Jet Grouting Method, Foundation Engineering & Equipment, Vol.37, No.5, pp.8-13, 2009 (in Japanese)

222) Ikeda, A. and Hatsuyama, Y.: Recent case examples of quality control in Jet Grouting Method, Foundation Engineering & Equipment, Vol.27, No.7, pp.84-87, 1999 (in Japanese)

223) Industrial Technology Service Center: Handbook of practical Technology for countermeasure works of soft ground for construction engineers, Industrial Technology Service Center, pp.803-804, 1993 (in Japanese)

224) Japanese Geotechnical Society: Investigation, design, and execution of ground improvement - from detached houses to artificial island -, Geotechnical Engineering - Series of practice No.31, pp.145-149, 2013 (in Japanese)

225) Yamazaki, J.: Overview and application example of Jet Grouting Method V-JET Method enabling large diameter and high ‐ speed construction, Journal of civil engineering, Vol.53, No.5, pp.26-29, 2012 (in Japanese)

226) Yamazaki, J.: V-JET Method enabling large diameter and high-speed construction, Issues and countermeasures for ground disasters caused by earthquake, Lessons of 2011 Great East Japan Earthquake and proposals (Second), pp.304-305, 2012 (in Japanese)

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.