[Technical Note]

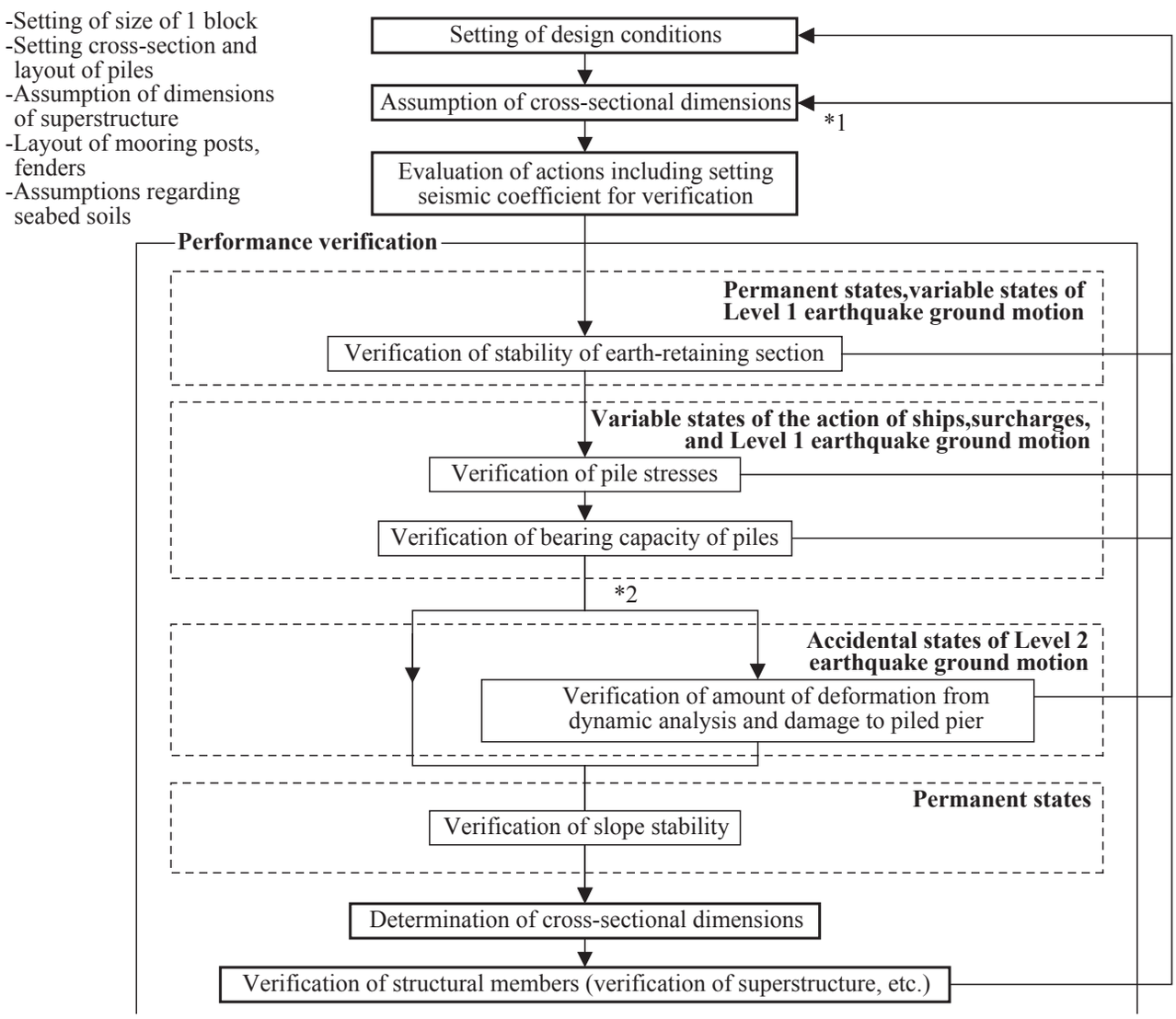
5.1 Common Items for Piled Piers

(1) The performance verification of piled piers in common may be in accordance with **2.1 Common Items for Quaywalls**.

(2) The structural types of piled piers include open-type wharves on vertical piles, open-type wharves on coupled raking piles, jacket type piers and strutted frame type pier.

(3) An example of the procedure of the performance verification of piled piers is shown in **Fig. 5.1.1**.

(4) Access Bridges
In setting the structure and cross-sectional dimensions of access bridges in the performance verification of piled piers, it is necessary to appropriately consider the conditions of use of the concerned piers, in order that the piled pier can be safely and efficiently used.
Also, in setting the structure and cross-sectional dimensions of access bridges in the performance verification of piled piers, it is necessary to appropriately consider the amount of relative deformation between the main structure of the piled pier and the earth-retaining section, and also the allowable horizontal displacement of the access bridge.



*1: Evaluation of the effect of liquefaction and settlement is not shown on the diagram, so it is necessary to separately into consider.
*2: Verification shall be carried out for high earthquake-resistance facilities against the Level 2 earthquake ground motion.

Fig. 5.1.1 Example of the Sequence of Performance Verification of a Piled Pier

5.2 Open-type Wharves on Vertical Piles

5.2.1 Fundamentals of Performance Verification

(1) The following refers to open-type wharves on vertical piles using steel pipe piles or steel sections, but it may also be applied to similar facilities provided that their dynamic characteristics are taken into account.

(2) For the procedure of performance verification of open-type wharves on vertical piles, it is possible to refer to **Fig. 5.1.1** of **5.1 Common Items for Piled Piers**. However, evaluation of the effect of liquefaction is not shown in **Fig. 5.1.1**, so it is necessary to appropriately investigate the potential for liquefaction and measures against it, (refer to **Part II, Chapter 6 Ground Liquefaction**).

(3) In the performance verification of open-type wharves on vertical piles, normally the cross-section is set with respect to actions other than that of Level 2 earthquake ground motion, while the seismic performance is verified with respect to Level 2 earthquake ground motion. This is because for verification of variable situation in respect of the action of ships and Level 1 earthquake ground motion, the performance verification is carried out based on the yield stress for the steel pipe piles, but for seismic performance verification of seismic-resistant with respect to Level 2 earthquake ground motion, a verification method that takes the extent of damage to the piled pier into account is used.

(4) For the variable situation in respect of Level 1 earthquake ground motion, it is possible to carry out verification by obtaining the natural periods of the piled pier based on a frame analysis, and then calculating the seismic coefficient for verification using the obtained natural periods and the acceleration response spectrum. However, for high earthquake-resistance facilities, verification may be carried out using an appropriate dynamic analysis method, such as nonlinear seismic response analysis taking into account the 3-dimensional dynamic interaction effect between piles and the ground. For open-type wharves on vertical piles other than high earthquake-resistance facilities, it is possible to omit the verification of the accidental situation for Level 2 earthquake ground motion.

(5) An example of cross-section of an open type piled pier on vertical piles is shown in **Fig. 5.2.1**.

(6) When cargo handling equipment, such as container cranes, is to be installed on an open-type wharf on vertical piles, it is preferable to install it in such a way that all of its feet are positioned on either the pile-supported section or earth-retaining section. If, for example, one foot of a cargo handling equipment is positioned on the pile-supported section and another on the earth-retaining section, the equipment becomes susceptible to adverse effects by uneven settlement and ground motions, due to the difference in the response characteristics of the two sections. When it is unavoidable to position one foot on the pile-supported section and another on the earth-retaining section, sufficient foundation work such as foundation piles should be provided to prevent uneven settlement due to the settlement on the earth-retaining section. In this case, in general, the fixed foot of cargo handling equipment such as portal crane should not be installed. When installing cargo handling equipment, such as container cranes, seismic response analysis should be performed, taking into consideration the coupled oscillation of the cargo handling equipment and the open-type wharf.

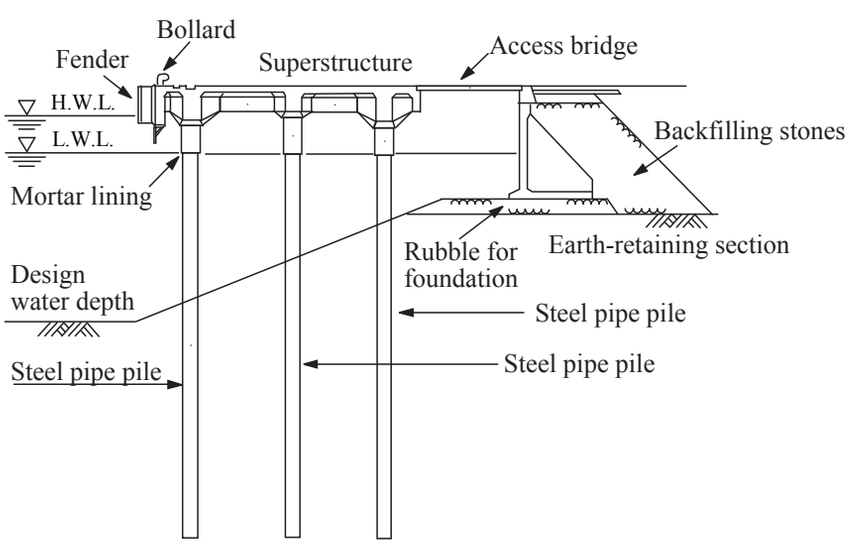


Fig. 5.2.1 Example of Cross-section of an Open Type wharf on Vertical Piles

5.2.2 Setting of Basic Cross-section

(1) The size of a deck block, the distances between piles, and the number of pile rows shall be determined appropriately in consideration of the following:
① apron width
② location of sheds
③ seabed, especially slope stability
④ existing revetments
⑤ matters related to construction work such as the concrete casting capacity
⑥ surcharges, especially crane specifications

(2) In such a case that large quay cranes for ships of 10,000 ton class are to be installed, piles are usually designed to be placed by 5m with 3-4 pile rows in the cross-section.

(3) The dimensions of the superstructure of open-type wharf shall be determined appropriately considering the following
① distances between piles, number of pile rows, and the shape and dimensions of piles
② construction problem of shattering forms and scaffold
③ ground conditions
④ arrangement of mooring posts
⑤ arrangement, shape and dimensions of fenders

(4) Assumptions regarding the Seabed Condition

① Determination of gradient of slope

(a) When an earth-retaining structure is provided behind the slope, the position of the earth-retaining structure should be appropriately determined considering the stability of the slope.

(b) It is necessary to examine the stability of slope with respect to circular slip failure. When an earth-retaining structure is installed behind the slope, it is preferable that the structure is not constructed in front of the slope surface from the toe of the slope at the slant angle indicated by equation **(5.2.1)** (see **Fig. 5.2.2**).

$$\alpha = \phi - \varepsilon \tag{5.2.1}$$

where,
α : angle between the slope and the horizontal surface (°)
ϕ : angle of shear resistance of the main material forming the slope (°)
$\varepsilon = \tan^{-1} k_h'$
k_h' : apparent horizontal seismic coefficient

For the seismic coefficient for verification for calculating the apparent horizontal seismic coefficient, the value calculated in the analysis of the earth-retaining section may be used. Refer to **(10)**⑥ below for calculation of the seismic coefficient for verification for the earth-retaining section. In addition, when the slope is composed of a hard mudstone or rock, equation **(5.2.1)** may not be applied.

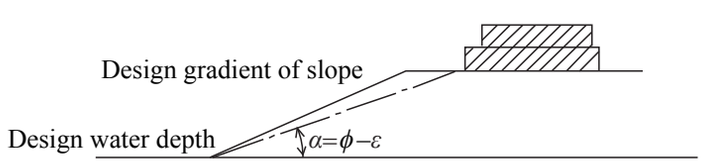


Fig. 5.2.2 Position of Earth Retaining Structure on the Slope

② Virtual Ground Surface

(a) In calculation of lateral resistance and bearing capacity of piles, a virtual ground surface shall be assumed at an appropriate elevation for each pile.

(b) When the inclination of the slope is considerably steep, the virtual ground surface for each pile to be used in the calculation of lateral resistance or bearing capacity may be set at an elevation that corresponds to 1/2 of the vertical distance between the surface of the slope at the pile axis and the seabed as shown in (**Fig. 5.2.3**).

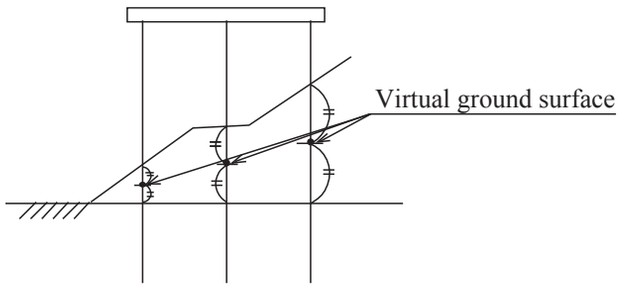


Fig. 5.2.3 Virtual Ground Surface

(5) Coefficient of Lateral Subgrade Reaction

① In the calculation of the lateral resistance of piles, it is preferable to obtain the coefficient of lateral subgrade reaction of the subsoil through lateral loading tests of piles in-situ. In case that no tests are conducted, it may be estimated by means of appropriate analytical methods derived from lateral resistance tests.

② There are some measured data available on the coefficient of lateral subgrade reaction obtained by the tests in which the lateral loads were applied to piles up to the yield points as observed in the case of piles of open-type wharves. Although some of these data have been related to the *N*-value, the coefficient of lateral subgrade reaction cannot be estimated accurately from the *N*-value. Thus, it is preferable to estimate the coefficient by lateral loading tests in-situ.

③ When lateral loading tests of piles are not carried out due to small scale construction works or time constraints, the coefficient of lateral subgrade reaction of the subsoil may unwillingly use the mean value of the minimum value and central value obtained from lateral resistance tests. When using Chang's method, equation (**5.2.2**) may be utilized and **Chapter 2, 2.4.5 [4] Estimation of Pile Behavior using Analytical Methods** can be referenced. However, some in-situ measurement data indicate that the coefficient value of lateral subgrade reaction of rubble stones is smaller than the estimate by equation (**5.2.2**) with Chang's method. In this case it is recommended to set the coefficient of lateral subgrade reaction equal to 3.0-4.0 N/cm^2 in Chang's method.

$$k_{CH} = 1.5N \tag{5.5.2}$$

where

k_{CH} : coefficient of horizontal subgrade reaction (N/cm^3)
N : average *N*-value of the ground down to a depth of about $1/\beta$
β : refer to **(6) Virtual Fixed Point**

The coefficient of lateral subgrade reaction shown in equation **(5.2.2)** is a static coefficient of subgrade reaction, and may be used when calculating the natural periods of piled piers by frame analysis. There is not much knowledge regarding the coefficient of subgrade reaction to be considered when carrying out the verification of seismic response analysis, hence there is a problem in applying equation **(5.2.2)** to dynamic analysis. Therefore it is preferable to set the coefficient equal to about double the value obtained from equation **(5.2.2)**.

(6) Virtual Fixed Point

With respect to an open-type wharf on vertical piles, the virtual fixed points of the piles may be considered to be located at a depth of $1/\beta$ below the virtual ground surface. The value of β is calculated by equation (**5.2.3**).

$$\beta = \sqrt[4]{\frac{k_{CH} D}{4EI}} \quad (\mathrm{cm}^{-1}) \tag{5.2.3}$$

where

k_{CH} : lateral subgrade reaction coefficient (N/cm^3) calculated by equation **(5.2.2)**
D : diameter or width of the pile (cm)
EI : flexural rigidity of the pile (N·cm^2)

5.2.3 Actions

(1) For the calculation of the self weight of reinforced concrete superstructures, each part of the dimensions is assumed based on the dimensions of the superstructure, and the volume is calculated on them. The self weight can be obtained by multiplying unit weight obtained from **Part II, Chapter 10, 2 Self weight** by the volume. In addition, for the calculation of the self weight of reinforced concrete superstructures, 21kN per 1.0m^2 of deck area of the superstructure of the piled pier may be assumed.

(2) At the site expected to be subject to waves, the following items should be examined regarding wave uplift on the super structure of piled pier and the access bridge.

① Stability of the access bridges and pulling resistance of piles against uplift.

② Member strength of the superstructures and access bridges against uplift.
For uplift, refer to **Part II, Chapter 2, 4.7.4(1) Uplift Acting on Horizontal Plates near the Water Surface**.

(3) The static loads may be determined in accordance with **Part II, Chapter 10, 3.1 Static Load**. The earthquake inertia forces due to static loads may normally be considered to act on the upper surface of the deck slab. However, when the center of gravity of the static loads is located at an especially high elevation, it is important to take the height of the center of gravity as the point of application of the horizontal force.

(4) Live loads should be determined in accordance with **Part II, Chapter 10, 3.2 Live Load**. The seismic force due to a rail mounted crane should be calculated by multiplying its self weight by the seismic coefficient for verification, and the force can be considered to be transmitted from the wheels of the crane to the pile-supported section. It is also necessary to carry out seismic response analysis considering the coupled oscillations of the cargo handling equipment and the open-type wharf (refer to **Part III, Chapter 7 Cargo Handling Facilities, 2.2 Fundamentals of Performance Verification)**. In this case, ground motion shall be applied in the form of a time-series seismic wave profile. The wind load acting on crane may be determined in accordance with **Part II, Chapter 2, 2.3 Wind Pressure**.

(5) The fender reaction force can be calculated in accordance with **Part II, Chapter 8, 2.2 Actions Caused by Ship Berthing** and **Part II, Chapter 8, 2.3 Actions Caused by Ship Motions** and **9.2 Fender Equipment**.

(6) The tractive force of vessels can be determined in accordance with **Part II, Chapter 8, 2.4 Actions due to Traction by Ships**. In many cases one bollard is installed to one deck block.

(7) When rubber fenders are installed as a damper on an ordinary large wharf with a unit deck block of 20 to 30m in length, a common practice is to provide two rubber fenders on one block. In many cases, fender intervals of 8 to 13m are used. The berthing behavior of various sizes of ships has been examined by installing 1.5-meter-long rubber fenders on an ordinary large wharf. The results of examination has revealed that it is appropriate to calculate the berthing force on the assumption that the ship's berthing energy is absorbed by one fender. Therefore, the reaction force may basically be calculated on the assumption that the berthing energy is absorbed by one fender when using rubber fenders as a damper. However, this does not apply when fenders are installed continuously along the face line of a wharf.

(8) The berthing energy is also absorbed by the displacement of the main structure of the pier. However, it is a common practice not to take this into consideration because in many cases the energy absorbed by the main structure of the pier accounts for less than 10% of the total berthing energy.

(9) **Fig. 5.2.4** shows an example of the displacement-energy curve and the displacement-reaction force curve of a rubber fender. If a single fender absorbs a berthing energy of $E1$, the corresponding fender deformation $\delta 1$ is obtained. Then, using the other curve, the corresponding reaction force acting on the pier is obtained as $H1$($\delta 1 \rightarrow C \rightarrow H1$). However, if fenders are installed too close to each other and the berthing energy is absorbed by two fenders, the berthing energy acting on one fender becomes $E2 = E1/2$ and the corresponding fender deformation becomes $\delta 2$. As can be obtained from the figure ($\delta 2 \rightarrow D \rightarrow H2$), the reaction force acting on the pier in the two fender case is almost the same as that generated in the single fender case because of the characteristics of rubber fender. Thus the horizontal reaction force acting on the pier becomes $2H2 \fallingdotseq 2H1$, which means that the horizontal force to be used in the performance verification becomes twofold. When using fenders that have such characteristics, therefore, it is preferable to give careful consideration to this behavior of reaction force in the performance verification and the determination of the locating of fenders.

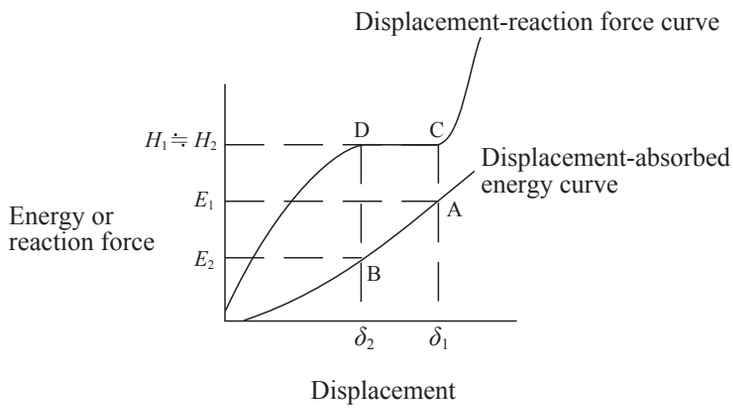


Fig. 5.2.4 Rubber Fender Characteristics Curve

(10) Ground Motion used in Performance Verification of Seismic-resistant

① Ground motion used in performance verification of seismic-resistant is set considering the effect of the surface strata using a ground seismic response analysis. It is necessary to use a seismic response analysis code capable of appropriately evaluating the amplification of ground motions in soft ground (refer to **ANNEX 4, 1 Seismic Response Analysis of Local Soil Deposit**).

② Using a one-dimensional seismic response analysis as described in **ANNEX 4, 1 Seismic Response Analysis of Local Soil Deposit**, the acceleration time history at a position $1/\beta$ below the virtual ground surface is calculated with the acceleration time history of the ground motion set at the seismic bedrock as the input ground motion. When calculating the acceleration time history, the average depth of the $1/\beta$ ground point for each pile may be taken, as shown in **Fig. 5.2.5**. From the acceleration response spectrum obtained in this way, the response accelerations corresponding to the natural periods of the piled pier are calculated, and the value obtained by dividing this by the gravitational acceleration can be regarded as the characteristic value of the seismic coefficient for verification. A damping factor of 0.2 may be used when calculating the acceleration response spectrum. An example of a typical procedure for setting the seismic coefficient for verification is shown in **Fig. 5.2.6**. When verifying the seismic performance of earth-retaining parts using the seismic coefficient method, the structural characteristics are different from those of the piled pier, so the seismic coefficient indicated here may not be used. For the calculation of the seismic coefficient for verification for earth-retaining parts, refer to ⑥ below.

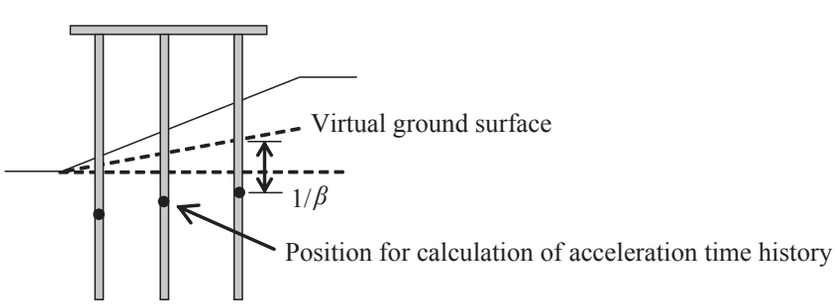


Fig. 5.2.5 Positions for Calculation of Earthquake Ground Motions

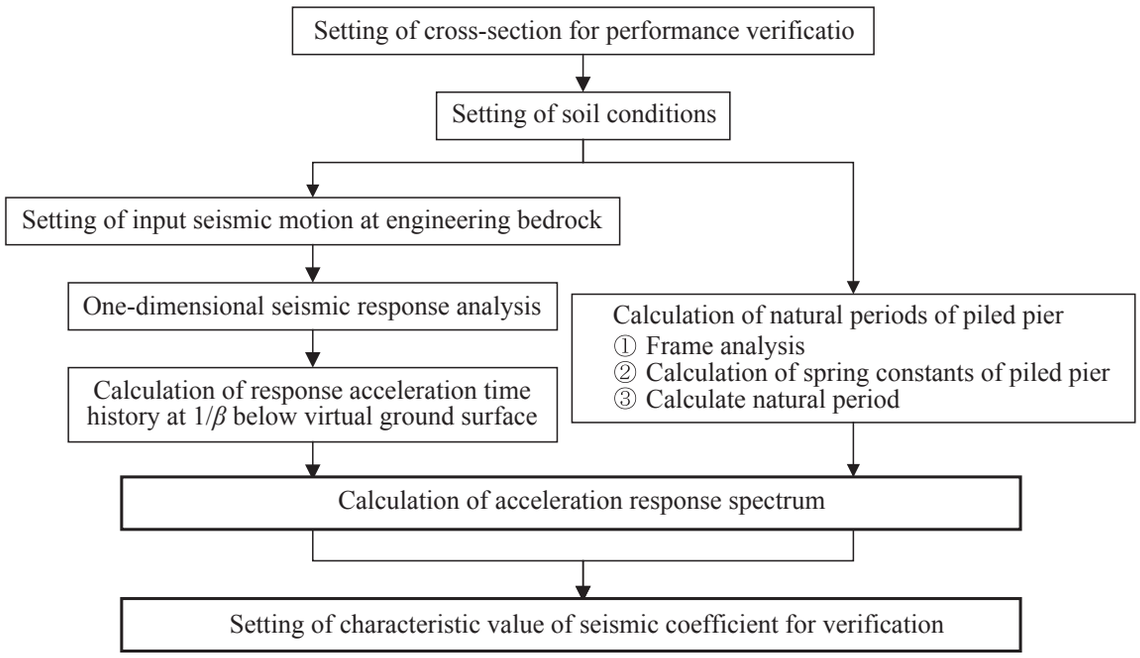


Fig. 5.2.6 Typical Procedure for Setting of Seismic Coefficient for Verification

③ Design value of seismic coefficient for verification

For variable situations under Level 1 earthquake ground motion, the minimum of the design value of seismic coefficient for verification is 0.05, and the maximum is 0.25. However, when the characteristic value of the seismic coefficient for verification exceeds 0.25, this value does not apply, and the characteristic value can be adopted as the design value of seismic coefficient for verification. In summary, the design value of seismic coefficient for verification is as follows.

$$k_{h_d} = \begin{cases} \gamma_{k_h} k_{h_k} & (0.05 \le k_{h_d} \le 0.25) \\ k_{h_k} & (0.25 < k_{h_k}) \end{cases} \tag{5.2.4}$$

where,

k_{hd} : design value of seismic coefficient for verification

k_{hk} : characteristic value of the seismic coefficient for verification

④ The natural periods of the piled pier may be calculated using a frame analysis. If the relationship between the displacement and load is obtained from the frame analysis, as shown in **Fig. 5.2.7**, when minute loads are acting on the piled pier, the spring constants of the piled pier can be set and the natural periods can be obtained from equation **(5.2.5)**. The ground spring constants used in the frame analysis may be calculated using equation **(5.2.2)**.

$$T_s = 2\pi\sqrt{\frac{W}{gK}} \tag{5.2.5}$$

where,

T_s : natural period of piled pier (s)

W : self weight and static load during an earthquake borne by one row of pile group (kN)

g : gravitational acceleration (m/s^2)

K : spring constant of the piled pier (kN/m)

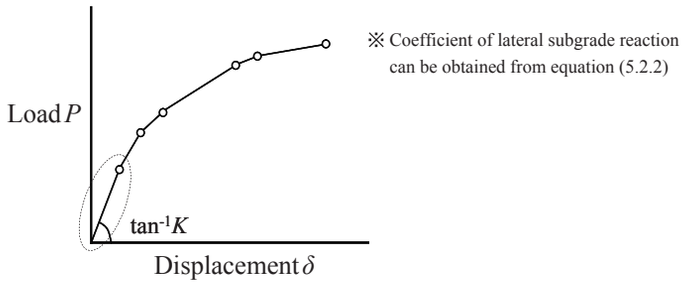
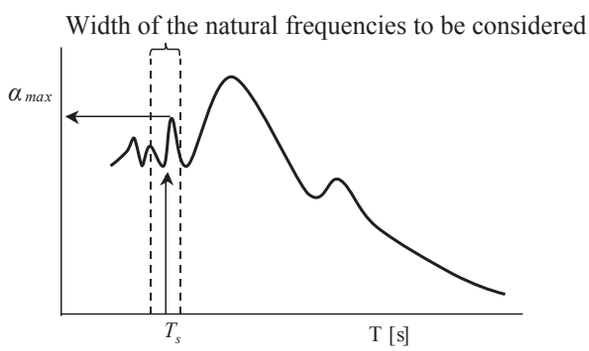


Fig. 5.2.7 Relationship between Load and Displacement from Frame Analysis

⑤ The natural period of the piled pier obtained from the spring constants of the piled pier by frame analysis usually involves some amount of errors. Therefore, if the value in the acceleration response spectrum corresponding to the natural period is a local minimum, the seismic coefficient for verification could be underestimated, and this should not be applied as it is. In addition, as indicated in **5.2.5 Performance Verification of Structural Members**, repeated verification for the variable situation under Level 1 earthquake ground motion is needed. Therefore, it is preferable that the spectral value be determined to calculate the seismic coefficient for verification with a certain range of natural periods. Thus, the number of repetitions of the performance verification may be reduced. However, this does not deny the importance of avoiding a local maximum in the acceleration response spectrum caused by the site effects. In the case that the natural period of the piled pier corresponds to a local maximum in the acceleration response spectrum, it is very likely that the cross-section will not be optimum from the viewpoint of seismic resistance performance and cost. It is necessary to pay attention to this point for setting the cross-section for verification.



α_{max}: Maximum value of acceleration used to determine the seismic coefficient for verification
T_s: Natural period of the piled pier calculated by frame analysis

Fig. 5.2.8 Consideration of Natural Period in Acceleration Response Spectrum

⑥ Seismic coefficient for verification used in performance verification of seismic-resistant of earth-retaining sections

(a) General

performance verification of seismic-resistant of earth-retaining sections can be carried out by directly evaluating the deformation of the earth-retaining section using a detailed method such as non-linear effective stress analysis. But simple methods such as the seismic coefficient method can be also used. In this case, it is necessary to appropriately set the seismic coefficient for verification used in the performance verification corresponding to the amount of deformation of the facility, considering the effect of the frequency characteristics of the ground motion and the duration. The normal procedure of calculating the seismic coefficient for verification is as shown in **Fig. 5.2.9**. For the calculation of the seismic coefficient for verification of earth-retaining sections of gravity-type, basically refer to **2.2.2 Actions**, prepared for gravity-type quaywalls. However, setting the filter taking into consideration the frequency characteristics as shown by the thick lines is different from gravity-type quaywalls, and this point should be carefully reflected in the analysis.

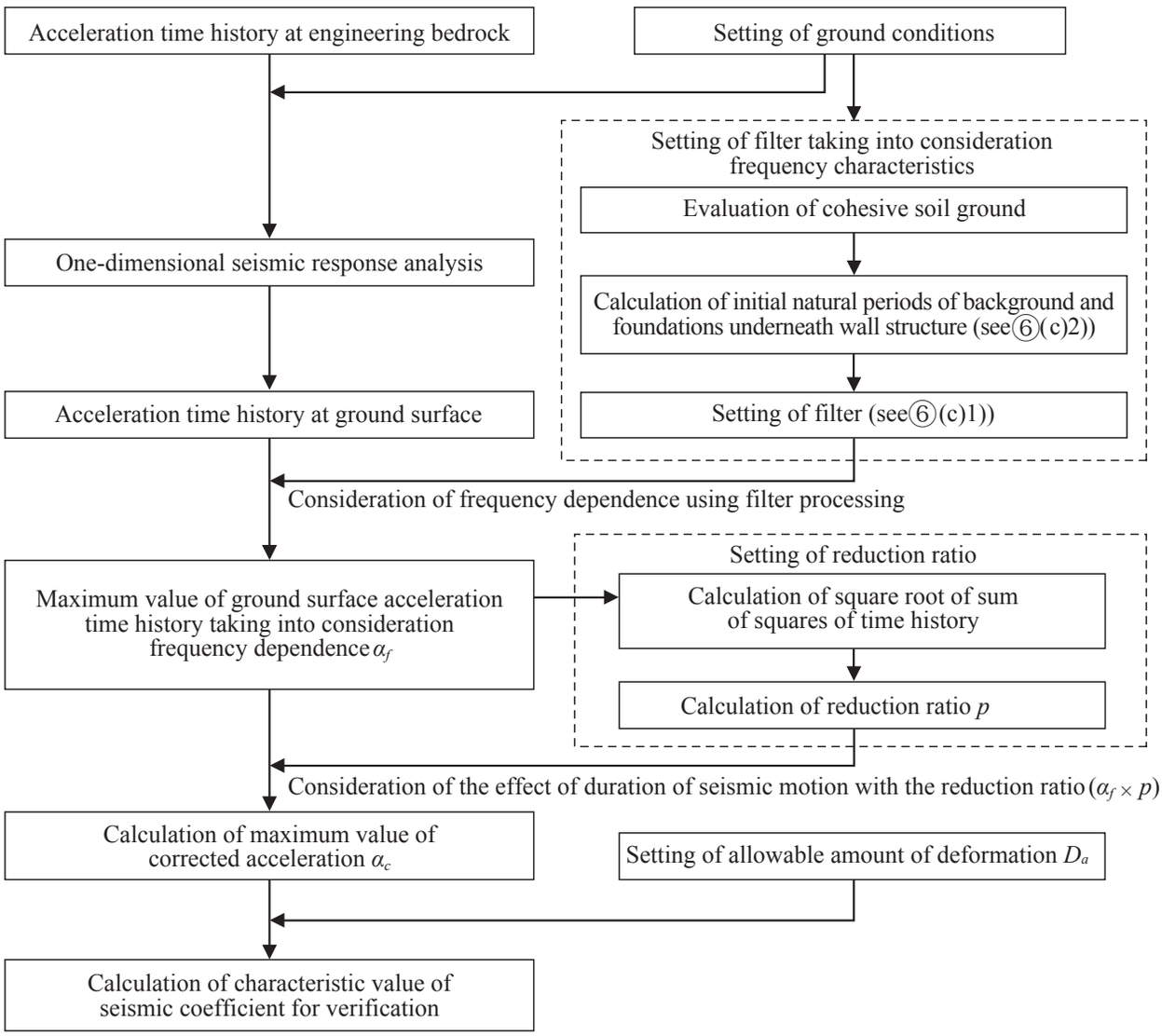


Fig. 5.2.9 Example of Procedure for Calculating Seismic Coefficient for Verification

(b) For the basic flow and points to be noticed in calculating the seismic coefficient for verification of earth-retaining sections of gravity-type structures, **2.2.2, Actions** for gravity-type quaywalls may be referred to. However, it is necessary to consider the effect on the deformation of the earth-retaining section influenced by the slopes at the front of the earth-retaining section and deep rubble mound. And thus setting of the filter considering the frequency characteristics shall be done by the calculation method described below.

(c) Setting of the filter considering the frequency characteristics

1) Setting of the filter
The filter obtained from equation **(2.2.1)** of **2.2.2 Actions** for gravity-type quaywalls may be used as the filter in consideration of the frequency characteristics of the ground motion used in verification of the earth-retaining section of gravity structures. However, as shown in **Fig. 5.2.10**, the height from the virtual ground surface to the top of the earth-retaining section may be substituted for the wall height H. The value of b may be set as the range of values indicated by equation **(5.2.6)** using the height H from the virtual ground surface to the top of the earth-retaining section.

$$0.04H + 0.08 \leq b \leq 0.08H + 0.44 \tag{5.2.6}$$

where,
H : Height from the virtual ground surface to the top of the earth-retaining section (m)

2) Calculation of the natural period of the background soils and soils underneath the wall structure
The method of calculation of the initial natural period T_b of the background soils used in setting the frequency filter that takes into consideration the ground motion of the earth-retaining section of gravity-type structures may be the same as the method for gravity-type quaywalls. Also, the initial natural period T_u of the soils underneath the wall structure may be calculated by evaluating the section from the virtual ground surface including rubble mound down to the seismic bedrock as a ground, and ignoring the ground from the virtual ground surface up to the bottom of the wall structure. In the case of gravity-type quaywalls, the T_u used in setting the filter is evaluated replacing the material properties of the original ground with the material properties of the rubble mound. However, when calculating the T_u of earth-retaining section of gravity-type structures, this may not be applied, so it is necessary to be careful about this. In other words, T_b and T_u should be calculated at the positions shown in **Fig. 5.2.10**.

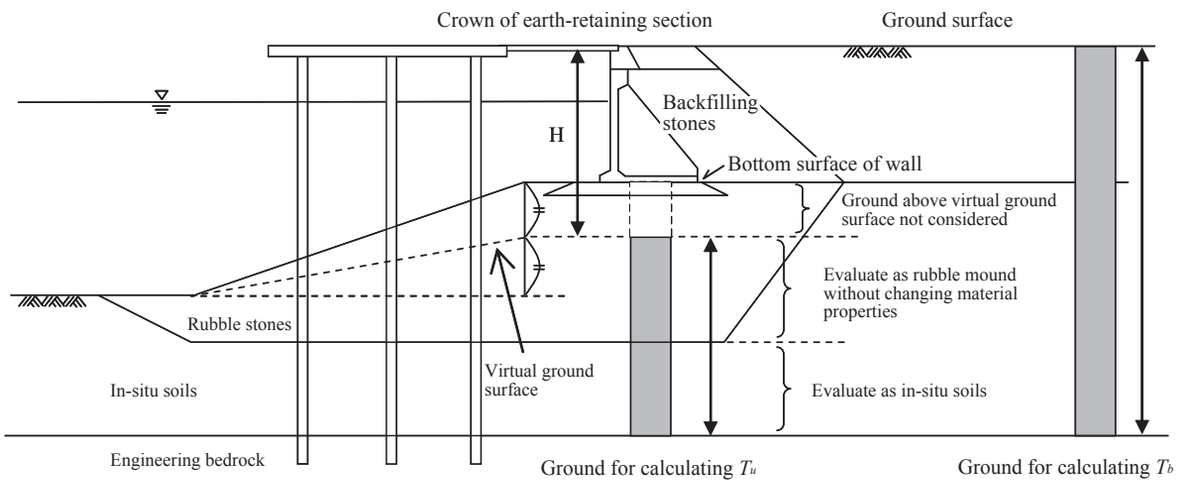


Fig. 5.2.10 Ground Calculation of Natural Periods

5.2.4 Performance Verification

(1) Items to be considered in the performance verification of open-type wharves on vertical piles
In the performance verification of open-type wharves on vertical piles, the necessary items among the following items shall be appropriately investigated and set as necessary.

① The cross-sectional forces in the superstructure (Variable situations: action of ships, Level 1 earthquake ground motion, surcharge and action of waves, accidental situations: Level 2 earthquake ground motion)

② Fatigue failure of the superstructure (Variable situations: repeated actions of surcharge)

③ Stresses in piles (Variable situations: action of ships, Level 1 earthquake ground motion and surcharge, Accidental situation: Level 2 earthquake ground motion)

④ Bearing capacity of piles (Variable situations: action of ships, Level 1 earthquake ground motion, surcharge and action of waves, accidental situations: Level 2 earthquake ground motion)

⑤ Deformation (accidental situations: Level 2 earthquake ground motion)
Performance verification under Level 2 earthquake ground motion shall be in accordance with **(11) Verification of Level 2 Earthquake Ground Motions with a Dynamic Analysis Method**. For the cross-sectional forces in the superstructure and fatigue failure, refer to **5.2.5 Performance Verification of Structural Members**.

(2) In the performance verification of the piled pier section of open-type wharves on vertical piles as described below, no load transmission is considered from the earth-retaining section to the wharves. A piled pier is a very flexible structure if affected by deformation of the ground, hence, piled pier section shall be structurally independent of earth-retaining section. However, in the case where the cross-sectional dimensions are such that it is not possible to eliminate the effect from the earth-retaining section, because of physical restrictions due to ground condition, it is necessary to carry out the verification using a method considering the interaction between the earth-retaining section and the piled pier section.[7)]

(3) In the performance verification for Level 1 earthquake ground motion, the seismic coefficient for verification is calculated from the acceleration response spectrum values corresponding to the natural periods of the piled pier, thus, when the dimensions of the piles are not determined, it is not possible to determine the natural periods of the

piled pier either. Therefore, the dimensions of the piles are assumed, and the seismic coefficient for verification is calculated from the acceleration response spectrum corresponding to the natural periods, then the verification is carried out. If the performance requirements are not satisfied, the pile dimensions are changed, and the same calculation needs to be repeated.

(4) Performance verification of the deformation may be carried out by setting an appropriate limiting value taking into consideration the dynamic deformation of the piled pier. For example, the amount of deformation to ensure that the access bridge does not fall down may be taken as the limiting value. In that case, it is appropriate to use the response displacement considering the dynamic action, such as the displacement response spectrum, and not the displacement considering the static action.

(5) Performance verification for stresses in the piles under design situation for other than accidental situations in respect of Level 2 earthquake ground motion

① Verification of the stresses occurring in the piles of a piled pier may be carried out using equation **(5.2.7)**. In the following equations, the symbol γ is the partial factor corresponding to the suffix, where the suffixes d and k indicate the design value and characteristic value respectively.

(a) When the axial forces are tensile

$$\sigma_{t_d} + \sigma_{bt_d} \le \sigma_{ty_d} \text{ and } -\sigma_{t_d} + \sigma_{bc_d} \le \sigma_{by_d}$$

(b) When the axial forces are compressive

$$\frac{\sigma_{c_d}}{\sigma_{cy_d}} + \frac{\sigma_{bc_d}}{\sigma_{by_d}} \le 1.0 \qquad (5.2.7)$$

where,

σ_t, σ_c : tensile stress due to axial tensile forces acting on the cross-section, and compressive stress due to axial compressive forces, respectively (N/mm²)

σ_{bt}, σ_{bc} : maximum tensile stress and maximum compressive stress due to the flexural moment acting on the cross-section, respectively (N/mm²)

σ_{ty}, σ_{cy} : tensile yield stress and axial compressive yield stress for the weak axis, respectively (N/mm²)

σ_{by} : bending compressive yield stress (N/mm²)

The design values in the equations may be calculated from equation **(5.2.8)**. The values shown in **Table 5.2.2** may be used as the partial factors in the equations.

$$\sigma_{td} = \frac{P_d}{A}, \quad \sigma_{cd} = \frac{P_d}{A}, \quad \sigma_{btd} = \frac{M_d}{Z}, \quad \sigma_{bcd} = \frac{M_d}{Z},$$

$$\sigma_{tyd} = \gamma_{\sigma_y}\sigma_{tyk}, \quad \sigma_{cyd} = \gamma_{\sigma_y}\sigma_{cyk}, \quad \sigma_{byd} = \gamma_{\sigma_y}\sigma_{byk} \qquad (5.2.8)$$

where,

A : cross-sectional area of piles (mm²)

P : axial force on pile (N)

Z : section modulus of piles (mm³)

M : flexural moment of piles (N·mm)

② For the yield stress of piles, refer to **Part II, Chapter 11, 2 Steel**. The axial compressive yield stress may be calculated from the equation in **Table 5.2.1**.

Table 5.2.1 Axial Compressive Yield Stresses (N/mm²)

SKK400 SHK400 SHK400M SKY400	SKK490 SHK490M SKY490
a) When $\frac{\ell}{r} \le 18$ 235	a) When $\frac{\ell}{r} \le 16$ 315
b) When $18 < \frac{\ell}{r} \le 92$ $235 - 1.38\left(\frac{\ell}{r} - 18\right)$	b) When $16 < \frac{\ell}{r} \le 79$ $315 - 2.04\left(\frac{\ell}{r} - 16\right)$
c) When $\frac{\ell}{r} > 92$ $\frac{2.01 \times 10^6}{6.7 \times 10^3 + \left(\frac{\ell}{r}\right)^2}$	c) When $\frac{\ell}{r} > 79$ $\frac{2.04 \times 10^6}{5.0 \times 10^3 + \left(\frac{\ell}{r}\right)^2}$

ℓ: Effective buckling length of member (cm), r: Radius of gyration of member gross cross-section (cm)

③ The design values of cross-sectional forces on the piles can be calculated by multiplying the characteristic values of parameters such as the coefficient of subgrade reaction, the action in the horizontal direction, and other probabilistic variables by the partial factors.

④ It is preferable to calculate the flexural moments on the piles for the direction both normal and parallel to the face line of the wharf. As in the example shown in **Fig. 5.2.1**, if the ground surface under the floor slab of the piled pier has a sloping surface, it is often the case that the flexural moments in the frontmost row of piles are maximized when the ground motion acts in the direction parallel to the face line.

⑤ When it is considered necessary to examine the rotation of the piled pier unit when evaluating the actions, the verification should take this into consideration. In this case the distribution of forces on each pile may be evaluated as described below.

(a) When the symmetry axis of the piled pier unit is perpendicular to the face line of the wharf and the direction of action of the horizontal force is parallel to the symmetry axis as shown in **Fig. 5.2.11**, the horizontal force may be calculated by equation (**5.2.9**).

$$H_i = \frac{K_{H_i}}{\sum_i K_{H_i}} H + \frac{K_{H_i} x_i}{\sum_i K_{H_i} x_i^2} eH \tag{5.2.9}$$

where

H_i : horizontal force on pile (kN)

K_{H_i} : horizontal spring constant of pile (kN/m)

$$K_{H_i} = \frac{12EI_i}{\left(h_i + \frac{1}{\beta_i}\right)^3}$$

h_i : vertical distance between the pile head and the virtual ground surface (m)

β_i : inverse of the distance between the virtual ground surface and the virtual fixed point of pile (m^{-1})

EI_i : flexural rigidity of pile ($kN \cdot m^2$)

H : horizontal force acting on the unit (kN)

e : distance between the block's symmetry axis and the horizontal force (m)

x_i : distance between the unit's symmetry axis and each pile (m)

The subscript i refers to the i-th pile.

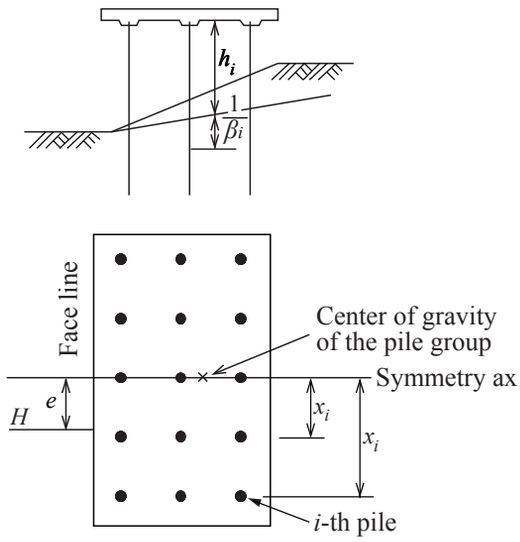


Fig. 5.2.11 Distance between the Center of Gravity of the Pile Group and Individual Piles

(b) The row of piles bearing the maximum total horizontally distributed forces is subject to the verification.

(c) When obtaining K_{H_i}, it is necessary to appropriately set the coefficient of subgrade reaction in the lateral direction of the ground, and calculate β.

⑥ Apart from accidental situations in respect of Level 2 earthquake ground motion, basically the performance is prescribed by yielding of the edge of the pile head. However, the piled pier is characterized with structural robustness, which means the capacity of structure may not be fatally damaged by local failure caused by ground motions, to the extent that the original function of the structure is lost. The reliability index for yielding of the edge of the pile within the ground is reported about 2.0 – 2.7 larger than that of the pile head.[8)]

(6) Performance verification of the bearing capacity in piles under design situations other than accidental situations in respect of Level 2 earthquake ground motion

① Verification of the bearing capacity of piles in piled piers can be carried out appropriately in accordance with **Chapter 2, 2.4.3 Static Maximum Axial Pushing Resistance of Piles Foundations**, and **Chapter 2, 2.4.4 Static Maximum Pulling Resistance of Piles Foundations**, corresponding to the ground characteristics and an analysis method for pile lateral resistance. In this case, for calculating the bearing capacity of piles on a sloping surface, the soil strata below the virtual ground surface can be considered as the effective bearing strata.

② Regarding the virtual ground surface, refer to **5.2.2 Setting the Basic Cross-section**.

(7) Partial factors under the design situations other than accidental situations in respect of Level 2 earthquake ground motion

① Regarding partial factors for stresses occurring in the piles of open-type wharves on vertical piles and partial factors for the bearing capacity of piles, refer to **Table 5.2.2**. The target reliability indices and target failure probabilities for stresses in piles shown in 1) and 4) of **Table 5.2.2** mean the values for edge yielding of the pile head of each single pile in the piled pier. In the table, for the variable situations in respect of the action of ships, the reliability index is 4.1 (failure probability of 2.3×10^{-5}), being based on the average level of safety in the conventional design methods. When the expected total cost represented by the sum of the initial cost and the expected value of the restoration cost due to failure is taken into consideration, the reliability index that minimizes the expected total cost is 3.2 (failure probability of 9.1×10^{-4}) for high earthquake-resistance facilities, and 2.9 (failure probability of 1.9×10^{-3}) for other piled piers.[9)] If here the level of safety is evaluated from reliability theory based on minimization of the expected total cost, the partial factors are as shown in **Table 5.2.2 1)**.[9)] Concerning the variable situations in respect of Level 1 earthquake ground motion shown in the **Table 5.5.2 (4)**, the average level of safety of a piled pier in accordance with the conventional design methods is evaluated and shown. Besides the above, the partial factors of **Table 5.2.2** are defined taking into consideration the settings based on the conventional design methods.

Table 5.2.2 Standard Partial Factors
(1) Variable situations in respect of the action of ships
(ship berthing, traction by ships), Variable situations in respect of surcharge (during operation)
(a) When SKK400 is used

				High earthquake-resistance facility				
Target reliability index β_T				3.2				
Target failure probability P_{fT}				9.1×10^{-4}				
				γ	α	μ/X_k	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		1.00	0.719	1.260	0.08	Normal
	$\gamma_{k_{CH}}$	Coefficient of subgrade reaction		0.60	0.257	1.333	0.76	Log normal
	γ_{P_H}	Horizontal forces		1.35	-0.645	0.870	0.25	Normal
	γ_q	Surcharges		1.00	-	-	-	-
	γ_a	Structural analysis coefficient		1.00	-	-	-	-
				Other than high earthquake-resistance facilities				
Target reliability index β_T				2.9				
Target failure probability P_{fT}				1.9×10^{-3}				
				γ	α	μ/X_k	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		1.00	0.719	1.260	0.08	Normal
	$\gamma_{k_{CH}}$	Coefficient of subgrade reaction		0.60	0.257	1.333	0.76	Log normal
	γ_{P_H}	Horizontal forces		1.30	-0.645	0.870	0.25	Normal
	γ_q	Surcharges		1.00	-	-	-	-
	γ_a	Structural analysis coefficient		1.00	-	-	-	-
				All open-type wharves on vertical piles				
				γ	α	μ/X_k	V	
Bearing capacity	$\gamma_{c'}$	Cohesion		1.00	-	-	-	
	γ_N	N-value		1.00	-	-	-	
	γ_a	Structural analysis coefficient	Pulling piles	0.33	-	-	-	
			Pushing piles	0.40	-	-	-	

※1: α: Sensitivity factor, μ/X_k: Deviation of average value (average value / characteristic value), V: coefficient of variation.
※2:Horizontal forces include fender reaction forces (during ship berthing), tractive forces (during traction), and crane horizontal forces (during operation of the crane).
※3:The design value of axial forces in piles used in the verification of bearing capacity can be obtained from the verification of stresses in piles.

Table 5.2.2 Standard Partial Factors
(1) Variable situations in respect of the action of ships
(ship berthing, traction by ships), Variable situations in respect of surcharge (during operation)
(b) When SKK490 is used

				High earthquake-resistance facility				
Target reliability index β_T				3.2				
Target failure probability P_{fT}				9.1×10^{-4}				
				γ	α	μ/X_k	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		0.95	0.719	1.196	0.08	Normal
	$\gamma_{k_{CH}}$	Coefficient of subgrade reaction		0.60	0.257	1.333	0.76	Log normal
	γ_{P_H}	Horizontal forces		1.35	-0.645	0.870	0.25	Normal
	γ_q	Surcharges		1.00	-	-	-	-
	γ_a	Structural analysis coefficient		1.00	-	-	-	-
				Other than high earthquake-resistance facilities				
Target reliability index β_T				2.9				
Target failure probability P_{fT}				1.9×10^{-3}				
				γ	α	μ/X_k	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		0.95	0.719	1.196	0.08	Normal
	$\gamma_{k_{CH}}$	Coefficient of subgrade reaction		0.60	0.257	1.333	0.76	Log normal
	γ_{P_H}	Horizontal forces		1.30	-0.645	0.870	0.25	Normal
	γ_q	Surcharges		1.00	-	-	-	-
	γ_a	Structural analysis coefficient		1.00	-	-	-	-
				All open-type wharves on vertical piles				
				γ	α	μ/X_k	V	
Bearing capacity	$\gamma_{c'}$	Cohesion		1.00	-	-	-	
	γ_N	N-value		1.00	-	-	-	
	γ_a	Structural analysis coefficient	Pulling piles	0.33	-	-	-	
			Pushing piles	0.40	-	-	-	

※1: α: Sensitivity factor, μ/X_k: Deviation of average value (average value / characteristic value), V: .Coefficient of variation.
※2: Horizontal forces include fender reaction forces (during the ship berthing), tractive forces (during traction), and crane horizontal forces (during operation of the crane).
※3:The design value of axial forces in piles used in verification of bearing capacity can be obtained from the verification of stresses in piles.

(2) Variable situations in respect of surcharges (during strong winds)

				All facilities			
				γ	α	μ/X_k	V
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		1.00	-	-	-
	$\gamma_{k_{CH}}$	Coefficient of subgrade reaction		1.00	-	-	-
	γ_{PH}	Horizontal forces		1.00	-	-	-
	γ_q	Surcharges		1.00	-	-	-
	γ_a	Structural analysis coefficient		1.12	-	-	-
Bearing capacity	$\gamma_{c'}$	Cohesion		1.00	-	-	-
	γ_N	N-value		1.00	-	-	-
	γ_a	Structural analysis coefficient	Pulling piles	0.40	-	-	-
			Pushing: end bearing piles	0.66	-	-	-
			Pushing: friction piles	0.50	-	-	-

※1: α: Sensitivity factor, μ/X_k: Deviation of average value (average value / characteristic value), V: coefficient of variation.
※2:The design value of axial forces in piles used in the verification of bearing capacity can be obtained from the verification of stresses in piles.

Table 5.2.2 Standard Partial Factors

(3) Variable situations in respect of the action of waves

				All facilities			
				γ	α	μ/X_k	V
Bearing capacity	γ_P	Axial forces in piles		1.00	-	-	-
	$\gamma_{c'}$	Cohesion		1.00	-	-	-
	γ_N	N-value		1.00	-	-	-
	γ_a	Structural analysis coefficient	Pulling piles	0.40	-	-	-
			Pushing: end bearing piles	0.66	-	-	-
			Pushing: friction piles	0.50	-	-	-

(4) Variable situations in respect of Level 1 earthquake ground motion

(a) When SKK400 is used

				High earthquake-resistance facility (specially designated)			
Target reliability index β_T				3.65			
Target failure probability P_{fT}				1.3×10^{-4}			
				γ	α	μ/X_k	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		1.00	0.423	1.260	0.08	Normal
	γ_{kCH}	Coefficient of subgrade reaction		0.66	0.194	1.333	0.76	Log normal
	γ_{kh}	Horizontal forces		1.68	-0.885	1.000	0.20	Log normal
	γ_q	Surcharges		1.00	-	-	-	-
	γa	Structural analysis coefficient		1.00	-	-	-	-
				High earthquake-resistance facility (standard)				
Target reliability index β_T				2.67				
Target failure probability P_{fT}				3.8×10^{-3}				
				γ	α	μ/X_k	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		1.00	0.443	1.260	0.08	Normal
	γ_{kCH}	Coefficient of subgrade reaction		0.72	0.215	1.333	0.76	Log normal
	γ_{kh}	Horizontal forces		1.36	-0.870	1.000	0.20	Log normal
	γ_q	Surcharges		1.00	-	-	-	-
	γ_a	Structural analysis coefficient		1.00	-	-	-	-
				Other than high earthquake-resistance facilities				
Target reliability index β_T				2.19				
Target failure probability P_{fT}				1.4×10^{-2}				
				γ	α	μ/X_k	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		1.00	0.455	1.260	0.08	Normal
	γ_{kCH}	Coefficient of subgrade reaction		0.80	0.195	1.333	0.76	Log normal
	γ_{kh}	Horizontal forces		1.23	-0.869	1.000	0.20	Log normal
	γ_q	Surcharges		1.00	-	-	-	-
	γ_a	Structural analysis coefficient		1.00	-	-	-	-
				All open-type wharves on vertical piles				
				γ	α	μ/X_k	V	
Bearing capacity	γ_c'	Cohesion		1.00	-	-	-	
	γ_N	N-value		1.00	-	-	-	
	γ_a	Structural analysis coefficient	Pulling pile	0.40	-	-	-	
			Pushing: end bearing pile	0.66	-	-	-	
			Pushing: friction pile	0.50	-	-	-	

※1: α: Sensitivity factor, μ/X_k: Deviation of average value (average value / characteristic value), V: coefficient of variation.
※2: The design value of axial forces in piles used in the verification of bearing capacity can be obtained from the verification of stresses in piles.

Table 5.2.2 Standard Partial Factors

(4) Variable situations in respect of Level 1 earthquake ground motion
(b) When SKK490 is used

				High earthquake-resistance facility (specially designated)				
Target reliability index β_T				3.65				
Target failure probability P_{fT}				1.3×10^{-4}				
				γ	α	μ/X_k	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		1.00	0.423	1.196	0.08	Normal
	γ_{kCH}	Coefficient of subgrade reaction		0.66	0.194	1.333	0.76	Log normal
	γ_{kh}	Horizontal forces		1.77	-0.885	1.000	0.20	Log normal
	γ_q	Surcharges		1.00	-	-	-	-
	γ_a	Structural analysis coefficient		1.00	-	-	-	-
				High earthquake-resistance facility (standard)				
Target reliability index β_T				2.67				
Target failure probability P_{fT}				3.8×10^{-3}				
				γ	α	μ/X_k	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		1.00	0.443	1.196	0.08	Normal
	γ_{kCH}	Coefficient of subgrade reaction		0.72	0.215	1.333	0.76	Log normal
	γ_{kh}	Horizontal forces		1.43	-0.870	1.000	0.20	Log normal
	γ_q	Surcharges		1.00	-	-	-	-
	γ_a	Structural analysis coefficient		1.00	-	-	-	-
				Other than high earthquake-resistance facilities				
Target reliability index β_T				2.19				
Target failure probability P_{fT}				1.4×10^{-2}				
				γ	α	μ/Xk	V	Probability distribution
Pile stress	$\gamma_{\sigma y}$	Steel yield strength		1.00	0.455	1.196	0.08	Normal
	γ_{kCH}	Coefficient of subgrade reaction		0.80	0.195	1.333	0.76	Log normal
	γ_{kh}	Horizontal forces		1.30	-0.869	1.000	0.20	Log normal
	γ_q	Surcharges		1.00	-	-	-	-
	γ_a	Structural analysis coefficient		1.00	-	-	-	-
				All open-type wharves on vertical piles				
				γ	α	μ/X_k	V	
Bearing capacity	$\gamma_{c'}$	Cohesion		1.00	-	-	-	
	γ_N	N-value		1.00	-	-	-	
	γ_a	Structural analysis coefficient	Pulling pile	0.40	-	-	-	
			Pushing: end bearing pile	0.66	-	-	-	
			Pushing: friction pile	0.50	-	-	-	

※1: α: Sensitivity factor, μ/X_k: Deviation of average value (average value / characteristic value), V: coefficient of variation.
※2: The design value of axial forces in piles used in the verification of bearing capacity can be obtained from the verification of stresses in piles.

(8) Examination of Embedment Length for Lateral Resistance

① The embedment length of each vertical pile may be determined appropriately in accordance with the method of analysis of the pile lateral resistance.

② The embedment lengths of vertical piles are generally set at $3/\beta$ below the virtual ground surface based on the results of pile lateral resistance analyses. The value of β can be set in accordance with **5.2.2 Setting of Basic Cross Section**.

(9) Examination of Pile Joints

① When a pile joint is needed in a pile, it is preferable to ensure that the pile can keep its stability against the impact stress generated in the joint during driving.

② The location of pile joint shall be determined carefully in such a manner as to avoid the portion with excessive stress.

③ For the method for joining piles, refer to **Chapter 2, 2.4.6 [4] Joints of Piles**.

(10) Change of Plate Thickness or Material of Steel Pipe Pile

① Any change on the plate thickness or material along the same steel pipe pile shall be made in accordance with **Chapter 2**,**2.4.6 [5] Change of Plate Thickness or Material Type of Steel Pipe Piles**.

② The strengths of joints and portion with steel thickness change should be examined carefully because there are some examples in which piles of open-type wharves buckled at these portions due to ground deformation in a deep ground where no bending stresses are generated under normal load conditions.

(11) Verification of Level 2 Earthquake Ground Motion with a Dynamic Analysis Method

① For setting the cross-section for the verification, a nonlinear dynamic analysis of a spring-mass model with single mass or double masses if there is a container crane installed may be used. The system consists of a spring equivalent to the modeled load-displacement relationship of the piled pier structure obtained from an elastic-plastic analysis.

② If container cranes or other cargo handling equipment are installed on a piled pier, the seismic response characteristics of the piled pier may be greatly altered depending on the ratio of the mass of the cargo handling equipment to that of the piled pier and the ratio of their natural periods. Therefore, it is necessary to carry out a seismic response analysis that takes into consideration the coupled oscillations of the cargo handling equipment and the piled pier. For details, refer to **Chapter 7 Cargo Handling Facilities**, **2.2 Fundamentals of Performance Verification**.

③ Besides the inertia forces acting on the superstructure of the piled pier, factors that have an adverse effect on the piles include transmission of the deformation of the ground around the earth-retaining section to the superstructure through the access bridge, and transmission of forces to the piles when the soil around the piles moves towards the sea due to the deformation of the soils there. Therefore, a structure of the access bridge should be such that deformation of the soils around the ground earth-retaining section does not adversely affect the superstructure of the piled pier.

(12) Performance Verification for the Stability of the Earth-retaining Section

① The examination of the structural stability of the earth-retaining section of open-type wharf on vertical piles can be made in accordance with the performance criteria prescribed in **2.2 Gravity-type Quaywalls, 2.3 Sheet Pile Quaywalls** depending on its structural type.

② The superstructure and the earth-retaining section of an open-type wharf should be connected by a simply supported slab having clearances on its both ends or buffer material provided on the both ends of slab, in order to prevent the forces acting on the earth-retaining section from being transmitted to the superstructure. It is also preferable to prepare measures against the relatively uneven settlement between the wharf and the earth-retaining section. Furthermore, the clearance between the superstructure and the earth-retaining section should be determined appropriately by considering the dynamic deformation of the superstructure and the earth-retaining section.

③ The stability of the earth-retaining section of open-type wharf on vertical piles against circular slip failure should be examined by applying **Chapter 2, 3.2.1 Stability Analysis by Circular Slip Failure Surface**.

5.2.5 Performance Verification of Structural Members

(1) It shall be well confirmed that there will be no loss of the required function caused by deterioration of the concrete superstructure and the steel pipe pile substructure due to material degradation during the design working life. In particular there have been many cases where the performance requirements of concrete superstructures have not been achieved as a result of salt injury, so a detailed maintenance management plan should be prepared and carried out.

(2) It shall be verified that the flexural moment, axial force, and shear force acting on the connections between the steel pipe piles and the superstructure do not reach the ultimate limit state.

(3) In the performance verification of piled piers, the analysis is carried out by assuming that rigid connections between the pile heads and the concrete beams are formed. Then, it is necessary that the pile head flexural moment can be smoothly distributed to the pile head and the concrete beam. The flexural moment that can be distributed to the beam M_{ud} may be calculated using the following equation, ignoring the reinforcement connection plates or vertical ribs which are provided, as necessary.

$$M_{ud} = \frac{DL^2 f'_{cd}}{6\gamma_b} \tag{5.2.10}$$

where,

M_{ud} : flexural moment that can be distributed to the part of the pile embedded in the beam (N.mm)
D : diameter of steel pipe pile (mm)
L : embedded length of steel pipe pile (mm)
f'_{cd} : design value of compressive strength of beam concrete (N/mm^2)
γ_b : member factor

(4) It is assumed that axial forces are distributed by only the bond between the outer peripheral surface of the piles and the vertical ribs, which are provided, as necessary, and the concrete. In this case, the axial force that can be distributed, P_{ud}, can be calculated from the following equation.

$$P_{ud} = \frac{1}{\gamma_b}\left(L\varphi + 2A_p\right) f_{bo_d} \tag{5.2.11}$$

where,

P_{ud} : axial force that can be distributed to the part of the pile embedded (N)
L : embedded length of steel pipe pile (mm)
φ : outer perimeter of steel pipe pile (mm)
f_{bo_d} : design value of the bond strength between the pile and the concrete (N/mm^2)
$f_{bo_d} = 0.11\, f'^{2/3}_{ck} / \gamma_c$
f'_{ck} : characteristic value of the compressive strength of the concrete (N/mm^2)
γ_c : material coefficient of concrete (= 1.3)
A_p : area of vertical ribs bonding with concrete (mm^2)
γ_b : member factor (may be taken to be 1.0)

(5) It shall be verified that failure due to punching shear forces in the horizontal direction shall not occur in the beam at the end of which the steel pipe pile is embedded. In this case the punching shear resistance, V_{pc_d}, may be calculated from the following equation.

$$V_{pc_d} = 0.2\sqrt{f'_{c_d}}\ \beta_d \beta_p \beta_\gamma\ A_\tau / \gamma_b \tag{5.2.12}$$

where,

V_{pc_d} : design value of punching shear resistance in the horizontal direction (N)
f'_{cd} : design compressive strength of concrete (N/mm^2)
$\beta_d = \sqrt[4]{1/d}$ if β_d>1.5, β_d shall be taken to be 1.5
$\beta_p = \sqrt[3]{100 p_w}$ if β_p>1.5, β_p shall be taken to be 1.5
d : effective height (m)
p_w : ratio of reinforcement to concrete sections
$\beta_\gamma = 1.0$
A_τ : shear resistance area (mm^2)
γ_b : member factor (may be taken to be 1.3)

5.3 Open-type Wharves on Coupled Raking Piles

5.3.1 Fundamentals of Performance Verification

(1) The following may be applied to the open-type wharves with a structure in which the horizontal forces acting on the piled pier are distributed to coupled raking piles.

(2) The performance verification of open-type wharves on coupled raking piles may be carried out in accordance with **5.2.4 Performance Verification** for open-type wharves on vertical piles, as well as the following.

(3) The open-type wharf on coupled raking piles is a structure that resists the horizontal force acting on the wharf such as the seismic actions, fender reaction force, and tractive force of ships with coupled raking piles. Therefore, this type of wharf must be constructed on the ground that yields sufficient bearing capacity for coupled raking piles. Because the coupled raking piles are so laid out to resist the horizontal forces in the direction normal to the face line of the wharf, the horizontal displacement in that direction is smaller than that of open-type wharves on vertical piles. Coupled raking piles are seldom laid out to resist the horizontal forces in the direction of wharf face line. Therefore, it is preferable to examine the strength of the wharf against the horizontal force parallel to the face line in the same manner as the examination for open-type wharves on vertical piles.

(4) In the case of coupled raking piles, the piles come close to adjacent vertical piles and the earth-retaining section, so it is preferable that the layout of the piles be carefully determined considering the construction conditions and the conditions of use.

(5) For the procedure for performance verification of open-type wharves on coupled raking piles, refer to **Fig. 5.3.1** of **5.2.4 Performance Verification** for open-type wharves on vertical piles.

(6) Verification for the variable situations in respect of Level 1 earthquake ground motion may be carried out by obtaining the natural periods of the piled pier with frame analysis and calculating the seismic coefficient for verification with the acceleration response spectrum corresponding to the natural periods.

(7) An example of the cross-section of the open type wharf on coupled raking piles is shown in **Fig. 5.3.1**.

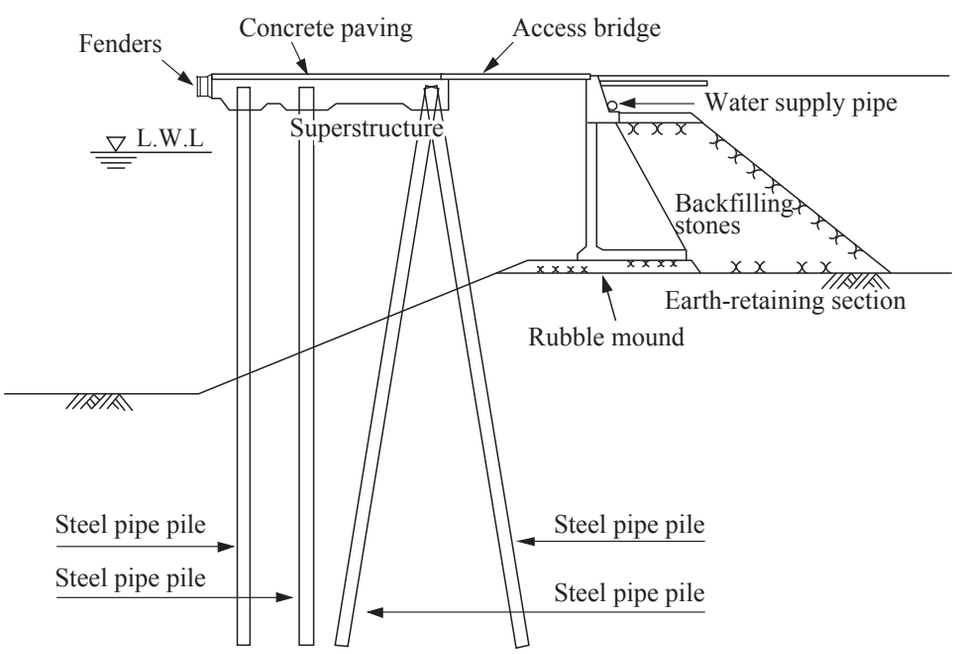


Fig. 5.3.1 Example of Cross-section of Open Type Wharf on Coupled Raking Piles

5.3.2 Setting of Basic Cross-section

(1) For setting the basic cross-section of open-type wharves on coupled raking piles, refer to **5.2.2 Setting of Basic Cross-section**.

(2) A large wharf for design ship size of 10,000 DTW class has one or two sets of coupled raking piles behind one vertical pile in the direction normal to the wharf face line. The distance between piles or between centers of

coupled raking piles is usually set to be 4 to 6 m in consideration of loading conditions and construction work. It is preferable to use a small raking angle of coupled piles from the viewpoint of securing resistance against horizontal force, but in many cases an inclination of 1: 0.33 to 1: 0.2 is used because of constraints related to the required distances from other piles and construction work-related constraints such as the capacity of pile driving equipment available.

5.3.3 Actions

The characteristic value of the seismic coefficient for verification used in performance verification of open-type wharves on coupled raking piles for the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics of the wharf. For calculation of the seismic coefficient for verification of open-type wharves on coupled raking piles, refer to **5.2.3(10) Ground Motion used in Performance Verification of Seismic-resistant**.

5.3.4 Performance Verification

(1) Items for the Performance Verification of Open-type Wharves on Coupled Raking Piles
The performance verification of open-type wharves on coupled raking piles shall apply **5.2.4 Performance Verification** and be based on the following.

(2) Performance Verification of Bearing Forces on Piles

① The pushing-in and pulling-out forces of each pair of coupled raking piles shall be calculated appropriately based on the vertical and horizontal forces defined in consideration of the wharf operation conditions.

② The pushing-in and pulling-out forces on each raking pile are obtained with a frame analysis method, taking into consideration the effect of the raking angle of the pile as indicated in **Chapter 2, 2.4.5 Static Maximum Lateral Resistance of Piles**, calculating the ratio of the coefficient of lateral subgrade reaction, and appropriately correcting the coefficient of lateral subgrade reaction.

③ For verification of pushing-in and pulling-out forces in each raking pile, refer to **Chapter 2 2.4.3 Static Maximum Axial Pushing Resistance of Pile Foundations**, and **2.4.4 Static Maximum Pulling Resistance of Pile Foundations**.

(3) Verification of Stresses in Piles
The cross-sectional stress in each pile may be calculated by applying **5.2.4 Performance Verification** for piles subject to axial forces or piles subject to axial forces and flexural moments.

(4) Horizontal Forces Distributed to the Pile Head of each Group when Rotation of the Piled Pier Block is Considered

① When it is necessary to consider rotation of the piled pier block, the horizontal forces distributed to the pile head of each group of piles in an open type wharf on coupled raking piles may be appropriately calculated in accordance with the cross-section of each pile and the raking angle and length of the raking piles. In this case, it may be assumed that all horizontal forces are distributed to the coupled raking piles. Normally the row of piles having the maximum distributed horizontal force among all the rows of piles is adopted as the row of piles used in the verification.

② In the case where the cross-section of each pile group and raking angle of the raking piles are different, the horizontal force distributed to the pile head of each group may be calculated using equation **(5.3.1)** (see **Fig. 5.3.2**).

(a) When the piles can be regarded as fully end bearing piles

$$H_i = \frac{C_i}{\sum_i C_i} H + \frac{C_i x_i}{\sum_i C_i x_i^2} eH \tag{5.3.1}$$

where,

$$C_i = \frac{\sin^2(\theta_{i1} + \theta_{i2})}{\frac{\ell_{i1}}{A_{i1}E_{i1}}\cos^2\theta_{i2} + \frac{\ell_{i2}}{A_{i2}E_{i2}}\cos^2\theta_{i1}} \quad \text{(N/m)}$$

H : horizontal force acting on the block (N/m)
H_i : horizontal force distributed to each pile (N/m)

e : distance between center line of pile group and the acting horizontal force (m)
x_i : distance from each pile group to the center line of a pile group (m)
ℓ_i : total pile length (m), being substituted the pile length of the friction pile when pulling-out forces are acting.
A_i : cross-sectional area of each pile (m^2)
E_i : Young's modulus of each pile (N/m^2)
θ_{i1}, θ_{i2} : angle of each pile with the vertical direction (°)

The subscript i refers to the i_{th} pile.
The subscripts 1, 2 refer to each pile in one pile group.
The center line of a pile group may be obtained from $\Sigma C_i \xi_i / \Sigma C_i$. ξ_i are the coordinates from an arbitrary coordinate origin of each pile group in face line direction.

(b) When the piles can be regarded fully as friction piles

1) Sandy soil
Equation **(5.3.1)** is used, substituting, $\dfrac{2\ell_i + \lambda_i}{3}$ for ℓ_i.

2) Cohesive soil
Equation **(5.3.1)** is used, substituting, $\dfrac{\ell_i + \lambda_i}{2}$ for ℓ_i.

where, λ_i: Pile length of the part over which the peripheral surface resistance force is not effectively working (m), ℓ_i: Total pile length (m).

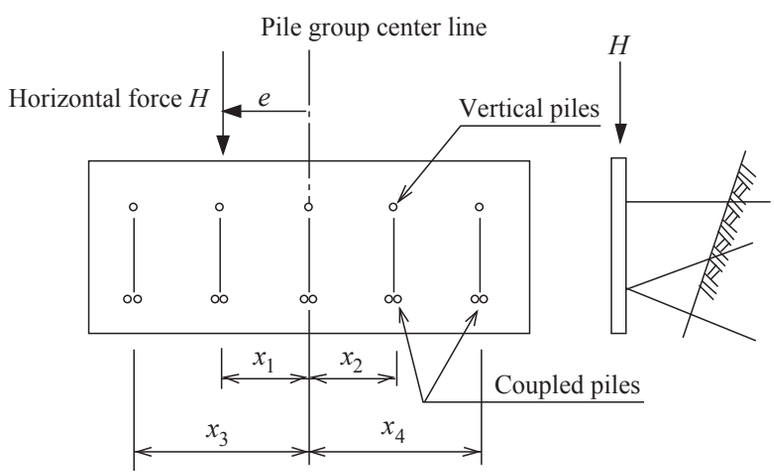


Fig. 5.3.2 Pile group Center Line and Distance from each Pile Group

③ When the cross-section, raking angle and length of the raking piles of each pile group are all equal, the horizontal force distributed to each pile group may be calculated from equation **(5.3.2)**.

$$H_i = \frac{1}{n} H + \frac{x_i}{\sum_i x_i^2} eH \tag{5.3.2}$$

(5) Partial Factors
Verification may be appropriately carried out using partial factors for verification of bearing capacity of piles and stresses in the piles of open-type wharves on coupled raking piles substituted with those for open-type wharves on vertical piles, considering the similarity of performance verification method among these two types of structures.

(6) Analysis in the Face Line Direction
If there are coupled raking piles in the face line direction, the analysis should be carried out using the method defined in **(2)** to **(5)**, in the same way as the direction perpendicular to the face line.

(7) Verification of Pile Embedment
For bearing capacity on raking piles, refer to **5.2.4 Performance Verification**.

(8) Performance Verification of Earth-retaining Sections

① For the performance verification of earth-retaining sections, refer to **5.2.4 Performance Verification**.

② It shall be ensured that the action due to deformation of the earth-retaining section by earthquakes shall not be transmitted to the superstructure of the piled pier via the access bridge, and that the piles are not adversely affected by significant deformation of the soil around the piles towards the sea.

5.4 Strutted Frame Type Pier

(1) Performance verification of strutted frame type piers shall apply **5.2 Open-type Wharves on Vertical Piles**, and **5.3 Open-type Wharves on Coupled Raking Piles**, and also refer to the **Strutted Frame Method Technical Manual**.[22)]

(2) The characteristic value of the seismic coefficient for verification used in the performance verification of strutted frame type piers against the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics. For calculation of the seismic coefficient for verification of strutted type piers, refer to **5.2.3(10) Ground Motions used in Performance Verification of Seismic-resistant**.

5.5 Jacket Type Piled Piers

Public Notice

Performance Criteria of Piled Piers

Article 55

1 The provisions of Article 48 shall be applied to the performance criteria of piled piers with modification as necessary.

2 In addition to the requirements of the preceding paragraph, the performance criteria of piled piers shall be as specified in the subsequent items:

(1) The access bridge of a piled pier shall satisfy the following criteria:

(a) It shall have the dimensions required for enabling the safe and smooth loading, unloading, embarkation and disembarkation, and others in consideration of the usage conditions.

(b) It shall not transmit the horizontal loads to the superstructure of the piled pier, and it shall not fall down even when the piled pier and the earth-retaining part are displaced owing to the actions of earthquakes or similar one.

(2) The following criteria shall be satisfied under the variable action situation in which the dominant actions are Level 1 earthquake ground motions, ship berthing and traction by ships, and imposed load:

(a) The risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(b) The risk that the axial forces acting in the piles may exceed the resistance capacity owing to failure of the ground shall be equal to or less than the threshold level.

(c) The risk that the stress in the piles may exceed the yield stress shall be equal to or less than the threshold level.

(3) The following criteria shall be satisfied under the variable action situation in which the dominant action is variable waves:

(a) The risk of losing the stability of the access bridge due to uplift acting on the access bridge shall be equal to or less than the threshold level.

(b) The risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(c) The risk that the axial forces acting in piles may exceed the resistance capacity owing to failure of the ground shall be equal to or less than the threshold level.

(4) In the case of structures having stiffening members, the risk of impairing the integrity of the stiffening members and their connection points under the variable action situation in which the dominant actions are variable waves, Level 1 earthquake ground motions, ship berthing and traction by ships, and imposed load shall be equal to or less than the threshold level.

3 The provisions of Article 49 through Article 52 shall be applied with modification as necessary to the performance criteria of the earth-retaining parts of piled piers in consideration of the structural type.

[Technical Note]

(1) The performance verification of jacket type piled piers or piled piers whose structure has stiffening members shall apply **5.2 Open-type Wharves on Vertical Piles**, and **5.3 Open-type Wharves on Coupled Raking Piles**, and for details refer to the **Jacket Method Technical Manual**.[23)]

(2) The characteristic value of the seismic coefficient for verification used in the performance verification of jacket type piled piers in the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics. For calculation of the seismic coefficient for the verification of jacket type piled piers, refer to **5.2.3(10) Ground Motions used in Performance Verification of Seismic-resistant**.

(3) Verification of Level 2 Earthquake Ground Motion with the Dynamic Analysis Method
The performance verification of jacket type piled piers in accidental situations in respect of level 2 earthquake ground motion shall be appropriately carried out considering the concerned circumstances around the facilities,

importance of the facility, and the accuracy of the method. The performance verification of jacket type piled piers may comply with that of open-type wharves on vertical piles, but the actions occurring in the members shall be appropriately set considering the structure of the trusses. The different points in the dynamic characteristics between jacket type piled piers and open-type wharves on vertical piles are as follows.

(a) The natural periods are short due to the nature of truss structure

(b) Because the structure has panel points, the failure mechanisms are complex

(c) Separate verification of the panel points is necessary

5.6 Dolphins

5.6.1 Fundamentals of Performance Verification

(1) The following may be applied to the performance verification of such mooring facilities as pile type, steel cell type, caisson type, and other type dolphin structures. Depending on their function, the types of dolphin include breasting dolphins, mooring dolphins and loading dolphins.

(2) The guidelines outlined in **5.6.2 Actions**, and **5.6.3 Performance Verification** may be used in simple verification methods, thus, this point should be noted when they are adopted.

(3) It is preferable that performance verification of dolphins be carried out considering the following items. For other items, it is preferable to appropriately carry out the performance verification in accordance with each structural form.

① The direction of actions on dolphins is not necessarily a constant direction, hence, the verification should be carried out for several directions, as necessary.

② Conventionally torsion in the case of pile type structures and rotation in the case of caisson type structures have not been examined very much. However, these factors may affect the stability of structures in certain cases, thus, it is necessary to be careful about these aspects.

③ It is preferable to appropriately set the crown height of the dolphin in accordance with its function. In this connection the position of installation of the fenders for breasting dolphins, the level of the deck of the ship for mooring dolphins, and the working range of the loading arm for loading dolphins should be taken into consideration. For connecting bridges, it is preferable that the height be sufficient not to be affected by the action of waves.

(4) An example of the cross-section of a pile type dolphin is shown in **Fig. 5.6.1**.

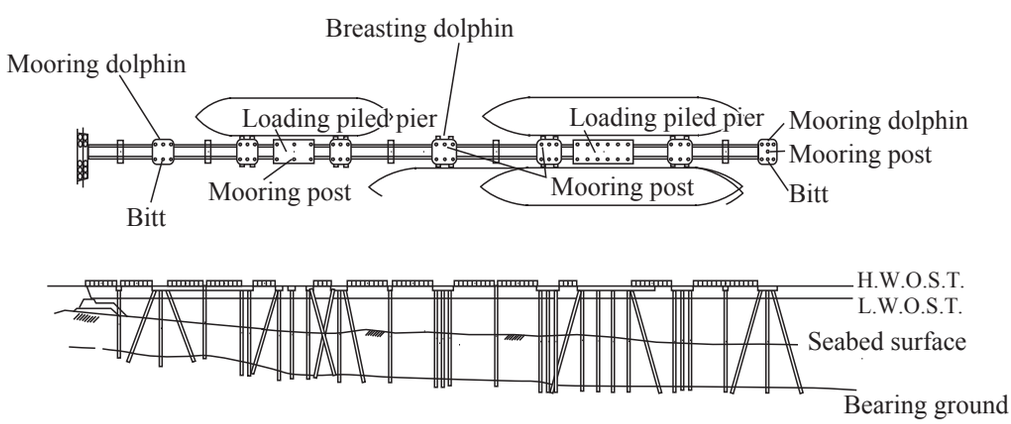


Fig. 5.6.1 Example of Cross-section of Pile Type Dolphin

(5) Layout

① The layout of a dolphin-berth shall be determined appropriately to avoid adverse effects on the navigation and anchorage of other ships in consideration of the dimensions of the design ships, water depth, wind direction, wave direction, and tidal currents.

② In the determination of the layout of breasting dolphins, the following items need to be examined:

(a) Dimensions of the design ship

1) The side of design ships is usually composed of curve lines forming the outlines of the bow and stern parts, each of which accounts for about 1/8 of the length overall (L) of ship, respectively and a straight line forming the outline of the central part which accounts for about 3/4 of the length overall (L) of ship. . It is preferable that the breasting dolphins are installed in such a way that the ships can be berthed to them with the straight line part. Normally the number of breasting dolphins is one each toward the bow and stern, but for dolphins serving for both large and small ships, two each toward bow and stern are sometimes provided.

2) When special cargo handling equipment is required for dolphins in such a case as dolphins for oil handling,

a cargo handling platform is installed midway between the breasting dolphins. In this case, it is preferable to locate the cargo handling platform with its seaside front slightly backward from that of the breasting dolphins, in order that the ship berthing force does not act directly on the cargo handling platform.

(b) It is preferable to layout dolphins in a way that the longitudinal axis of dolphins becomes parallel to the prevailing directions of winds, waves, and tidal currents. This helps to ease ship maneuvering during berthing and unberthing and to reduce external forces acting on the dolphins when the ship is moored.

③ Mooring dolphins are normally set at the positions with the angle of 45º from the rope bitts on ship's bow and stern, having a certain setback from the front face of the breasting dolphins.

④ The distance between breasting dolphins is closely related to the length overall (L) of the design ships. **Fig. 5.6.2** gives the relationship between the breasting dolphin interval and the water depth derived from the past construction data for reference.

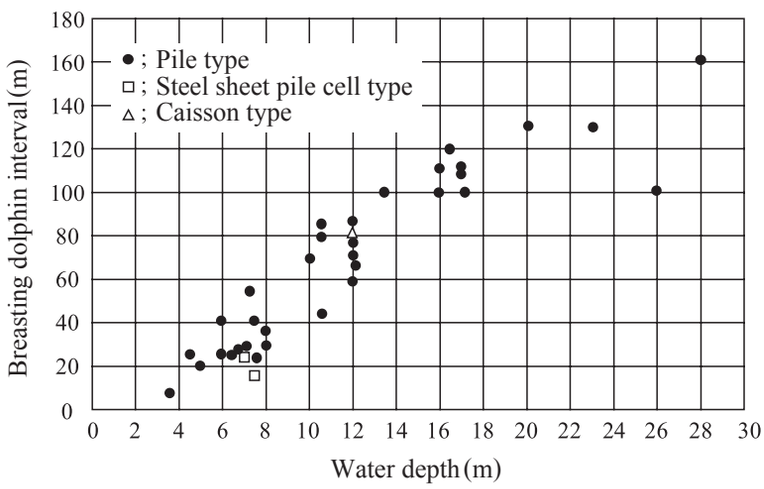


Fig. 5.6.2 Distance between Breasting Dolphins

5.6.2 Actions

(1) For calculation of the reaction force from the fenders onto the dolphins, refer to **Part II, Chapter 8, 2.2 Action Caused by Ship Berthing**, and **Chapter 5, 9.2 Fender Equipment**.

(2) For calculation of the tractive force of ships, refer to **Part II, Chapter 8, 2.4 Action due to Traction by Ships**.

(3) For calculation of vertical loads due to self weight and live load, refer to **Part II, Chapter 10, Self Weight and Surcharge, 5.2.3 Actions**, as applied for open-type wharves on vertical piles.

(4) For the action due to earthquakes, refer to **Part II, Chapter 4, Earthquakes** and **5.2.3 Actions**, as applied for open-type wharves on vertical piles.

(5) For calculation of the dynamic water pressure during an earthquake, refer to **Part II, Chapter 5, 2.2 Dynamic Water Pressure**.

(6) For calculation of wind pressure forces acting on cargo handling equipment, refer to **Part II, Chapter 2, 2.3 Wind Pressure**.

5.6.3 Performance Verification

[1] Pile Type Dolphins

(1) For the performance verification of pile type dolphins, refer to **5.2 Open-type Wharves on Vertical Piles**, and **5.3 Open-type Wharves on Coupled Raking Piles**.

(2) The characteristic value of the seismic coefficient for the verification used in the performance verification of pile type dolphins in variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics. For calculation of the seismic coefficient for the verification of pile type dolphins, refer to **5.2.3(10) Ground Motions used in Performance Verification of Seismic-resistant**.

(3) In the case of pile type dolphins, the berthing energy may normally be calculated on the assumption that it is absorbed by the deformations of the fenders and the piles.

(4) Large tankers are usually berthed at a slant angle with the dolphin alignment line. As the characteristics of fenders vary depending on the berthing angle, it is recommended in such a case to use the characteristics curve appropriate to the berthing angle. In addition, a slanting berthing entails the risk that some of the fenders attached to a breasting dolphin may not absorb the berthing energy effectively. Therefore, it is preferable to examine carefully which fenders will come in contact with the hull of ship in consideration of the berthing angle.

[2] Steel Cell Type Dolphins

(1) For the performance verification of steel cell type dolphins, refer to **2.9 Cellular-bulkhead Quaywalls with Embedded Sections**.

(2) The characteristic value of the seismic coefficient for the verification for the performance verification of steel cell type dolphins in variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics. The characteristic value of the seismic coefficient for verification of steel cell type dolphins may be calculated in accordance with gravity-type quaywall by applying **2.2.2(1) Seismic coefficient for verification used for verification of sliding and overturning of wall body and insufficient bearing capacity of foundation grounds in variable situations in respect of level 1 earthquake ground motion** when soil pressure is acting, or composite breakwaters by applying **Chapter 4, 3.1.4(12) Seismic Coefficient for Verification of Sliding, Overturning, and Bearing Capacity of Upright Sections for Level 1 Earthquake Ground Motion,** when soil pressure is not acting.

(3) For the foundations of cargo handling equipment and mooring posts, refer to **Chapter 2, 2.4 Pile Foundations,** and **9.15 Foundations for Cargo Handling Equipment**.

(4) In the case of a cylindrical cell type dolphin, the equivalent wall width can be calculated using equation **(5.6.1)**.

$$B = \sqrt{3}R \tag{5.6.1}$$

where

B : equivalent wall width (m)
R : radius of cylindrical cell (m)

[3] Caisson Type Dolphins

(1) For the performance verification of caisson type dolphins, refer to **2.2 Gravity-type Quaywalls**.

(2) The characteristic value of the seismic coefficient for verification of caisson type dolphins may apply steel cell type dolphins.

(3) Rotation of a caisson occurs when an eccentric external force acts on a dolphin. Examination of stability against rotation must be made even when the stability against sliding and overturning as well as against failure of the foundation ground due to insufficient bearing capacity are found to be satisfactory, because the confirmation of the stability with respect to these items does not guarantee that the caisson is safe against rotation. In this case, in calculating the resistance force, attention should be given to the friction force of the caisson bottom which is proportional to the bottom reaction force as described in **Chapter 2, 1.2 Caissons**.

(3) For the performance verification of structural members, refer to **[1] Pile Type Dolphins**. In addition, for the verification of caisson members, refer to **Chapter 2, 1.2 Caissons**.

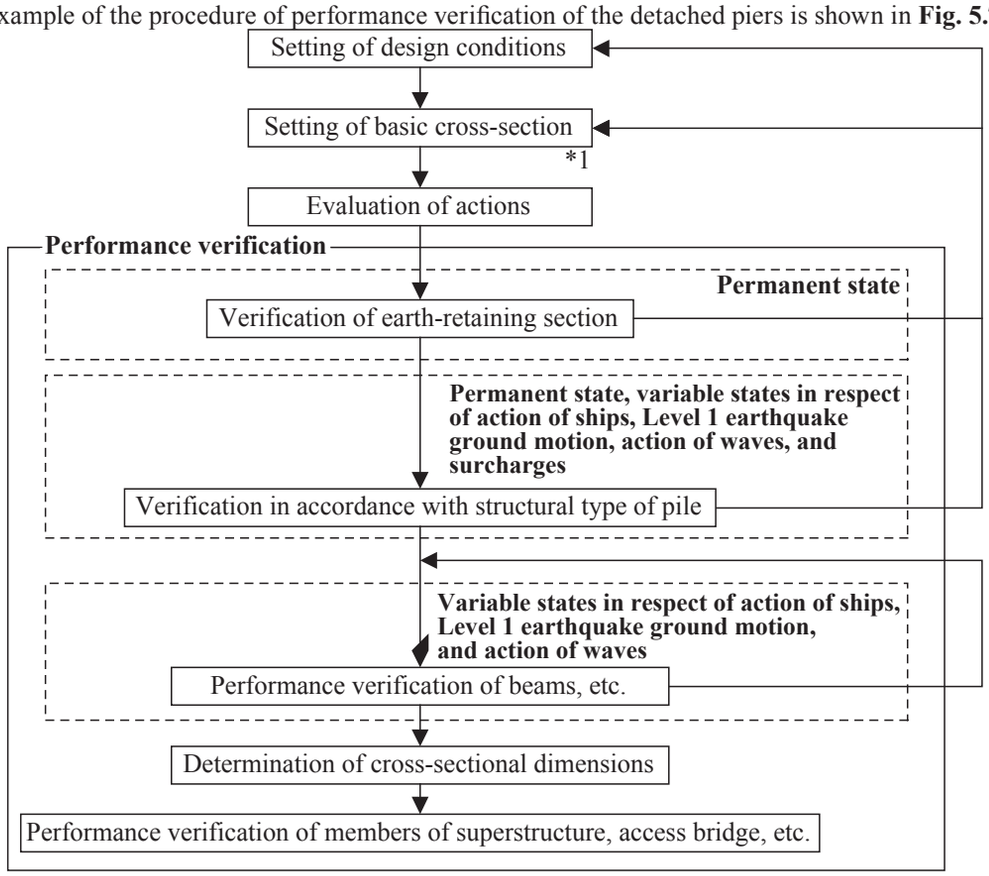
5.7 Detached Piers

5.7.1 Fundamentals of Performance Verification

(1) The performance verification of the detached piers may be carried out by appropriately selecting items from **5.2 Open-type Wharves on Vertical Piles**, **5.3 Open-type Wharves on Coupled Raking Piles**, **2.2 Gravity-type Quaywalls,** and **2.9 Cellular-bulkhead Quaywalls with Embedded Sections**, in accordance with the structure type. Also, the performance verification of the earth-retaining part may be carried out by appropriately selecting items from performance criteria of **2.2 Gravity-type Quaywalls, 2.3 Sheet Pile Quaywalls,** and **2.4 Cantilevered Sheet Pile Quaywalls**, and in addition refer to the following.

(2) The following may be applied to the performance verification of the detached piers comprising the detached pier and the earth-retaining section.

(3) An example of the procedure of performance verification of the detached piers is shown in **Fig. 5.7.1**.



*1: The evaluation of the effect of liquefaction is not indicated, therefore it is necessary to consider this separately.

Fig. 5.7.1 Example of Procedure of Performance Verification of Detached Piers

(4) An example of a cross-section of a detached pier is shown in **Fig. 5.7.2**.

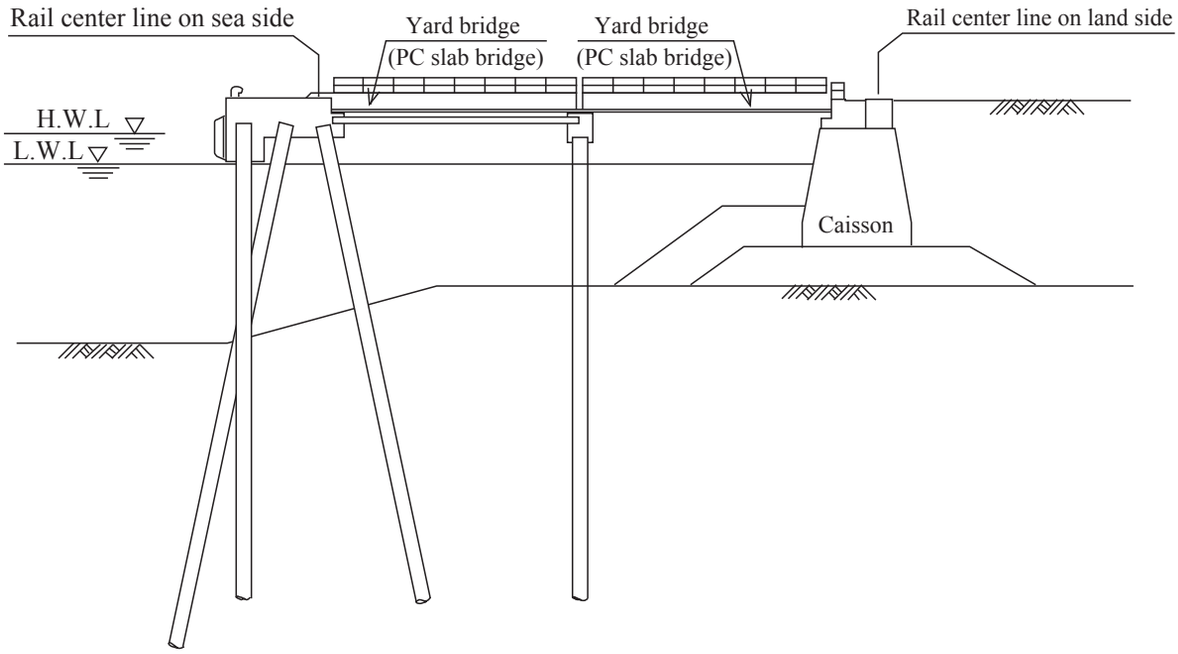


Fig. 5.7.2 Example of Cross-section of Detached Pier

(5) It is necessary to pay adequate attention to the deformation of the earth-retaining section due to the action of earthquakes.

(6) The performance verification of the detached pier shall be conducted so that it is stable against all the actions on the piles and girders. In addition, it is preferable for the detached pier to have a structure with due consideration for the type and dimensions of portal bridge crane, the traveling characteristics, and the settlement of rails after installation.

(7) Rail mounted cranes are installed on detached piers, therefore it is preferable that the structure shall have a small deformation.

5.7.2 Actions

(1) For the wheel loads of cargo handling equipment, refer to **Part II, Chapter 10, 3.2 Live Load**.

(2) For tractive forces of ship, refer to **Part II, Chapter 8, 2.4 Action due to Traction by Ships**.

(3) For the self weight of superstructures, and self weight of piles, refer to **Part II, Chapter 10, 2 Self Weight**, and **Chapter 10, 3 Surcharge**.

(4) For fender reactions, refer to **Part II, Chapter 8, 2.2 Action Caused by Ship Berthing, Part II, Chapter 8, 2.3 Action Caused by Ship Motions**.

(5) For wind loads acting on cargo handling equipment and superstructures, refer to **Part II, Chapter 2, 2.3 Wind Pressure**.

(6) For the ground motions acting on cargo handling equipment, superstructures, and piles, refer to **Part II, Chapter 4, 2 Seismic Action**.

(7) The characteristic value of the seismic coefficient for verification for the performance verification of the detached piers against the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics. For calculation of the seismic coefficient for verification of the detached piers, refer to **5.2.3(10) Ground Motion used in Performance Verification of Seismic-resistant**.

(8) For the performance verification of the detached piers, it is preferable to consider wave forces, uplift pressure, and wind loads acting on superstructures, when necessary.

(9) For the performance verification of the beams, braking forces on cargo handling equipment shall be considered as a horizontal force, but for piles shall be considered, as necessary.

(10) For the performance verification of the access bridges and the floor slabs, a live load of 5.0kN/m^2 may be assumed.

5.7.3 Performance Verification

Performance Verification of Girder

① The performance verification of girders shall be conducted so that they are safe against the vertical as well as horizontal forces and loads.

② Structural elements with sufficient strength against the designated vertical and horizontal forces shall be used for the girders of the detached pier, because the crane rails for a crane are directly installed on the girders. In the examination of vertical loads, the increase in the wheel loads due to the wind load or seismic force acting on the bridge crane shall be taken into account.

③ When both legs of the bridge crane are fixed ones, the horizontal load acting on each leg is determined by distributing the total horizontal load to each leg based on the proportion of the wheel load. When the bridge crane has a fixed leg and a suspended leg, the whole horizontal load shall be borne by the fixed leg for making the design on the safer side. At the same time, however, the horizontal force being one-half of the force acting on one fixed leg in the case of the both legs being fixed shall be borne by the suspended leg.

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6 Floating Piers

Ministerial Ordinance

Performance Requirements for Floating Piers

Article 30

1 The performance requirements for floating piers shall be as specified in the subsequent items in consideration of its structure type.

(1) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable the safe and smooth mooring of ships, embarkation and disembarkation of people, and handling of cargo.

(2) The damage due to self weight, variable waves, Level 1 earthquake ground motions, ship berthing and traction by ships, and/or other actions shall not impair the function of the floating pier nor affect its continued use.

2 In addition to the provisions of the preceding paragraph, the performance requirement of the floating piers in the place where there is a risk of having serious impact on human lives, property, and/or socioeconomic activity by the damage to the mooring buoys concerned shall be such that the structural stability of the floating pier is not seriously affected even in cases when the function of the mooring buoys concerned is impaired by tsunamis, accidental waves, and/or other actions.

Public Notice

Performance Criteria of Floating Piers

Article 56

1 The provisions of paragraph 1 of Article 48 (excluding item ii)) shall be applied to the performance criteria of floating piers.

2 In addition to the provisions in the preceding paragraph, the performance criteria of floating piers shall be as specified in the subsequent items in consideration of the structural type:

(1) The floating pier shall have the dimensions required for containment of their movements and tilting within the allowable range in consideration of the usage conditions.

(2) The risk of capsizing of the floating body under the variable action situation in which the dominant action is variable waves shall be equal to or less than the threshold level.

(3) The floating pier shall have the freeboard required for the dimensions of the design ships and the usage conditions.

(4) The following criteria shall be satisfied under the variable action situation in which the dominant actions are Level 1 earthquake ground motions, ship berthing and traction by ships, and imposed load:

(a) The risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(b) The risk of impairing the integrity of the members of the floating mooring facilities and losing the structural stability shall be equal to or less than the threshold level.

3 In addition to the provisions of the preceding two paragraphs, the performance criteria of floating piers for which there is a risk of serious impact on human lives, property, or socioeconomic activity by the damage to the facilities concerned shall be such that the degree of damage under the accidental action situation, in which the dominant actions are tsunamis or accidental waves, is equal to or less than the threshold level.

4 The provisions of Article 64 and Article 91 shall be applied with modification as necessary to the performance criteria of the access facilities of the floating body by taking account of the utilization conditions.

[Commentary]

(1) Performance Criteria of Floating Piers

① Common for floating piers

(a) In setting the cross-sectional dimensions for the performance verification of floating piers, it shall be

appropriately verified that the amount of motions of the floating body and the amount of tilting of the floating body are within the allowable range, in accordance with the envisaged conditions of use, as necessary.

(b) Freeboard (usability)

For the performance criteria of floating piers, the freeboard of the floating pier shall be appropriately set considering the dimensions of the design ships and the envisaged conditions of use to allow safe and efficient embarkation and disembarkation of passengers and safe and efficient handling of cargo.

(c) Structural stability and soundness of members (serviceability)

1) The setting of the performance criteria for the structural stability and soundness of members of floating piers and the design situations excluding accidental situations shall be in accordance with **Attached Table 50**. In the performance verification of floating piers, the performance criteria for variable situation in respect of variable waves, Level 1 earthquake ground motion, berthing and traction by ships, surcharges, for which performance verification is necessary, shall be appropriately set, in accordance with the structure type of the facility. The items within parentheses in the column of "Design situation" in **Attached Table 50** may be applies individually.

Attached Table 50 Setting of Performance Criteria for Structural Stability and Soundness of Structural Members of Floating Piers and Design Situations (excluding accidental situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
30	1	2	56	2	2	Serviceability	Variable	Variable waves	Self weight, wind, water pressure, water flow	Capsizing of floating body	Limit value for capsizing
					4a			(L1 earthquake ground motion)	(Self weight, wind, water pressure, water flow)	Soundness of members of floating body	–
					4b			(Berthing and traction by ships)	(Self weight, support reactions of connecting facilities, wind, water pressure, water flow, surcharges)	Soundness of members of mooring equipment	–
								(Surcharges)	(Self weight, wind, water pressure, water flow)	Structural stability of mooring equipment	–

2) Capsizing of floating bodies

For the performance verification against capsizing of floating bodies, the performance criteria for capsizing shall be appropriately set considering the conditions of use of the floating body and the natural conditions.

3) Soundness of the structural members of a floating body

For the performance verification of the structural members of a floating body, the performance criteria for their soundness shall be appropriately set considering the structural type and material of the members.

4) Soundness of the structural members of mooring equipment

i) For the performance verification of the structural members of mooring equipment, the performance criteria for their soundness shall be appropriately set considering the structural type and material of the members. The setting of the performance criteria for the soundness of structural members of the mooring equipment in the mooring system and the design conditions excluding accidental situations shall be in accordance with **Attached Table 51**. The items within parentheses in the column of "Design situation" in **Attached Table 51** may be applied individually.

Attached Table 51 Setting of Performance Criteria for soundness of Structural Members of Mooring Equipment with Mooring Ropes and Design Conditions (excluding accidental situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
30	1	2	56	2	4b	Serviceability	Variable	Variable waves	Self weight, wind, water pressure, water flow	Yielding of mooring ropes	Design yield stress
								(Berthing and traction by ships)	(Self weight, support reactions of connecting facilities, wind, water pressure, water flow, surcharges)	Stability of mooring anchors, etc.	Resistance force of mooring anchors (horizontal, vertical)

ii) Yielding of the mooring ropes
Verification of yielding of the mooring ropes is to verify that the risk that the design stresses in the mooring ropes will exceed the design yield stresses is equal to or less than the limit value.

iii) Stability of mooring anchors
Verification of the stability of mooring anchors is to verify that the risk that the tractive forces acting in the mooring anchors will exceed the resistance force is equal to or less than the limit value. Mooring anchors is a general term for equipment installed on the bottom of the sea to retain floating bodies, and this includes sinkers.

5) Structural stability of mooring equipment
For the performance verification of the structure of mooring equipment, the performance criteria of its stability shall be appropriately set in accordance with the structure type and materials, of the equipment.

② Floating piers against accidental incident (safety)

(a) The setting of the performance criteria of floating piers against accidental incident and design situations only limited to accidental situations shall be as shown in **Attached Table 52**.

Attached Table 52 Setting of Performance Criteria of Floating Piers against Accidental Incident and Design Situations only limited to Accidental Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
30	2	–	56	3	–	Safety	Accidental	Tsunamis	Self weight, wind, water pressure, water flow	Yielding of mooring ropes	Design yield stress
								accidental waves		Stability of mooring anchors	Resistance force of mooring anchors (horizontal, vertical)

(b) Required function of floating piers against accidental incident
The verification of mooring anchors against accidental situations where the dominating actions are tsunamis and accidental waves shall ensure that floating structures do not drift due to tsunamis and accidental waves resulting in a major effect on its vicinities.

③ Access facilities
The setting of the performance criteria of access facilities of floating piers shall apply the performance criteria for vehicle ramp, which is ancillary equipment of mooring facilities defined in the Standard Public Notice Article 64, and the performance criteria of fixed facilities for embarkation and disembarkation of passengers defined in Article 91, in accordance with the envisaged conditions of use of the floating pier. The access facilities of floating piers are those which function between a floating body and the land or between floating bodies as the passage of passengers or vehicles, such as access bridges, connecting bridges, and adjustment towers.

[Technical Note]

6.1 Fundamentals of Performance Verification

(1) The provisions in this chapter shall be applied to the floating piers with floating bodies (hereinafter referred to as "pontoons") that are moored by mooring chains, etc.

(2) The performance verification methods given in this chapter can be applied to the floating piers installed in places where the actions from waves, tidal currents, and winds are relatively weak.

(3) In setting the cross-sectional dimensions of the floating body of floating piers, it is necessary to appropriately verify that the amount of motion of the floating bodies and the amount of tilting of the floating bodies are controlled within the allowable range in accordance with the envisaged conditions of use.

(4) Freeboard
In the performance verification of the floating piers, it is necessary to appropriately set the freeboard of the floating pier to enable safe and smooth embarkation and disembarkation of passengers, and safe and smooth loading of cargo, considering the dimensions of the design ships and the envisaged conditions of use of the facility.

(5) **Fig. 6.1.1** and **Fig. 6.1.2** show the main components of a floating pier and the structure of a pontoon. A floating pier comprises pontoons, an access bridge that connects the pontoons with land, connecting bridges that interconnect pontoons, mooring chains that moor pontoons, mooring anchors, and other elements.

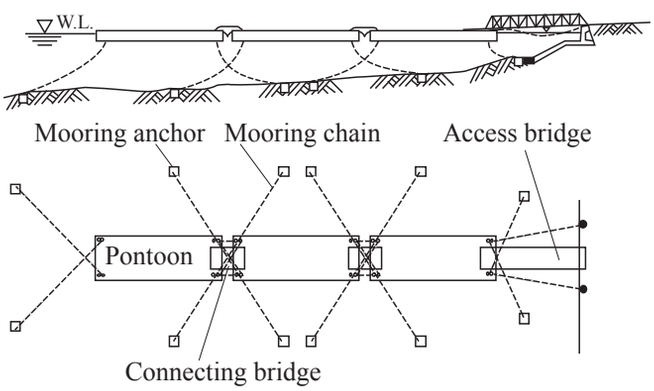

Fig. 6.1.1 Notation of Respective Parts of Floating Pier

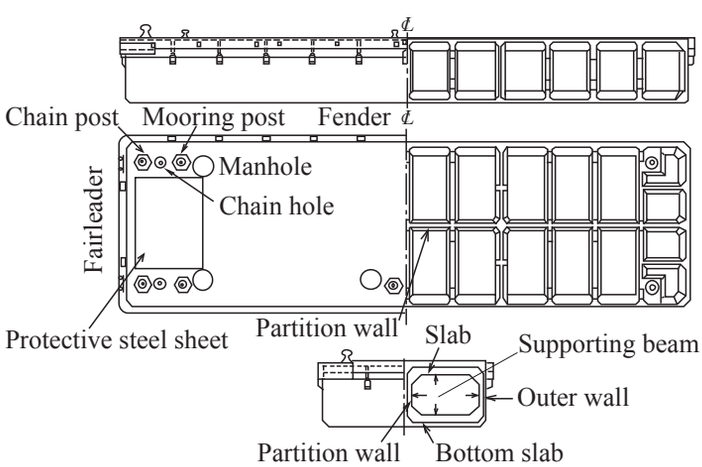

Fig. 6.1.2 Notation of Respective Parts of Pontoon

(6) When the site conditions are outside the coverage of this chapter, **The Technical Manual for Floating Body Structures**[1]" can be referred to. In addition, **Part II, Chapter 2, 4.7.4 Wave Force acting on Structures near the Water Surface**, **Part II**, **Chapter 2, 4.9 Actions on Floating Body and its Motions**, and **Chapter 4, 3.10 Floating Breakwaters** can be used as references as necessary.

(7) Normally, the floating piers are not used in locations where the waves or currents are large, but are frequently used

in locations where the wave height is 1m or less, and the current is 0.5m/s or less.

(8) An example of the procedure of performance verification of floating piers is shown in **Fig. 6.1.3**.

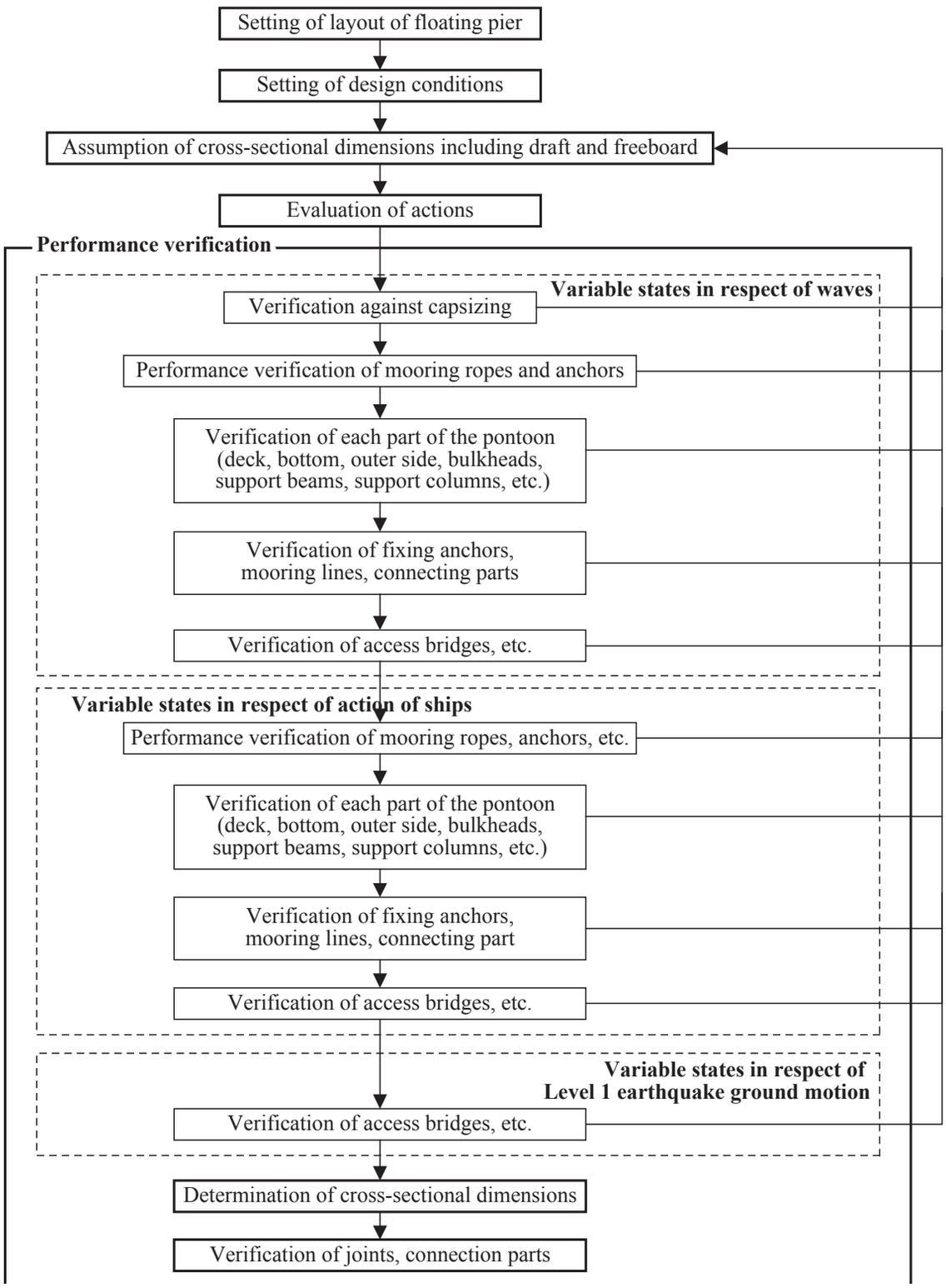

Fig. 6.1.3 Example of Sequence of Performance Verification of Floating Piers

6.2 Setting the Basic Cross-section

(1) A pontoon shall have a surface area and freeboard appropriate for its purpose of utilization. Dimensions of a pontoon shall be appropriate to make it stable against the external actions on it.

(2) The freeboard of a pontoon shall be set to an appropriate height to provide good conditions for cargo handling and passenger use when it is fully loaded and lightly loaded with cargo and passengers. Normally the height is set to about 1.0m. Generally the freeboard may be calculated using equation **(6.2.1)**.

$$h' = d - \frac{W_1}{\gamma_W A} \tag{6.2.1}$$

where,

h' : freeboard (m)
d : pontoon height (m)
W_1 : pontoon weight (kN)
γ_W : unit weight of seawater (kN/m^3)
A : horizontal cross-sectional area of pontoon (m^2)

(3) In the case of a reinforced concrete pontoon, it is preferable that the dimensions are determined considering the imperviousness of the concrete.

(4) Regarding the type of mooring, normally a chain type or a wire type are used for fairly deep water depths, and for shallow water depths, an intermediate wire type, an intermediate buoy type, or a dolphin-fender type are mainly used.[1)] It is preferable that the type of mooring be selected based on a comparison of the function and safety of the floating pier and the characteristics of the mooring facilities.

6.3 Actions

(1) The fender reaction force, wave force and current force need not be considered unless required to do so. However, when there is an anticipated risk that the pontoon may be subjected to wave actions, it is necessary to consider the following forces: the wave forces exerted upon the stationary pontoon that are assumed to be rigidly fixed in position and the fluid forces due to the oscillations of the pontoon [5] (refer to **Part II, Chapter 2, 4.9 Actions on Floating Body and its Motions**). In this case, the mooring force is to be calculated by considering the oscillations of the pontoon.

(2) A live load of 5.0 kN/m^2 for passengers is commonly used for floating piers, which are mainly used for passenger of ships.

(3) The fender reaction forces used in the performance verification of mooring chains may be calculated by reference to **Part II, Chapter 8, 2.2 Actions Caused by Ship Berthing**, and **Part II, Chapter 8, 2.3 Actions Caused by Ship Motions**. Also, for the tractive force of ship, refer to **Part II, Chapter 8, 2.4 Actions Caused by Traction by Ships**.

(4) The wave forces used in the performance verification of mooring chains may be calculated by an appropriate method by reference to **Part II, Chapter 2, 4.7.4 Wave Forces acting on Structures near Water Surface**, **Part II, Chapter 2, 4.9 Actions on Floating Body and its Motions**. The drag coefficient for cubes may be used. The area over which the drag force acts may be taken to be that below the still water surface. The above mentioned wave forces are those that act on a stationary pontoon, but if the natural period of the oscillations of the pontoon is close to the natural period of the waves, resonance may occur, causing a large force in the chain. This point should be carefully considered. In particular, for floating piers located in places where it is envisaged that swells and other long period waves penetrate, it is preferable that an motion analysis of the moored floating bodies be carried out using a numerical simulation method.[7]

6.4 Performance Verification

(1) Items to be considered in the Verification of the Stability of Floating Piers
Normally the following items are examined for the floating piers.
① pontoon stability
② stability of each part of the pontoon
③ stability of the mooring system (mooring chains, mooring anchors)
④ stability of access bridges and connecting bridges

(2) Performance Verification of the Stability of Pontoons

① Structural stability levels required for the pontoons should be secured appropriately in accordance with the conditions of use.

② In the examination of the stability of a pontoon, the following requirements should be satisfied:

(a) The pontoon must satisfy the stability condition of a floating body with the required freeboard against actions of the reaction force from the access bridge supporting point, full surcharge on the deck and even against the presence of some water inside the pontoon owing to leakage of the pontoon.

(b) Even when the full surcharge is placed on only one side of the deck divided by the longitudinal symmetrical axis of the pontoon and the reaction force from an access bridge supporting point acts on this side, if the bridge is attached there, the pontoon must satisfy the stability condition of a floating body and the inclination of the deck should be equal to or less than 1:10 with the smallest freeboard of 0 or more.

③ The height of the water accumulated inside the pontoon by leakage is usually assumed at 10% of the height of pontoon in the examination of pontoon stability. The freeboard to be maintained in this case is mostly about 0.5 m.

④ When being subjected to a uniformly distributed load, the pontoon may be regarded as stable, if equation **(6.4.1)** is satisfied.

$$\frac{I\gamma_W}{W} - \overline{\mathrm{CG}} > 0 \tag{6.4.1}$$

where

I : geometrical moment of inertia of the cross-sectional area at the still water level with respect to the longitudinal axis (m^4)
W : weight of pontoon and uniformly distributed load (kN)
γ_w : unit weight of seawater (kN/m3)
C : center of buoyancy of pontoon
G : center of gravity of pontoon

When the pontoon is partially filled with water by leakage, the pontoon may be regarded as stable when equation **(6.4.2)** is satisfied. The symbols W, I, C, and G of the equation refer to those at the state with water inside.

$$\frac{\gamma_W}{W}\left(I - \sum i\right) - \overline{\mathrm{CG}} > 0 \tag{6.4.2}$$

where

i : geometrical moment of inertia of the water surface inside each chamber with respect to its central axis parallel to the rotation axis of the pontoon (m^4)

When being subjected to an eccentric load, the pontoon may be regarded as stable if the value of $\tan\alpha$ obtained by solving equation **(6.4.3)** satisfies equation **(6.4.4)** (see **Fig. 6.4.1**).

$$\left(W_1 + P\right)\left[\frac{b^2 \tan\alpha}{12d\cos^2\alpha} - \left(\frac{b^2}{24d}\tan^2\alpha + c - \frac{d}{2}\right)\tan\alpha\right] - p\left[a + \left(h - c\right)\tan\alpha\right] = 0 \tag{6.4.3}$$

$$\left.\begin{aligned} &\tan\alpha < \frac{2\left(h-d\right)}{b} \\ &\tan\alpha < \frac{1}{10} \end{aligned}\right\} \tag{6.4.4}$$

where

W_1 : weight of pontoon (kN)
P : total force of eccentric load (kN)
b : width of pontoon (m)
h : height of pontoon (m)
d : draft of pontoon when P is applied to the center of pontoon (m)
c : height of the center of gravity of the pontoon measured from the bottom (m)
a : deviation of P from the center axis of pontoon (m)
α : inclination angle of pontoon (º)

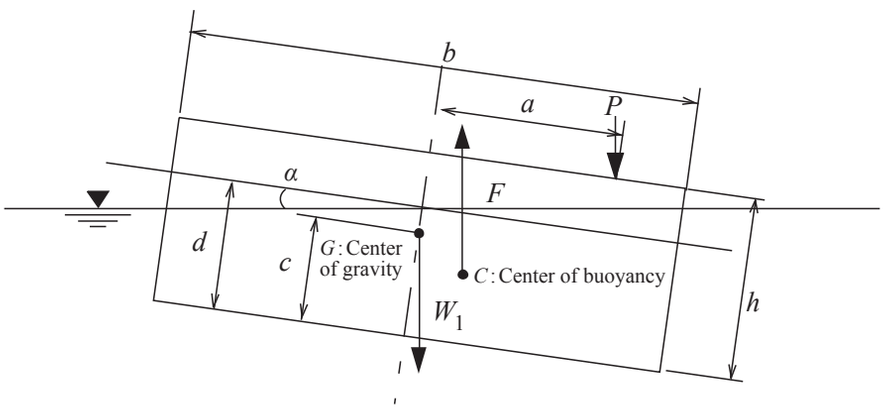

Fig. 6.4.1 Stability of Pontoon subjected to Eccentric Load

(3) Performance Verification of Each Part of a Pontoon

① Stresses generated in the structural parts of the pontoon shall be examined by using an appropriate method selected considering the use conditions of the pontoon, external actions on the respective parts, and their structural characteristics.

② A floor slab can normally be verified for performance as a two-way slab fixed on four sides with supporting beams and side walls against the actions that yield the largest stress out of the following combinations of actions:

(a) When only static load acts on a pontoon
(Static load) + (Self weight)

(b) When live load acts on a pontoon
(live load) + (Self weight)

(c) When the supporting point of an access bridge is set on a pontoon without adjustment tower
(Supporting point reaction force of an access bridge) + (Self weight)

③ A outer wall can normally be verified for performance as a two-way slab fixed on four sides with a floor slab, a bottom slab, and outer walls or supporting beams, against hydrostatic pressure acting when the pontoon submerges by 0.5 m above the deck.

④ A bottom slab can normally be verified for performance as a two-way slab fixed on four sides with outer walls or supporting beams, against hydrostatic pressure acting when the pontoon submerges by 0.5 m above the deck.

⑤ A partition wall can normally be verified for performance as a slab fixed on four sides when one compartment has become fully waterlogged and is exerting hydrostatic pressure .

⑥ The supporting beams of the floor slab, bottom slab and outer walls and the center support can normally be calculated as a rigid frame box under the condition that the maximum action is on the floor slab of the pontoon and the hydrostatic pressure for the draft of the pontoon being equal to its height is applied.

⑦ When the wave actions are to be considered, calculations of section forces are made using Muller's equation, [11] the method of Prestressed Concrete Barge, or Veritus Rule. When it is necessary to consider effects of the oscillations of the floating body, wave parameters, and water depth, the method with cross-sectional division by Ueda et al. [5), 12), 13)] may be used.

(4) Performance Verification of Mooring Chain

① The structure of mooring chains should be examined by using an appropriate method in such a way that the chains can hold securely a pontoon in position under the action of whichever force is the largest among the

fender reaction force generated during berthing, the tractive force of ship and the wave force, with the addition of the tidal current force to each of aforementioned forces.

② Normally, the length of a mooring chain is 5 times the water depth plus the tidal range. When a chain is stretched, it is necessary to pay attention to the following points:
(a) During high tide, the chain should not be over stretched causing an excessive tension force in it.
(b) During high tide there should be no interference with ship berthing.
(c) Sufficient anchor holding power should be ensured for mooring anchors during high tide.
(d) The amount of horizontal movement of the pontoon during low tide should be small.

③ It is said that the anchor holding power of steel mooring anchors is significantly reduced when the angle between the chain at the connection part and the horizontal surface is 3° or higher.

④ The maximum tension acting on each chain is ideally determined through dynamic analysis of the chain and the pontoon, but as this is very difficult, static analysis may be used as the second-best method. A chain can normally be verified for performance on the condition that only one chain is assumed to resist against all the external forces as shown in **Fig. 6.4.2**.

Assuming that the chain forms a catenary line, the maximum tension acting on the chain is given by equation **(6.4.5)**. In the equations below, the symbol γ represents the partial factor of its suffix and suffixes k and d stand for the characteristic value and the design value, respectively.

$$T_d = P_d \sec\theta_2 \tag{6.4.5}$$

The horizontal force acting on the mooring anchor is the same as the horizontal force acting on the pontoon, and the vertical force acting on the anchor is given by equation (**6.4.6**).

$$V_{ad} = P_d \tan\theta_1 \tag{6.4.6}$$

The vertical force acting on the joint between the chain and the pontoon is given by equation (**6.4.7**).

$$V_{bd} = P_d \tan\theta_2 \tag{6.4.7}$$

The angles θ_1 and θ_2 are calculated by equation **(6.4.8)** with an assumed chain length l and an assumed chain weight w per unit length of the chain.

$$\left.\begin{aligned} l &= \frac{P_d}{w}\left(\tan\theta_2 - \tan\theta_1\right) \\ h &= \frac{P_d}{w}\left(\sec\theta_2 - \sec\theta_1\right) \end{aligned}\right\} \tag{6.4.8}$$

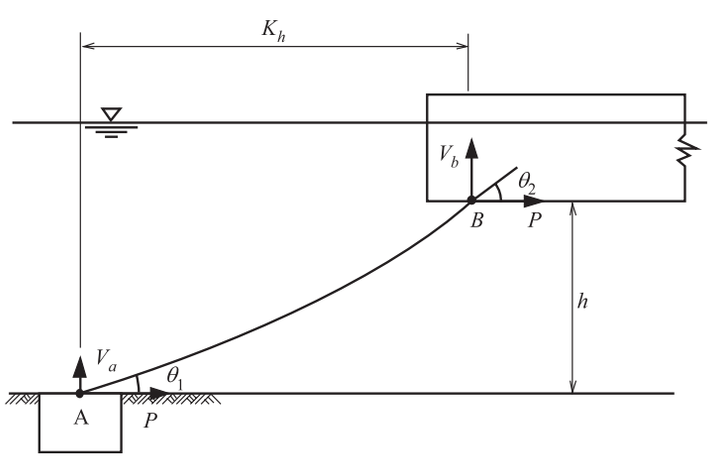

Fig. 6.4.2 Performance Verification of Mooring Chain

The horizontal distance between a mooring anchor and the pontoon when a horizontal force is acting on the pontoon is given by equation (**6.4.9**), and thus the amount of horizontal shift of the pontoon from its stationary position under no horizontal force can be evaluated.

$$K_h = \frac{P_d}{w}\left[\sinh^{-1}\left(\tan\theta_2\right) - \sinh^{-1}\left(\tan\theta_1\right)\right] \tag{6.4.9}$$

Because the catenary line of the chain of normal diameter can be approximately represented with a straight

line, it can be assumed in equations (**6.4.5**) to (**6.4.9**) that $\theta_2 = \theta_1 = \sin^{-1}(h/l)$ and $K_h = \sqrt{l^2 - h^2}$.

where

T : maximum tension acting on the chain (kN)
P : horizontal external force (kN)
V_a : vertical force acting on the mooring anchor (kN)
V_b : vertical force acting on the joint between the chain and pontoon (kN)
θ_1 : angle that the chain makes with the horizontal plane at the joint between the mooring anchor and chain (º)
θ_2 : angle that the chain makes with the horizontal plane at the joint between the mooring chain and pontoon (º)
l : length of the chain (m)
w : weight per unit length of the chain in water (kN/m)
h : water depth under the bottom of pontoon (m)
K_h : horizontal distance between the mooring anchor and the joint between the pontoon and chain (m)

The design values in the equations can be calculated by the following equation. The partial factor can be set at 1.0.

$$P_d = \gamma_P P_k$$

⑤ In the determination of the diameter of the chain, careful consideration should be given to the abrasion, corrosion, and biofouling of chain. In addition, appropriate maintenance work needs to be carried out on the chain, including periodical checks on the chain and its replacement, as necessary.

⑥ When determining the chain diameter with numerical ship motion simulation, the characteristics of displacement -restoration force relationship of the mooring system need to be determined using an appropriate method such as the catenary theory.[14)]

(5) Performance Verification of Mooring Anchor

① A mooring anchor shall be capable of providing the resistance forces required to keep the pontoon stable against the maximum tension acting on the mooring chain and shall have an appropriate stability.

② For the verification of the stability of mooring anchors, equation **(6.4.10)** may be used. In the following, the subscripts *k* and *d* indicate the characteristic values and design values respectively. Also, the structural analysis factor may be taken to be an appropriate value equal to or greater than 1.2.

$$\left.\begin{aligned} R_{hd} &\geq \gamma_a P_d \\ R_{vd} &\geq \gamma_a V_{ad} \end{aligned}\right\} \qquad (6.4.10)$$

where,

R_h : horizontal resistance force of mooring anchor (kN)
R_v : vertical resistance force of mooring anchor (kN)
P : horizontal force acting on mooring anchor (kN)
V_a : vertical force acting on mooring anchor (kN)
γ_a : structural analysis factor

In calculating the design values in the equation, the following equations may be used. Here, V_a, P, and θ_1 in the equations are as shown in **Fig. 6.4.2**. For the characteristic value of the maximum tension force acting in the mooring chain P_k, the value obtained in **(4) Performance Verification of Mooring Chains**, may be used. The value 1.0 may be used for the partial factors.

$$V_{ad} = \gamma_P P_k \tan\theta_1, P_d = \gamma_P P_k, R_{hd} = \gamma_{R_h} R_{hk}, R_{vd} = \gamma_{R_v} R_{vk}$$

③ Normally the following forces are considered as the resistance forces of a mooring anchor, but it is preferable that in-situ stability tests be made for a mooring anchor:

(a) In the case of concrete block:

1) For clay ground:
Horizontal resistance force Rh : Cohesion of the surfaces of bottom and sides, difference between the passive and active earth pressures
Vertical resistance force Rv : Weight in water, effective overburden weight in water

2) For sand ground:

Horizontal resistance force Rh : Bottom friction force, difference between the passive and active earth pressures
Vertical resistance force Rv : Weight in water, effective overburden weight in water
The vertical force employed in the calculation of the bottom friction force is the difference between the weight of the block in water and the vertical component of the chain tension acting on the block.

(b) In the case of steel mooring anchor:
Horizontal resistance force Rh : Holding power
Vertical resistance force Rv : Weight in water

The holding power of a steel mooring anchor is calculated by equation (**6.4.11**).

$$\begin{aligned} &\text{on soft mud:} && T_A = 17\ W_{Ad}^{2/3} \\ &\text{on hard mud:} && T_A = 10\ W_{Ad}^{2/3} \\ &\text{on sand :} && T_A = 3\ W_{Ad} \\ &\text{on flat rock:} && T_A = 0.4\ W_{Ad} \end{aligned} \tag{6.4.11}$$

where
T_A : holding power of the mooring anchor (kN)
W_A : weight of the mooring anchor in water (kN)

The design values in the equations can be calculated from the following equation. Also, the partial factor may be taken to be 1.0.

$$W_{Ad} = \gamma_{W_A} W_{Ak}$$

④ When a rectangular solid anchor block is deeply embedded in a cohesive soil, Hansen obtained equation **(6.4.12)** for the horizontal resistance force by assuming the slip surface around the block.

$$P = 11.4ch \tag{6.4.12}$$

Also, Mackenzie experimentally obtained equation **(6.4.13)** for blocks embedded to a depth of 12 times or more the height of the block.[15)]

$$P = 8.5ch \tag{6.4.13}$$

where,
P : resistance force of the block per unit width (kN/m)
c : cohesion of the cohesive soil (kN/m^2)
h : block height (m)

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7 Shallow Draft Wharves

Ministerial Ordinance

Performance Requirements for Shallow Draft Wharves

Article 31
The provisions of Article 26 or Article 29 shall be applied correspondingly to the performance requirements for shallow draft wharves.

Public Notice

Performance Criteria of Shallow Draft Wharves

Article 57
The provisions of Article 48 through Article 52, or Article 55 shall be applied with modification as necessary to the performance criteria of shallow draft wharves in consideration of the structural type.

[Technical Note]

(1) Performance Requirements

① Common for shallow draft wharves
The performance requirements of shallow draft wharves shall ensure the necessary usability to enable safe and efficient mooring of ships, safe and efficient boarding and embarkation and disembarkation of passengers, and safe and efficient loading and unloading of cargo. In addition, it is necessary that the performance verification shall ensure serviceability in respect of the necessary design situations, from among the permanent situations in respect of self weight and earth pressure, and variable situations in respect of variable waves, Level 1 earthquake ground motion, berthing and traction by ships and surcharges in accordance with the structural type of the facilities.