Chapter 5 Mooring Facilities

1 General

[Ministerial Ordinance] (General Provisions)

Article 25

Mooring facilities shall be installed in appropriate locations in light of geotechnical characteristics, meteorological characteristics, sea states, and other environmental conditions, as well as ship navigation and other usage conditions of the water area around the facilities so as to secure the safe and smooth usage by ships.

[Ministerial Ordinance] (Necessary Items concerning Mooring Facilities)

Article 34

The items necessary for the performance requirements of mooring facilities as specified in this Chapter by the Minister of Land, Infrastructure, Transport and Tourism and other requirements shall be provided by the Public Notice.

[Public Notice] (Mooring Facilities)

Article 47

The items to be specified by the Public Notice under Article 34 of the Ministerial Ordinance concerning the performance requirements of mooring facilities shall be as provided in the following Article to Article 74.

1.1 Purpose of Mooring Facilities

The purpose of installing mooring facilities is to ensure the safety and smoothness of the mooring and landing operation of ships, the embarkation and disembarkation of passengers, and the loading and unloading of cargo.

1.2 General

(1) Mooring facilities include quaywalls, piers, lighter's wharves, floating piers, docks, mooring buoys, mooring piles, dolphins, detached piers, and air-cushion-craft landing facilities. Among quaywalls, piers, and lighter's wharves, facilities that are particularly important from the viewpoint of earthquake preparedness and require the strengthening of earthquake-resistant performance are considered high earthquake-resistant facilities are classified as high earthquake-resistant facilities (specially designated emergency supply transport), high earthquake-resistance facilities (specially designated trunk line cargo transport), and high earthquake-resistance facilities (standard emergency supply transport), which correspond to the functions required in the objective facilities after the action of ground motion.

(2) The structural types of mooring facilities shall be determined by taking into consideration natural conditions, use conditions, construction conditions, and economic efficiency. The structural types of mooring facilities are classified into gravity-type quaywalls, sheet pile quaywalls, cantilevered sheet pile quaywalls, double sheet pile quaywalls, quaywalls with relieving platforms, embedded-type cellular-bulkhead quaywalls, placement-type cellular-bulkhead quaywalls, open-type wharves on vertical piles, open-type wharves on coupled raking piles, and jacket piers.

1.3 Dimensions and Layout of Mooring Facilities

(1) The dimensions of mooring facilities are preferably determined on the basis of actual circumstances, including the number and type of cargoes and passengers utilizing the port; packing type; marine and land transportation; and other relevant factors, with due consideration given to future trends in cargo and passenger volumes, increase in vessel size, changes in transportation systems, etc.

(2) The layout of mooring facilities is preferably determined to facilitate the berthing and unberthing of ships with due consideration given to oceanographic, topographic, and subsoil conditions and to identify the relationships of land transport network systems and land use in the hinterland. In particular, the locations of the following mooring facilities shall be determined in accordance with the provisions in respective items.

① Mooring facilities for passenger ships: They should be isolated from areas where hazardous cargoes are handled and should be provided with sufficient areas of land for passenger-related facilities such as waiting rooms and parking lots in the vicinity of the facilities.

② Mooring facilities used by vessels loaded with hazardous cargoes: They should be isolated from facilities for which a healthy living environment needs to be preserved (e.g., houses, schools, and hospitals), and kept at a predetermined safe distance from other mooring facilities and sailing vessels. In addition, measures should be in place to quickly respond to incidents such as hazardous material spills.

③ Mooring facilities that are accommodating ships or cargo-handling machines that are generating considerable noise: They should be isolated from facilities for which a healthy living environment needs to be preserved (e.g., houses, schools, and hospitals).

④ Mooring facilities for ships loaded with cargoes that may generate significant dust or offensive odors while being handled: They should be isolated from facilities for which a healthy living environment needs to be preserved (e.g., houses, schools, and hospitals).

⑤ Offshore mooring facilities: They should be in a location that does not hinder the navigation or anchorage of vessels.

⑥ High earthquake-resistant facilities and large-scale mooring facilities: They should be located in areas with good ground conditions that do not amplify earthquake motions as much as possible to avoid huge investments in possible ground improvement depending on ground conditions and earthquake motion amplification characteristics. Refer to [Part II], Chapter 1, 4.6 Application of Microtremor Observation Results to Port Planning for the explanation of the method for simply estimating the distribution of earthquake motion amplification characteristics inside ports by utilizing the microtremor observation results and by applying the estimated distribution to port planning.

⑦ Facilities and high earthquake-resistant facilities that may have serious effects on human lives, property, and socioeconomic activities when damaged by disasters: When located close to inland active faults as hypocenters, these facilities should be installed with their face line directions arranged perpendicular to earthquake source faults. Regarding the occurrence of earthquakes with inland active faults as hypocenters, the areas close to the hypocenters may experience particularly strong ground motions in the direction perpendicular to the faults. Therefore, facilities that are installed with their face line directions arranged perpendicular to possible source faults are structurally advantageous for alleviating the actions of the earthquake ground motions caused by the source faults.

1.4 Selection of the Structural Types of Mooring Facilities

(1) The structural types of mooring facilities are preferably selected on the basis of the comparative examination of the following items and by taking into consideration the characteristics and economic efficiency of the respective structural types:

① **Natural conditions**

Natural conditions relevant to mooring facilities mainly include the mechanical property of soil, earthquakes, waves, tidal levels, and currents. In many cases, the mechanical property of soil becomes a decisive factor in selecting the structural types of mooring facilities because major Japanese ports are located within the vicinities of river mouths or inner bays with seafloors that are generally formed by the development of alluvium and mostly comprise soft soil. In cases of constructing mooring facilities on soft ground, structures that are capable of reducing burdens on the ground are selected, and ground improvement may be implemented if needed.

② **Use conditions**

Use conditions indicate the constraints in the use of mooring facilities after their construction imposed by the types of berthing ships, the types and quantities of cargoes to be handled, and the cargo-handling methods. Use conditions are the decisive factors of the fender reaction force, ship traction force, surcharges, and allowable deformation amounts to be set for the performance verification of mooring facilities.

③ **Construction conditions**

Considering that mooring facilities are often located offshore, they are subjected to various constraints in offshore work, i.e., the working hours involved in constructing mooring facilities offshore are largely affected not only by weather and temperature but also by waves, tides, and currents. Given that offshore work may cause serious problems with turbid seawater, it is necessary to take measures to prevent the surrounding environment from taking in turbid seawater. Furthermore, it is preferable to sufficiently examine the construction methods for mooring facilities by taking into consideration the difficulty in achieving and confirming the accuracy of undersea construction work. Onshore fabrication facilities such as caisson and block fabrication yards may become constraining factors in selecting the structural types of mooring facilities.

(2) The outlines and characteristics of the structural types of mooring facilities are as follows:

① **Gravity-type quaywalls**

(a) Outline:

Gravity-type quaywalls resist horizontal actions such as earth and water pressure by the weight of wall bodies. Fig. 1.4.1 shows an example of a cross section of a gravity-type quaywall.

(b) Characteristics:

1) Wall bodies are relatively firm and durable because they are made of concrete or similar materials.

2) The use of precast concrete members facilitates construction work and prevents reworking and accidents during construction.

3) The horizontal actions such as earth and water pressure acting on wall bodies increase when the water depths involved in mooring facility installations become deeper. This situation requires the weights of wall bodies to be drastically increased. Therefore, ground improvements may be required when constructing gravity-type quaywalls on soft ground where a large bearing capacity cannot be expected.

4) To prevent the ground behind quaywalls from being washed out, sand invasion prevention plates are installed between caissons and backfill stones, or sand invasion prevention sheets are laid on backfill stones. It is necessary to consider the damage that will be incurred by sand invasion prevention sheets because this type of damage may cause the sagging of aprons and reclaimed land behind quaywalls.

5) The actions on wall bodies due to earthquake ground motions are proportional to the weight of the wall bodies. Therefore, when wall bodies are designed to resist strong earthquakes by increasing quaywall widths, the wall bodies are also subjected to increased actions, i.e., it is difficult to prevent wall bodies from experiencing deformation during strong earthquakes. By contrast, gravity-type quaywalls, particularly caisson-type quaywalls, do not suddenly lose their stability in many cases even when they undergo deformation. Therefore, gravity-type quaywalls are advantageous in terms of serviceability after earthquakes. As described, the structures of gravity-type quaywalls have both advantages and disadvantages with respect to earthquakes.

6) Gravity-type quaywalls require large-scale onshore facilities such as caisson and block fabrication yards and special workboats such as crane barges and tug boats. Therefore, the construction of small-scale gravity-type quaywalls that do not require long construction periods is uneconomical if construction areas do not have onshore fabrication facilities and workboats.

7) In cases wherein existing seafloor surfaces are shallower than the design depths, gravity-type quaywalls are not advantageous because of the possible increase in dredging volume.

8) Gravity-type quaywalls should be carefully designed on soft cohesive ground because of the possible consolidation of cohesive layers, which cause gradual settlement over a long period of time.

9) Gravity-type quaywalls are classified into the following categories on the basis of the forms of wall bodies and construction methods:

i) Caisson-type quaywalls;

ii) L-shaped block quaywalls;

iii) Cellular-bulkhead-type quaywalls;

iv) Concrete block quaywalls;

v) Cast-in-place concrete quaywalls;

vi) Upright wave-absorbing-type quaywalls.

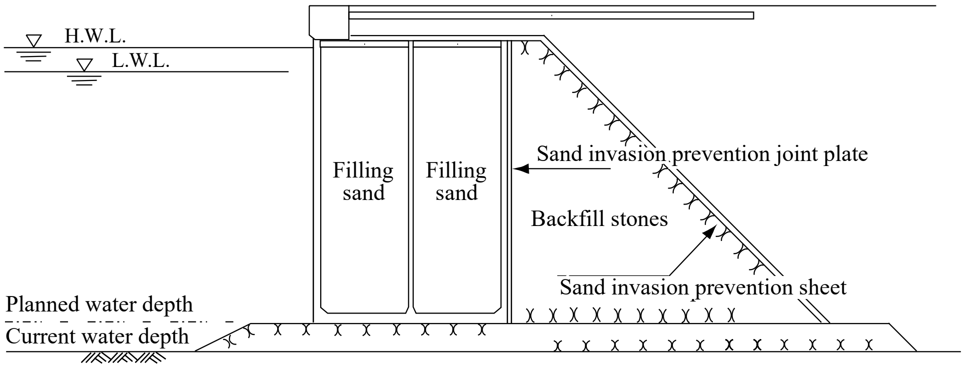


Fig. 1.4.1 Cross Section of a Gravity-Type Quaywall

② **Sheet pile quaywalls**

(a) Outline:

1) Sheet pile quaywalls are quaywalls with sheet piles driven as earth-retaining walls. Sheet piles can be made of steel, reinforced concrete, pre-stressed concrete, or wood. Among these materials, steel has been most frequently used for sheet piles. Given the large yield stress and availability of models with large section moduli, steel sheet piles can be used for quaywalls with large depths.

2) The cross-sectional shapes of normally used steel piles are classified into three types: hat-type, U-shaped, and steel pipes with joints. When using sheet piles with U-shaped cross sections, it is necessary to design their joints in a manner that prevents them from sliding because the configuration of the cross sections of U-shaped sheet piles with joints arranged along the neutral axes of sheet pile walls is subjected to reductions in section moduli obtained from sheet pile walls as integrated structures when sliding occurs between joints.

3) Steel pipe sheet piles fabricated by connecting large diameter steel pipes with joints can enlarge section moduli without large increments in the weight of steel per unit width, thereby enabling sheet piles with larger section moduli than steel sheet piles to be easily fabricated.

4) Reinforced concrete and pre-stressed concrete sheet piles are not often used for large-scale quaywalls because it is difficult to drive sheet piles with increased thicknesses owing to large section moduli. Even if they can be driven, there may be cases wherein sheet piles are damaged while being driven into stiff ground. Therefore, when driving reinforced concrete or pre-stressed concrete sheet piles into stiff ground, possible damage should be inspected by pullout tests or the sheet piles should be driven using water jetting to protect them from being damaged. Furthermore, sheet piles need to have joint plates to prevent the possible washing out of backfill soil through the joints.

(b) Characteristics:

1) Sheet piles quaywalls can be constructed using relatively simple machines at low costs.

2) In many cases, sheet pile quaywalls do not require undersea construction as foundation; therefore, such quaywalls can be rapidly constructed.

3) In cases wherein the existing seafloor is deep, sheet pile walls are placed into a state that is vulnerable to waves until backfill or anchorage work is constructed.

4) Sheet pile quaywalls are classified into the following categories on the basis of the method used to resist the earth and water pressure acting on the quaywalls:

i) Sheet pile quaywalls (in which sheet pile walls are connected to anchorage work via tie rods. Refer to **Fig. 1.4.2**);

ii) Cantilevered sheet pile quaywalls (refer to **Fig. 1.4.3**);

iii) Sheet pile quaywalls with raking pile anchorages (refer to **Fig. 1.4.4**);

iv) Sheet pile quaywalls anchored by forward batter piles (refer to **Fig. 1.4.5**);

v) Double sheet pile quaywalls (refer to **Fig. 1.4.6**).

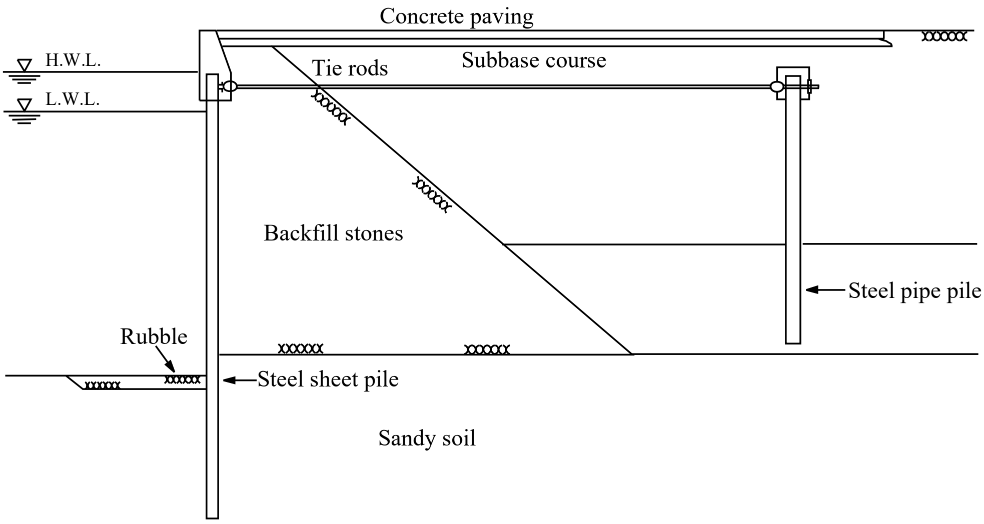


Fig. 1.4.2 Cross Section of a Sheet Pile Quaywall

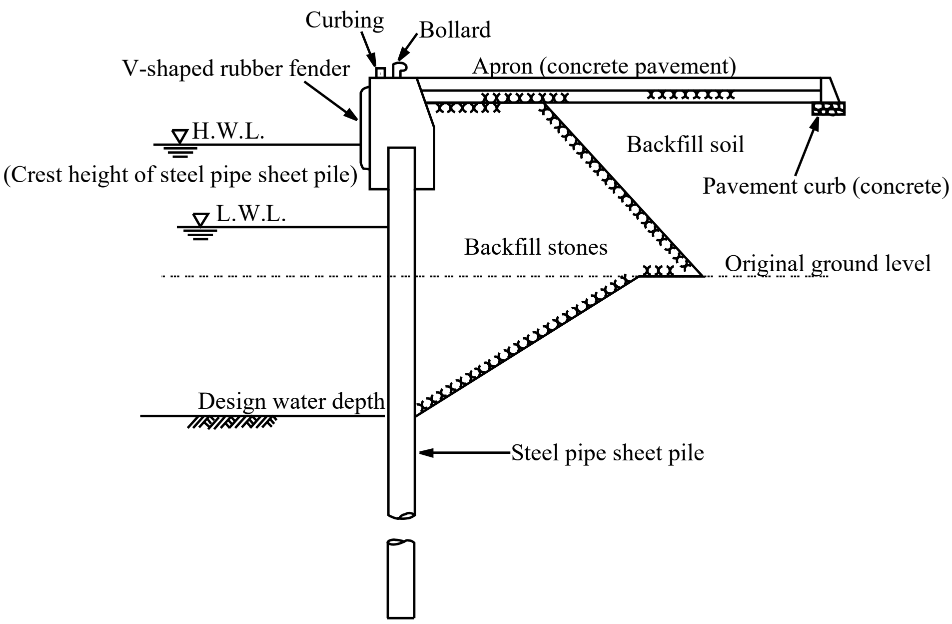


Fig. 1.4.3 Cross Section of a Cantilevered Sheet Pile Quaywall

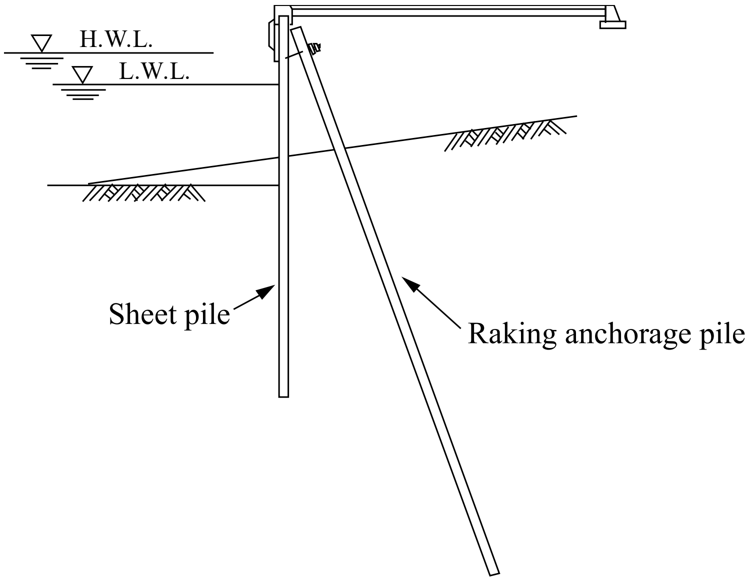


Fig. 1.4.4 Cross Section of a Sheet Pile Quaywall with Raking Pile Anchorage

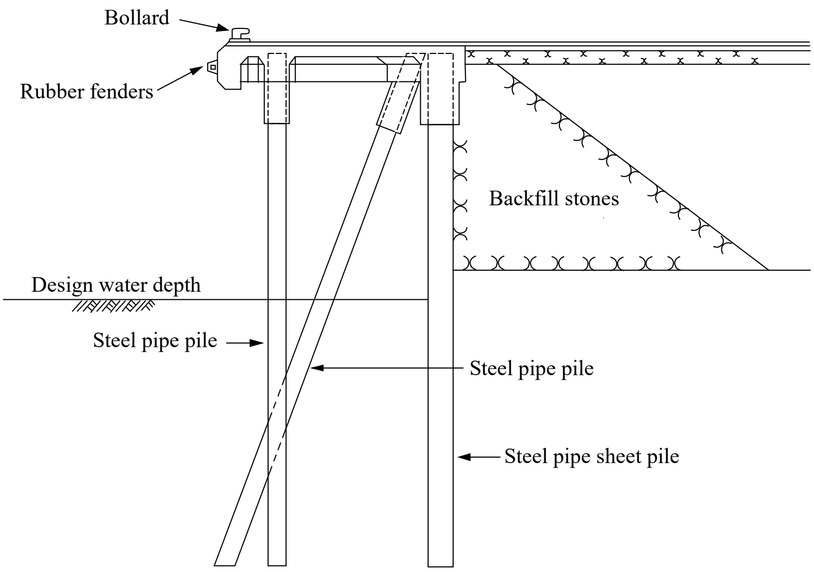


Fig. 1.4.5 Cross Section of a Sheet Pile Quaywall Anchored by Forward Batter Piles

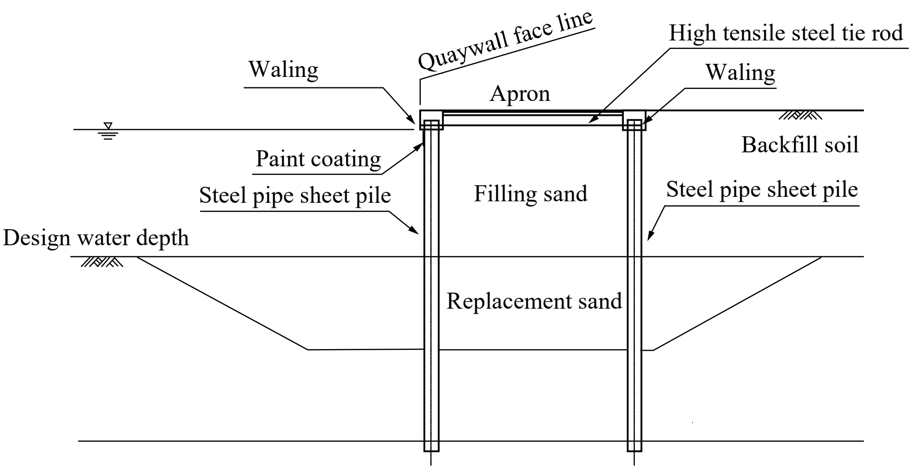


Fig. 1.4.6 Cross Section of a Double Sheet Pile Quaywall

③ Quaywalls with relieving platforms

(a) Outline:

Quaywalls with relieving platforms have structures that alleviate earth pressure acting on sheet piles with relieving platforms above sheet pile walls and resist horizontal loads with the active earth pressure at the embedded sections of sheet piles and with the horizontal resistance of relieving platform piles driven behind the sheet pile walls. Some types of quaywalls with relieving platforms have sheet pile walls at the rear face of relieving platforms. **Fig. 1.4.7** shows an example of a cross section of a quaywall with a relieving platform.

(b) Characteristics:

1) Quaywalls with relieving platforms enable piles that resist surcharges and can be constructed on soft ground, to which the structures of sheet pile quaywalls cannot be applied because sufficient passive earth pressure cannot be obtained as resistance at the embedded sections of sheet pile walls.
2) The sheet pile walls for quaywalls with relieving platforms can be smaller than those for sheet pile quaywalls.
3) The construction procedures of quaywalls with relieving platforms are complex compared with those of sheet pile quaywalls.
4) Quaywalls with relieving platforms require long construction periods.

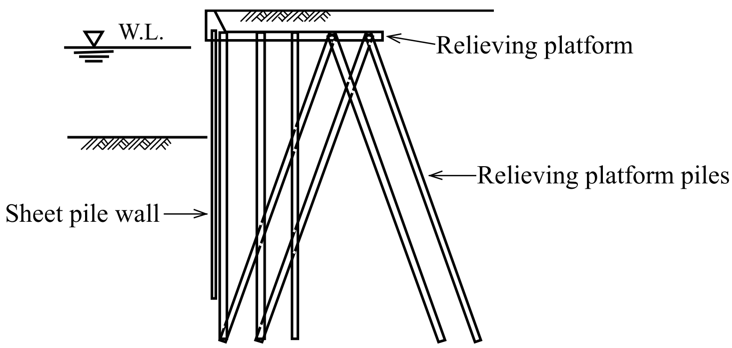


Fig. 1.4.7 Cross Section of a Quaywall with a Relieving Platform (L-Shaped Platform)

④ Cellular-bulkhead type quaywalls

(a) Outline:

Cellular-bulkhead type quaywalls have structures that comprise cylindrical cellular-bulkheads that are made of steel plates or steel sheet piles and are installed on the seafloor and infill materials placed inside the cellular bulkhead to ensure quaywall stability. The structures of cellular-bulkhead type quaywalls are largely classified into placement-type cellular bulkhead without embedment and embedded-type cellular bulkhead with embedment. Placement-type cellular-bulkhead quaywalls have structures that resist an external force with the weight and shear resistance of infill. Embedded-type cellular-bulkhead quaywalls can utilize the resistance at the embedded sections to resist external force, in addition to the weight and shear resistance of infill. Cellular bulkheads are normally made of steel, particularly steel plates for placement-type quaywalls and steel plates or linear sheet piles for embedded-type quaywalls. **Fig. 1.4.8** shows an example of a cross section of a cellular-bulkhead quaywall (embedded type).

(b) Characteristics:

1) The structures of cellular-bulkhead quaywalls are relatively simple; therefore, these quaywalls are suitable for rapid construction. The structures are also economical when ground conditions are suitable.
2) Embedded-type cellular-bulkhead quaywalls can eliminate the necessity of constructing foundation mounds; therefore, these quaywalls can reduce the workload associated with undersea construction.

3) Sufficient embedded lengths are required when constructing cellular-bulkhead quaywalls on the ground with small bearing capacity.

4) Cellular-bulkhead quaywalls are classified into the following categories:
   - i) Placement-type cellular-bulkhead quaywalls;
   - ii) Embedded-type cellular-bulkhead quaywalls.

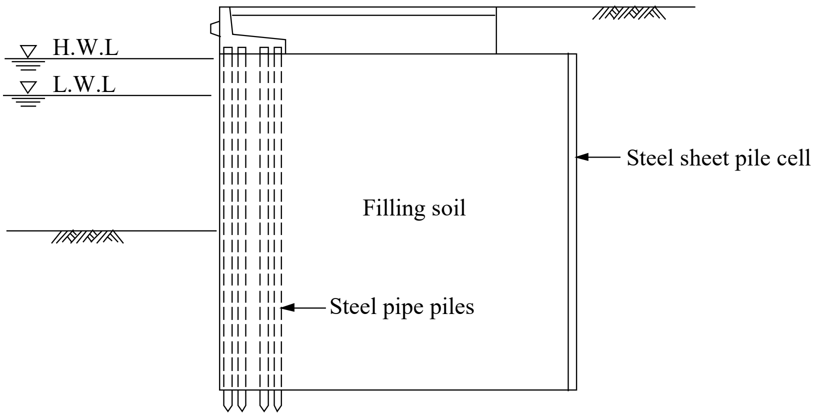


Fig. 1.4.8 Cross Section of a Steel Sheet Pile Cellular-Bulkhead Quaywall

⑤ Open-type piled wharves and piled wharves

(a) Outline:

Open-type piled wharves generally comprise earth-retaining revetments and open-type wharves constructed in front of revetments. Earth-retaining revetments have earth-retaining walls with earth slopes behind them. Piled wharves comprise columns such as piles and slabs installed on the columns. **Fig. 1.4.9** shows an example of an open-type wharf on vertical piles.

(b) Characteristics (open-type wharves):

1) Open-type wharves are suitable for soft ground where quaywalls with vertical front faces may cause ground failures.

2) Plural piles are arranged in the direction perpendicular to face lines (perpendicular to berths); therefore, the yield of one pile does not immediately lead to the failures of entire structures.

3) Open-type wharves can utilize existing facilities when constructing new wharves in front of existing revetments or extending the depths of existing shallow berths.

4) In cases of open-type wharves constructed in the areas with high waves, they may be at risk of failures with slabs and access bridges subjected to upward wave force.

5) The structures of open-type wharves comprising the combination of earth-retaining and wharf sections require complex construction procedures.

(c) Characteristics (piled wharves):

1) The structures of piled wharves where superstructures are supported by piles are light in weight compared with other structural types; therefore, these structures are suitable for soft ground to which gravity-type or sheet pile quaywalls cannot be applied.

2) Considering that piled wharves barely disturb the flow of seawater, they can be constructed even in areas with large influences of littoral drift and currents without disturbing the balance of natural conditions.

3) Piled wharves do not require soil for reclamation.

4) Piled wharves may interfere with ship berthing because they do not disturb the flow of seawater.

(d) Characteristics (common to both open-type piled wharves and piled wharves):

1) Both types have a disadvantage for large concentrated loads.

2) Both types are relatively vulnerable to horizontal force.

3) Open-type piled wharves and piled wharves are classified into the following categories depending on the structures of studs supporting slabs:

   i) Piled wharf;

   ii) Cylindrical or square-tube-type wharf;

   iii) Bridge-pier-type wharf

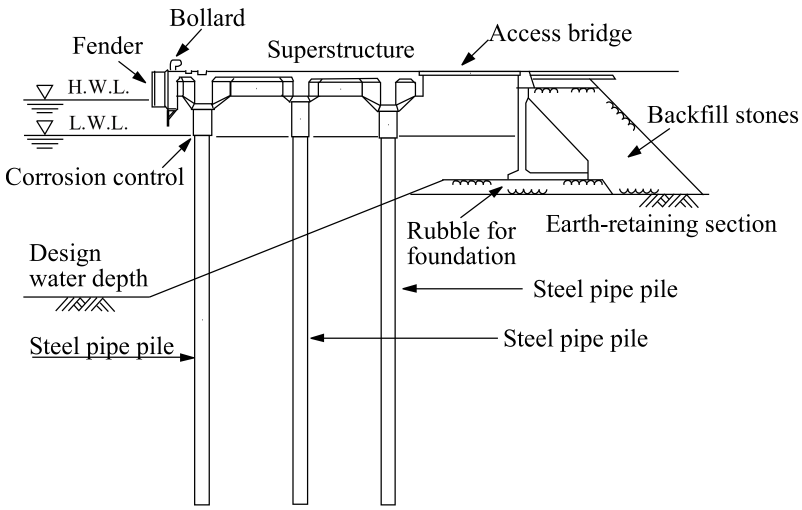


Fig. 1.4.9 Cross-section of an Open Type Wharf on Vertical Piles

⑥ **Detached piers**

(a) Outline:

Detached piers are mooring facilities that are used for handling bulk cargoes, such as coal and iron ore, in large quantities and have rail-mounted portal bridge cranes or loaders with their foundations installed at appropriate depths. Generally, detached piers do not require floor structures and comprise column sections and beam sections installed between column sections. Detached piers are categorized as special open-type piled wharves with no slabs and access bridges; therefore, these piers have characteristics that are similar to open-type piled wharves. **Fig. 1.4.10** shows an example of a cross section of a detached pier.

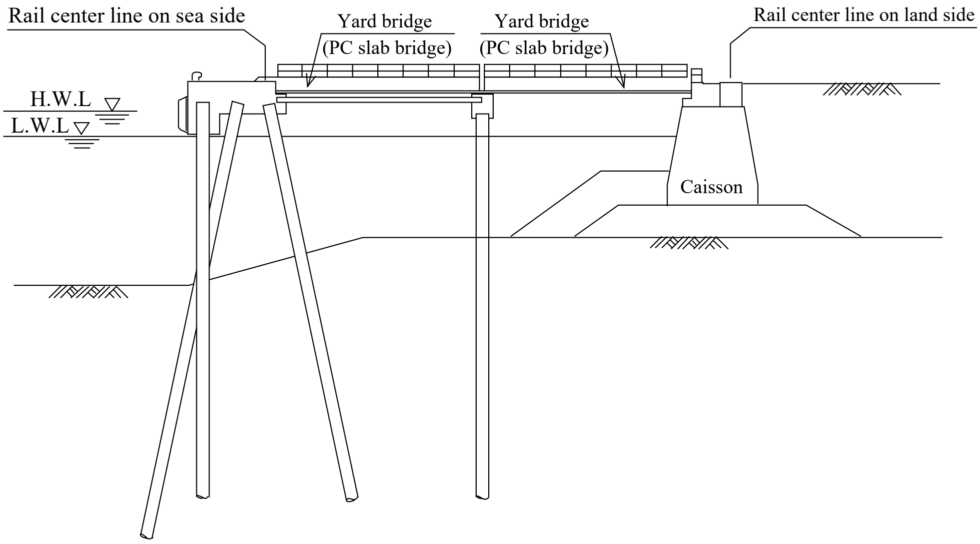


Fig. 1.4.10 Cross Section of Detached Pier

⑦ **Floating piers**

(a) Outline:

Floating piers are mooring facilities that comprise pontoons connected to land and to other pontoons by access bridges. **Fig. 1.4.11** shows an example of a cross sections and plan of a floating pier.

(b) Characteristics:

1) Pontoons maintain fixed distances between the top surfaces of floating piers and water surfaces in a manner that moves up and down in accordance with water levels. Therefore, they are suitable for mooring small vessels and ferry boats mainly for passenger use.

2) Floating piers allow seawater to flow more freely than piled wharves; therefore, these piers have little influence on littoral drift.

3) Floating piers can be easily constructed and relocated.

4) Floating piers are suitable for relatively soft ground.

5) Floating piers have small cargo-handling capacities because of difficulties in mounting cargo-handling equipment on them.

6) Floating piers are unsuitable for areas subjected to large influences of waves and currents.

7) Considering that steel materials are generally used for mooring wires and anchors, it is necessary to maintain respective sections that are made of steel materials with particular focus not only on corrosion but also mechanical wear.

8) Depending on the materials used for fabrication, pontoons are classified into reinforced concrete, steel, prestressed concrete, wood, and FRP types.

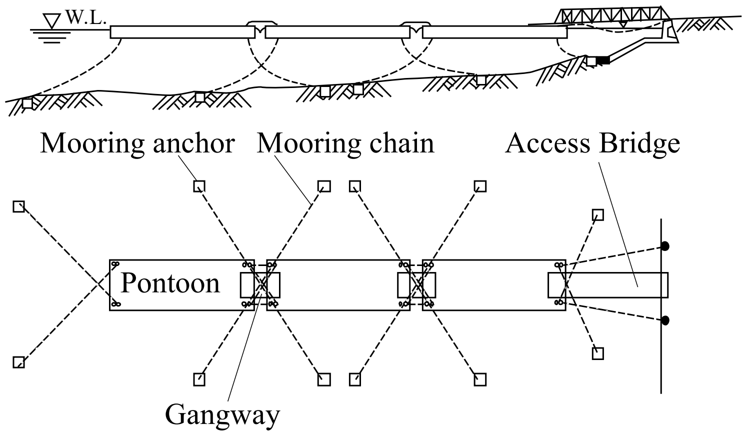


Fig. 1.4.11 Cross Section and Plan of a Floating Pier

⑧ **Dolphins**

(a) Outline:

Dolphins are mooring facilities that comprise a group of columnar structures installed offshore. **Fig. 1.4.12** shows an example of a cross section of a dolphin.

(b) Characteristics:

1) In coastal areas with predetermined depths, dolphins can be easily constructed at low cost and in a short period of time without involving dredging and reclamation work.

2) Dolphins have not been used for handling general cargoes but for handling large quantities of oil, cement, grains, and powdery bulk cargoes with dedicated cargo-handling machines installed on them.

3) Dolphins can be constructed as parts of other main mooring facilities at their bow and stern ends to reduce the lengths of main mooring facilities. When attached to existing mooring facilities, dolphins can extend the effective lengths of existing facilities to be used for mooring.

4) Dolphins are classified into the following categories depending on their structures:

i) Pile-type dolphin;

ii) Steel cellular-bulkhead-type dolphin;

iii) Caisson-type dolphin.

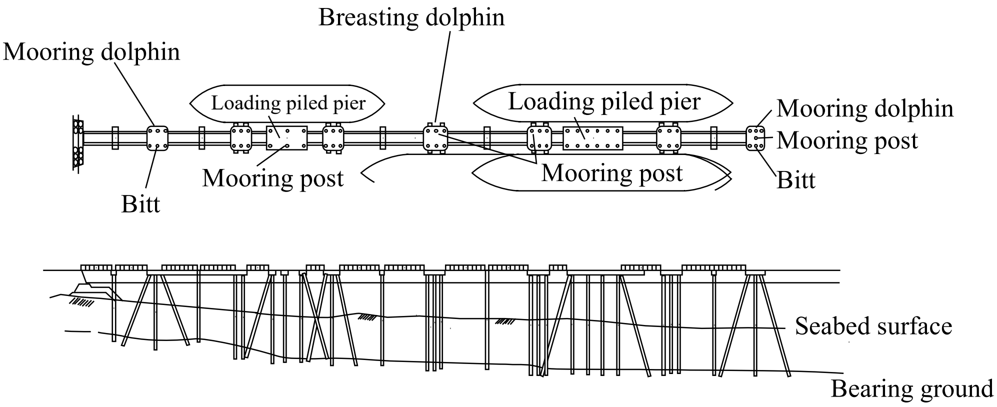


Fig. 1.4.12 Cross Section of a Dolphin

⑨ **Docks**

(a) Outline:

Docks are facilities provided with slip ways to bring ships onshore. **Fig. 1.4.13** shows an example of a cross section of a dock.

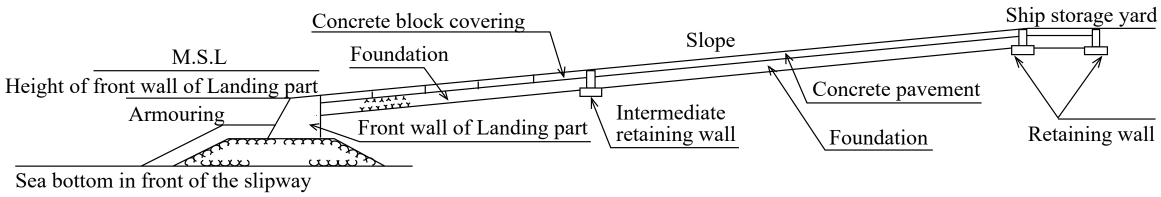


Fig. 1.4.13 Cross Section of a Dock

⑩ **Air-cushion-craft landing facilities**

(a) Outline:

Air cushion crafts are high-speed crafts that navigate above sea surfaces with craft bodies levitated by strong downward airflow. The mooring facilities for air cushion crafts include slip ways, aprons, pontoons, and open-type wharves. Considering that craft bodies have special structures, air cushion crafts require ancillary facilities that are different from normal mooring facilities. Furthermore, the locations of facilities for air cushion crafts should be carefully selected with due consideration to the fact that the navigation of air cushion crafts is largely affected by meteorological and oceanographic conditions and that air cushion crafts generate noise and ship wake waves.

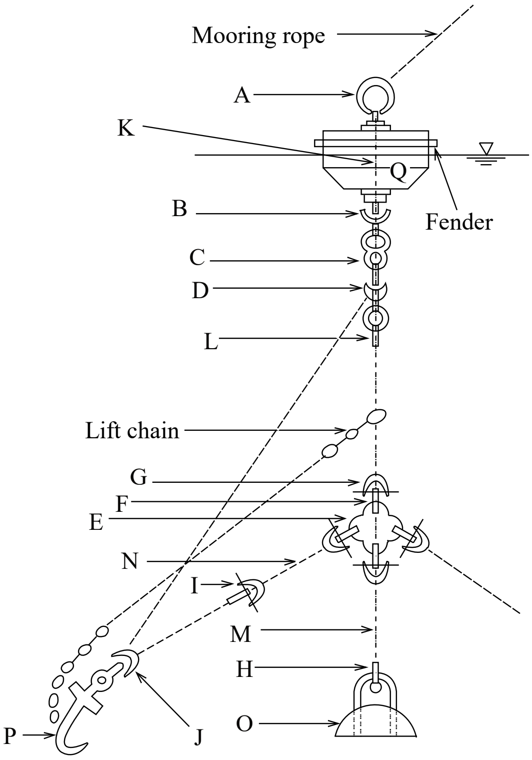
⑪ **Mooring buoys**

(a) Outline:

Mooring buoys are used mainly for mooring ships in basins. Mooring buoys generally comprise floating bodies, mooring rings, anchoring chains, sinkers, and mooring anchors. **Fig. 1.4.14** shows an example of a cross section of a mooring buoy. Mooring buoys have been used for handling petroleum products and timber, cargo handling with barges, and mooring ships without cargo handling operations.

(b) Characteristics:

1) Mooring buoys enable ships to be moored in narrower basin areas than anchoring.
2) Mooring buoys enable ports to accommodate ships that cannot anchor because of exposed bedrock on seafloor surfaces.
3) In coastal areas with predetermined depths, mooring buoys can be easily constructed at low costs and in a short period of time without involving dredging and reclamation work.
4) Mooring buoys can be easily relocated.
5) The operational wave heights of mooring buoys are higher than other types of mooring facilities.
6) It is generally difficult to mechanize cargo-handling operations with mooring buoys. Therefore, in many cases, mooring buoys have lower cargo-handling efficiency than other mooring facilities.
7) Generally, mooring buoys require large basin areas than other mooring facilities.



A; Harp shackle (mooring ring) or quick release hook	1
B; Anchor shackle	1
C; Swivel piece	1
D; Joining shackle	1
E; Mooring piece	1
F; Long ring	2
G; Joining shackle	2
H; Anchor shackle	1
I; Joining shackle	2
J; Anchor shackle	2
K; Chain	1
L; Main chain (Anchoring chain of floating body)	1
M; Sinker chain	1
N; Chain or ground chain	4
O; Sinker	1
P; Anchor or screw	1
Q; Buoy	1

Fig. 1.4.14 Example Cross Section of a Mooring Buoy

1.5 Points of Caution Regarding High-Earthquake-Resistance Facilities

(1) Mooring facilities that are categorized as high-earthquake-resistance facilities shall ensure restorability under the accidental situation with respect to Level 2 earthquake ground motions. Facilities that require additional enhancement in earthquake resistance in accordance with their natural and social conditions shall also ensure usability under this accidental situation. Restorability and usability are defined as the required performances for the functions that need to be operative after the actions of Level 2 earthquake ground motions and not for the functions that need to be fulfilled in normal time.

(2) Classifications of high-earthquake-resistance facilities

Depending on the functions that need to be maintained after the actions of Level 2 earthquake ground motions, high-earthquake-resistance facilities are classified into the following: high-earthquake-resistance facilities (specially designated emergency supply transport), high-earthquake-resistance facilities (specially designated trunk line cargo transport), and high-earthquake-resistance facilities (standard emergency supply transport). The performance requirements and the contents of the design situations are set for the respective facilities according to this classification. Refer to **Table 1.5.1** for the details of the classification of high-earthquake-resistance facilities.

Table 1.5.1 Classification of High-Earthquake-Resistance Facilities

	High-earthquake-resistance facility		
	Specially designated		Standard
	Emergency supply transport	Trunk line cargo transport	Emergency supply transport
Functions required after the actions of Level 2 earthquake ground motions	Facilities need to maintain structural stability after earthquakes so that they can promptly be used for the mooring and landing operation of ships, the embarkation and disembarkation of passengers, and the loading and unloading	Facilities need to maintain structural stability after earthquakes so that they can promptly (in a short period of time) be used for the mooring and landing of ships and the loading and unloading of trunk line cargoes.	Facilities need to maintain structural stability after earthquakes so that they can be used for the loading and unloading of emergency relief supplies after a lapse of a certain period.

	High-earthquake-resistance facility		
	Specially designated		Standard
	Emergency supply transport	Trunk line cargo transport	Emergency supply transport
	of cargoes, including emergency relief supplies.		
	Functions required after earthquakes (Primary functions are not required)	Primary functions	Functions required after earthquakes (Primary functions are not required)
Required performance	Usability*)	Restorability	Restorability*)
Allowable degree of restoration	Minor repairs	Minor repairs	A certain level of repairs

*): The required performance is for the functions to be fulfilled after earthquakes (to transport emergency relief supplies) and not for the primary functions of respective facilities.

① High-earthquake-resistance facilities for emergency supply transport

(a) Specially designated (emergency supply transport)

High-earthquake-resistance facilities (specially designated emergency supply transport) shall ensure usability under the accidental situation with respect to Level 2 earthquake ground motions. Here, usability does not always mean that facilities need to resist the actions of Level 2 earthquake ground motions without being damaged but indicates that facilities need to protect against damage up to a level that retains their function for the loading and unloading of emergency relief supplies.

(b) Standard (emergency supply transport)

High-earthquake-resistance facilities (standard emergency supply transport) shall ensure restorability under the accidental situation with respect to Level 2 earthquake ground motions. Here, restorability means that the facilities need to protect against damage due to the actions of Level 2 earthquake ground motions up to a level wherein emergency restoration can allow the loading and unloading of emergency relief supplies after a lapse of a certain period, i.e., approximately one week after the facilities were subjected to the actions of Level 2 earthquake ground motions.

(c) Difference between specially designated and standard emergency supply transports

The major difference between high-earthquake-resistance facilities (specially designated emergency supply transport) and high-earthquake-resistance facilities (standard emergency supply transport) is the time allowed for these facilities to become available for the loading and unloading of emergency relief supplies after they were subjected to the actions of Level 2 earthquake ground motions. In this regard, it is necessary to plan high-earthquake-resistance facilities for emergency supply transport by taking into consideration this difference.

Specifically, the Basic Disaster Prevention Plan (Article 34 of the Disaster Countermeasure Basic Act) can consider a phased emergency supply transportation operation according to the degrees of emergency of relief supplies (**Table 1.5.2**). According to the table, the planning of high-earthquake-resistance facilities can incorporate such phased operations in a manner that enables high-earthquake-resistance facilities (specially designated emergency supply transport) to serve emergency supply transports from the early phase in region I and high-earthquake-resistance facilities (standard emergency supply transport) to additionally serve emergency supply transports from the intermediary phase in region II.

Table 1.5.2 Phased Emergency Supply Transportation Operation in the Basic Disaster Prevention Plan

Region	Phase		Subject of emergency transportation
I	Phase 1	(1)	Manpower and supplies for lifesaving operations such as rescue, first aid, and medical activities
		(2)	Manpower and supplies for operations to prevent disasters from expanding, such as firefighting and flood control
		(3)	Manpower and supplies for governmental and municipal disaster countermeasures, as well as emergency measures, to restore and protect communication, power, gas and water
		(4)	Injured persons to be transferred to backup medical institutions
		(5)	Manpower and supplies for the urgent restoration of transportation facilities, bases, and traffic control
	Phase 2	(1)	Continuation of phase 1 activities
		(2)	Supplies such as food and water necessary for the maintenance of life
		(3)	Patients, injured persons, and disaster victims to be transferred out of the disaster areas
		(4)	Manpower and supplies necessary for the urgent restoration of transportation facilities
II	Phase 3	(1)	Continuation of phase 2 activities
		(2)	Manpower and supplies necessary for disaster restoration
		(3)	Daily essentials

② **High-earthquake-resistance facilities for trunk line cargo transport**

High-earthquake-resistance facilities (specially designated trunk line cargo transport) shall ensure restorability under the accidental situation with respect to Level 2 earthquake ground motions. Here, restorability means that these facilities need to protect against damage due to the actions of Level 2 earthquake ground motions within predetermined levels (e.g., displacement of the facilities in the ranges allowed for the operation of respective cargo handling machines) to enable these facilities to remain functional for trunk line cargo transport again after minor repairs following the lapse of short periods. The short periods need to be appropriately defined in accordance with the required functions, which differ for each facility.

(3) Standard concept of the limit values for the deformation of high-earthquake-resistance facilities with respect to Level 2 earthquake ground motions

The standard limit values for the deformation of high-earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions can be set in accordance with the required performance of respective facilities, as shown below. However, this provision shall not be applied to cases wherein the limit values for deformation are set on the basis of comprehensive determination by taking into consideration the situations of areas where facilities are constructed, the required performance, and the structural types of the facilities.

① **High-earthquake-resistance facilities for emergency supply transport**

(a) Specially designated (emergency supply transport)

The standard limited values for the residual horizontal deformation and residual inclination angles of high-earthquake-resistance facilities (specially designated emergency supply transport) can be set from a functional viewpoint at approximately 30 to 100 cm and 3°, respectively. For example, the limit value for a residual horizontal deformation of 100 cm can be set for facilities that can ensure the required usability even if the deformation may be large because the materials for emergency repairs are always ready and because emergency response systems are established. When setting the limit values, refer to the performance records of the emergency supply transport that was established immediately after the 1995 South Hyogo Prefecture Earthquake.[1] It has been indicated that the meandering (uneven displacement) of face lines is a more important factor than residual horizontal deformation when evaluating the usability of mooring facilities in terms of ship berthing during emergencies.[2] Accordingly, there is an idea of deriving the limit values for residual deformation by first setting the limit values for uneven displacement and then using the correlation between the uneven displacement and residual deformation. It is also important to curb the level differences between mooring facilities and pavements at their back in terms of the facilitation of cargo handling during emergencies. Curbing horizontal deformation generally curbs the level differences between mooring facilities and pavements at their back.

(b) Standard (emergency supply transport)

The limit values for the residual horizontal deformation of high-earthquake-resistance facilities (standard emergency supply transport) shall be appropriately set at approximately 100 cm or more by taking into consideration the availability of cargo handling operation after a lapse of a certain period following the actions of Level 2 earthquake ground motions.

② **High-earthquake-resistance facilities for trunk line cargo transport**

The limit values for the residual deformation of high-earthquake-resistance facilities (specially designated trunk line cargo transport) shall be set on the basis of the periods necessary for restoring the required functions. Regarding the periods, there are cases wherein it is rational to set shorter periods for earthquakes such as subduction-zone earthquakes, which cause more extensive damage than inland active fault earthquakes, which cause concentrated damage on relatively narrow areas, from the viewpoint of maintaining functions for trunk line cargo transport. In such cases, smaller limit values can be set for subduction-zone earthquakes than for inland active fault earthquakes.

There are many cases of high-earthquake-resistance facilities (specially designated trunk line cargo transport) that are provided with cranes with a seismic isolation or vibration control function to equalize the earthquake resistance of mooring facilities and cranes. In such cases, earthquake response analyses that consider the dynamic interaction between mooring facilities and cranes shall be performed to keep the responses of the structural members of cranes within the elastic limits. For the details of seismic isolation or vibration control cranes, refer to **Part III**, **Chapter 7**, **2.2 Container Cranes**.

[References]

1) Takahashi, H., T. Nakamoto and F. Yoshimura: Analysis of maritime transportation in Kobe Port after the 1995 Hyogoken-Nanbu Earthquake, Technical Note of PHRI No. 861, 1997

2) Kazui, K., H. Takahashi, T. Nakamoto and Y. Akakura: Evaluation of allowable damage deformation of gravity type quaywall during earthquake, Proceedings of 10th Symposium on Earthquake Engineering, K-4, 1998

2 Wharves

2.1 Common Items for Wharves

[Ministerial Ordinance] (Performance Requirements for Quaywalls)

Article 26

1 The performance requirements for quaywalls shall be as prescribed respectively in the following items in consideration of the structural type:

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

(2) Damage to the quaywall, etc. due to the action of self-weight, earth pressure, Level 1 earthquake ground motion, berthing and traction by ships, surcharge loads, etc. shall not impair the functions of the quaywalls and shall not adversely affect the continuous use of the quaywall.

2 In addition to the provisions of the previous paragraph, the performance requirements for quaywalls provided in the following items shall be as prescribed respectively in those items:

(1) "Performance requirements for quaywalls to protect environment" shall be such that quaywalls shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to contribute to conservation of the environment of ports and harbors without impairing the original functions of the quaywalls.

(2) "Performance requirements for quaywalls classified as high earthquake-resistance facilities" shall be such that damage due to the action of Level 2 earthquake ground motions, etc. shall not affect the restoration through minor repair works of functions required for the quatwalls in the aftermath of the occurrence of Level 2 earthquake ground motion. Provided, however, that for the performance requirements for the quaywall which requires further improvements in earthquake-resistant performance due to environmental conditions, social conditions, etc. to which the quaywalls are subjected, damage due to Level 2 earthquake ground motions, etc. shall not impair the functions required for the quatwalls in the aftermath of the occurrence of Level 2 earthquake ground motion, and shall not adversely affect the continuous uses of the quaywalls.

[Public Notice] (Performance Criteria of Quaywalls)

Article 48

1 The performance criteria common to quaywalls shall be as prescribed respectively in the following items:

(1) Quaywalls shall have the water depth and length necessary for accommodating the design ships in consideration of their dimensions.

(2) Quaywalls shall have a crown height that considers the range of tidal levels, the dimensions of the design ship, and the usage conditions of the facilities.

(3) Quaywalls shall have ancillary equipment as necessary in consideration of the usage conditions.

2 In addition to the provisions of the preceding paragraph, the performance criteria for quaywalls specified in the following items shall be as prescribed respectively in those items:

(1) "Performance criteria for quaywalls for the purpose of environmental conservation" shall be such that quaywalls have the dimensions necessary to contribute to conservation of environments of ports and harbors in consideration of the environmental conditions, etc. to which the quaywalls are subjected, without impairing the original functions of the quaywalls.

(2) "Performance criteria for quaywalls classified as high earthquake-resistance facilities" shall be such that the degree of damage under the accidental situation, in which dominating action is Level 2 earthquake ground motion, shall be equal to or less than the threshold level in consideration of the performance requirements.

[Interpretation]

11. Mooring Facilities

(1) Green Quaywalls (Article 26, Paragraph 2, Item 1 of the Ministerial Ordinance on Criteria and Interpretation related toArticle 48, Paragraph 2, Item 1of the Public Notice)

① Quaywalls for protecting the environment are classified as green quaywalls to which the subsequent items shall be applied, in addition to the provisions for quaywalls.

② The performance requirement for green quaywalls shall focus on serviceability. The term “protective capability” refers to the performance of quaywalls in protecting port environments for organisms, ecosystems, and others without impairing their essential functions.

③ The dimensions of quaywalls for protecting environments shall indicate the structure, cross-sectional dimensions, and ancillary facilities. When setting the structure and cross-sectional dimensions in the performance verifications of quaywalls to protect environments and installing ancillary facilities, appropriate consideration shall be given to factors that affect the objective to protect port environments for organisms and ecosystems without impairing the essential functions of the quaywalls.

(2) Quaywalls That Are Classified as High Earthquake-resistance Facilities (Article 26, Paragraph 2, Item 2 of the Ministerial Ordinance on Criteria and Interpretation related to Article 48, Paragraph 2, Item 2 of the Public Notice)

① The following classifications are used as standards in provisions stipulating the appropriate performance of high earthquake-resistance facilities corresponding to the functions necessary after the action of Level 2 earthquake ground motions and the allowable period for restoration to demonstrate those functions.

a) Specially designated (emergency supply transport): facilities that can be used by ships and perform embarkation/disembarkation of persons, cargo handling of emergency supplies, etc., immediately after the action of Level 2 earthquake ground motions.

b) Specially designated (trunk line cargo transport): facilities that can be used by ships and perform cargo handling of trunk line cargoes within a short period after the action of Level 2 earthquake ground motions.

c) Standard (emergency supply transport): facilities that can be used by ships and perform the embarkation/disembarkation of persons, cargo handling of emergency supplies, etc., within a certain period after the action of Level 2 earthquake ground motions.

② The performance requirements for high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action are stipulated in the subsequent items corresponding to the classifications of high earthquake-resistance facilities:

(a) The performance requirement for high earthquake-resistance facilities (specially designated (emergency supply transport)) shall focus on serviceability. Serviceability refers to the limited performance requirements for the functions of facilities deemed necessary for transporting emergency supplies after earthquakes and is independent of the serviceability required for normal cargo handling work in facilities.

(b) The performance requirement for high earthquake-resistance facilities (specially designated (trunk line cargo transport)) shall focus on restorability.

(c) The performance requirement for high earthquake-resistance facilities (standard (emergency supply transport)) shall focus on restorability.

③ **Attached Table 11-1** shows the verification items and standard indexes to determine the limit values that are common to quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action. “Damage” has been adopted as the verification item in **Attached Table 11-1** from the viewpoint of comprehensiveness and by considering the fact that verification items will differ depending on the structural type. Furthermore, the indexes for determining limit values shall be appropriately set for performance verification. It may also be noted that settings in connection with the Public Notice Article 22 (Common Performance Criteria of Component Members of Target Facilities Subject to the Technical Standard) may also be applied when necessary, in addition to this code.

Attached Table 11-1 Verification Items and Standard Indexes for Determining the Limit Values that Are Common to Quaywalls Classified as High Earthquake-resistance Facilities

Ministerial Ordinance			Public Notice			Performance requirements*	Design state			Verification item	Standard indexes to determine the limit values
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	2	2	48	2	2	Restorability, serviceability	Accidental	L2 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Damage	–

*) In this table, "serviceability" refers to the "necessary function after earthquake (emergency supply transport)."
*) In this table, "restorability" refers to the "essential function" or "necessary function after earthquake (emergency supply transport)."

④ **Attached Table 11-2** shows the verification items and standard indexes for determining the limit values for gravity-type quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action. The standard indexes for determining the limit values for the face line deformation quantity of a quaywall in the table can be set with reference to the descriptions in subsequent items corresponding to the classification of high earthquake-resistance facilities.

Attached Table 11-2 Verification Items and Standard Indexes for Determining the Limit Values for Gravity-type Quaywalls Classified as High Earthquake-resistance Facilities

Ministerial Ordinance			Public Notice			Performance requirements*	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	2	2	48	2	2	Restorability, serviceability	Accidental	L2 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Deformation of the face line of the quaywall	Limit of residual deformation

*) In this table, "serviceability" refers to the "necessary function after earthquake (emergency supply transport)."
*) In this table, "restorability" refers to the "essential function" or "necessary function after earthquake (emergency supply transport)."

(a) High earthquake-resistance facilities (specially designated (emergency supply transport))

The limit of deformation of high earthquake-resistance facilities (specially designated (emergency supply transport)) shall be the deformation of a degree such that the berthing of ships for the marine transport of emergency supplies, evacuees, construction machinery for removing obstructions, etc., is possible and shall be set appropriately. In general, the residual horizontal displacement of the quaywall can be used as the index of deformation.

(b) High earthquake-resistance facilities (specially designated trunk line supply transport)

The limit of deformation of high earthquake-resistance facilities (specially designated (trunk line cargo transport)) shall be the deformation of a degree such that trunk line cargo transport can be performed after slight restoration, within the permissible displacement set in line with the characteristics of the cargo handling equipment, or similar and shall be set appropriately. In general, the residual horizontal displacement of the quaywall, residual inclination angle of the wall, and relative displacement of the rail span can be used as indexes of deformation. In the case of quaywalls using cargo handling equipment for trunk line cargo transport, appropriate consideration shall be given to the form, type, and characteristics of the cargo handling equipment when setting limit values.

(c) High earthquake-resistance facilities (standard (emergency supply transport))

The limit of deformation of high earthquake-resistance facilities (standard (emergency supply transport)) shall be the deformation of a degree such that cargo handling of emergency supplies can be performed after emergency restoration within a given period of time and shall be set appropriately. In general, the residual horizontal displacement of the quaywall can be used as the index of deformation.

⑤ **Attached Table 11-3** shows the verification items and standard indexes to determine the limit values for sheet pile quaywalls classified as high earthquake-resistance facilities (specially designated (emergency supply transport) and specially designated (trunk line supply transport)) under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action. The structural types of anchorages are broadly classified as vertical pile anchorage, coupled-pile anchorage, sheet pile anchorage, and concrete wall anchorage. In the performance verification of anchorages, appropriate verification items shall be set corresponding to the structural type. The standard indexes for determining the limit values for the face line deformation quantity in the table shall be equivalent to the performance criteria of gravity-type quaywalls classified as high earthquake-resistance facilities (specially designated (emergency supply transport) and specially designated (trunk line supply transport)).

Attached Table 11-3 Verification Items and Standard Indexes to Determine the Limit Values for Sheet Pile Quaywalls Classified as High Earthquake-resistance Facilities (Specially Designated (Emergency Supply Transport) and Specially Designated (Trunk Line Supply Transport)) with respect to the Accidental Situation

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements*</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index to determine limit values</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="7">26</td><td rowspan="7">2</td><td rowspan="7">2</td><td rowspan="7">48</td><td rowspan="7">2</td><td rowspan="7">2</td><td rowspan="7">Serviceability, restorability</td><td rowspan="7">Accidental</td><td rowspan="7">L2 earthquake ground motion</td><td rowspan="7">Self-weight, earth pressure, water pressure, surcharge</td><td>Deformation of the face line of the quaywall</td><td>Limit of the residual deformation</td></tr>
<tr><td>Yielding of sheet piles</td><td>Design yield stress</td></tr>
<tr><td>Rupture of the tie member</td><td>Design rupture strength</td></tr>
<tr><td>Damage to anchorage*1)</td><td>Limit curvature</td></tr>
<tr><td>Axial force acting on anchorage*2)</td><td>Action-resistance ratio with respect to the bearing force of anchorage (pushing and pulling)</td></tr>
<tr><td>Stability of the anchorage*3)</td><td>Design ultimate capacity of the section</td></tr>
<tr><td>Cross-sectional failure of the superstructure</td><td>Design ultimate capacity of the section</td></tr>
</table>

*1): The structural types of anchorages are limited to cases of vertical pile anchorage, coupled-pile anchorage, and sheet pile anchorage.
*2): The structural types of anchorages are limited to the case of coupled-pile anchorage.
*3): The structural types of anchorages are limited to the case of concrete wall anchorage.
*) In this table, "serviceability" refers to the "necessary function after earthquake (emergency supply transport)" and indicates the required capacity for specially designated (emergency supply transport).
*) In this table, "restorability" refers to the "essential function" and indicates the required capacity for specially designated (trunk line cargo transport).

⑥ **Attached Table 11-4** shows the verification items and standard indexes for determining the limit values for sheet pile quaywalls classified as high earthquake-resistance facilities (standard (emergency supply transport)) under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action. The standard indexes for determining the limit values for the face line deformation quantity in the table shall be equivalent to the performance criteria of gravity-type quaywalls classified as high earthquake-resistance facilities (standard (emergency supply transport)).

Attached Table 11-4 Verification Items and Standard Indexes for Determining the Limit Values for Sheet Pile Quaywalls Classified as High Earthquake-resistance Facilities (Standard (Emergency Supply Transport)) with respect to the Accidental Situation

Ministerial Ordinance			Public Notice			Performance requirements*	Design state			Verification item	Standard index for determining the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	2	2	48	2	2	Restorability	Accidental	L2 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Deformation of the face line of the quaywall	Limit of the residual deformation
										Damage to sheet pile	Limit curvature
										Rupture of the tie member	Design rupture strength
										Damage to anchorage*1)	Limit curvature
										Axial force acting on anchorage*2)	Action-resistance ratio with respect to the bearing force of anchorage (pushing and pulling)
										Stability of the anchorage*3)	Design ultimate capacity of the section
										Cross-sectional failure of the superstructure	Design ultimate capacity of the section

*1): The structural types of anchorages are limited to the cases of vertical pile anchorage, coupled-pile anchorage, and sheet pile anchorage.
*2): The structural types of anchorages are limited to the case of coupled-pile anchorage.
*3): The structural types of anchorages are limited to the case of concrete wall anchorage.
*) In this table, "restorability" refers to the "necessary function after earthquake (emergency supply transport),"

⑦ The verification items and standard indexes for determining the limit values for cantilevered sheet pile quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action shall be equivalent to the provisions for sheet pile quaywalls classified as high earthquake-resistance facilities with the exception of the verification items for tie rods and anchorages.

⑧ The verification items and standard indexes for determining the limit values for double sheet pile quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action shall be equivalent to the provisions for sheet pile quaywalls classified as high earthquake-resistance facilities.

⑨ The verification items and standard indexes for determining the limit values for quaywalls with relieving platforms classified as high earthquake-resistance facilities under the accidental situation with respect to Level 2 earthquake ground motions as the dominant action shall be equivalent to the provisions for gravity-type quaywalls and sheet pile quaywalls classified as high earthquake-resistance facilities corresponding to the structural characteristics of respective members.

⑩ The verification items and standard indexes for determining the limit values for cellular-bulkhead quaywalls classified as high earthquake-resistance facilities under the accidental situation with respect to

Level 2 earthquake ground motions as the dominant action shall be equivalent to the provisions for gravity-type quaywalls classified as high earthquake-resistance facilities.

2.1.1 Dimensions of Wharves

(1) Dimensions of Wharves

① **Length**

The length of a wharf used in the performance verification of a wharf shall be set as the value obtained by adding the necessary lengths of the bow and stern mooring lines to the length overall of the design ship on the premise that the wharf is used exclusively by the design ship.

② **Water depth**

The water depth used in the performance verification of a wharf shall be set as the value obtained by adding the keel clearance corresponding to the design ship to the maximum draft such as the full load draft of the design ship to obtain a value that will not interfere with the use of the design ship.

③ **Crown height**

For the performance verification of the crown height of a wharf, the assumed use conditions of the facilities shall be considered for the safe and smooth use of the wharf.

④ **Ancillary equipment**

For the performance verification of a wharf, the ancillary equipment shall be considered for the safe and smooth use of the wharf. The performance requirements for the ancillary equipment of mooring facilities are stipulated in Article 33 of the Ministerial Ordinance on Criteria (Performance Requirements for the Ancillary Facilities of Mooring Facilities), and the performance criteria for the ancillary equipment are stipulated in Articles 60 to 74 of the Public Notice corresponding to the types of ancillary equipment.

⑤ **Shape of the wall and front toe**

In addition to the items provided here, for the performance verification of a wharf, the shape of the wall and front toe of the wharf (clearance limits of structure) shall be appropriately set so as to prevent them from coming into contact with the wharf during berthing.

(2) Length, Water Depth, and Layout of Berths

① The length and water depth of berths should be appropriately set based on the ship dimensions.

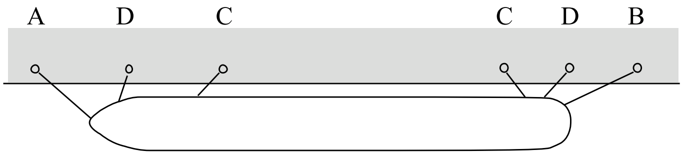
② The mooring lines shown in **Fig. 2.1.1** are desirable when a vessel is moored parallel to a wharf. The bow and stern lines are usually set at an angle of 30° to 45° with the quay face because these lines are used to prevent both the longitudinal movement (in the bow and stern directions) and lateral movement (in the onshore and offshore directions) of the vessel.

③ The water depth of berths can be calculated using **equation (2.1.1)**. The maximum draft represents the maximum draft in a calm water condition such as the condition when the ship is moored, e.g., full-load draft of the design ship. For keel clearance, a value of approximately 10% of the maximum draft or more is desirable. However, in mooring facilities where ships may moor for harborage in abnormal weather or similar conditions, the addition of a keel clearance that considers wind and wave factors is necessary.

Berth water depth = Maximum draft + Under keel clearance **(2.1.1)**

④ In the case of a berth where flammable dangerous cargoes are handled, it is necessary to keep a distance of 30 m or more between oil tanks, boilers, and working areas that use open fire in the cargo handling area and the mooring vessel at the berth. However, the distance may be shortened to approximately 15 m when there is no risk that the cargo may catch fire in the event of leakage because of the surrounding topography or structure of the facilities of the berth.

⑤ In the case of a berth where flammable dangerous cargoes are handled by tankers and so on, a distance of 30 m or more should be kept between tankers and other anchored vessels and the space for the vessels navigating nearby to navigate 30 m or more away from those tankers should be secured. However, this distance may be increased or decreased as necessary in consideration of the size of the cargo carrying vessel, the type and size of the vessels anchored or navigating nearby, and the condition of ship congestion.



A: bow line
B: stern line
C: spring lines
D: breast lines

Fig. 2.1.1 Arrangement of Mooring Ropes

⑥ **Standard values of wharf dimensions**

For setting the length and water depth of a wharf in cases where the design ship cannot be identified, the standard values of the main dimensions of wharves by ship type shown in **Table 2.1.1** can be used. These standard values have been set on the basis of the standard values of the main dimensions of the design ships shown in **Part II, Chapter 8 Ships, Table 1.1.1**. In principle, the standard values shown in **Table 2.1.1** have been set assuming that the design ship moors parallel to the wharf; however, the standard values for ferries have been also set by assuming cases of bow and stern-side berthing-type wharves in addition to that case. With regard to the standard values for small cargo ships, given that there are large deviations in comparison with other ship types, due consideration of this point is necessary when applying the standard values shown in **Table 2.1.1** in setting the length and water depth of wharves for small cargo ships, as well as applying the standard values of main dimensions of small cargo ships. The gross tonnage GT in **Table 2.1.1, 4** to **7** is either international gross tonnage or Japanese gross tonnage, and the gross tonnage used is specified at the table.

Table 2.1.1 Standard Values of the Main Dimensions of Berths in Cases Where the Design Ship Cannot Be Identified

1. Cargo Ships

Deadweight tonnage DWT (t)	Length of the berth (m)	Water depth of the berth (m)
1,000	80	4.5
2,000	100	5.5
3,000	110	6.0
5,000	130	7.0
6,000	140	7.5
10,000	160	9.0
12,000	170	9.0
15,000	180	10.0
18,000	190	11.0
30,000	230	12.0
40,000	250	13.0
50,000	260	14.0
55,000	270	15.0
70,000	280	16.0
90,000	310	17.0
120,000	340	19.0
150,000	360	20.0
200,000	390	22.0
250,000	420	23.0
300,000	430	25.0
400,000	470	26.0

2. Container Ships

Deadweight tonnage DWT (t)	Length of the berth (m)	Water depth of the berth (m)
10,000	170	9.0
20,000	220	11.0
23,000	230	12.0
27,000	240	13.0
30,000	250	13.0
40,000	290	13.0
50,000	330	14.0
60,000	350	15.0
100,000	410	16.0
140,000	440	17.0
165,000	470	18.0
185,000	500	18.0
200,000	500	18.0

3. Tankers

Deadweight tonnage DWT (t)	Length of the berth (m)	Water depth of the berth (m)
1,000	80	4.5
2,000	100	5.5
3,000	110	6.5
5,000	130	7.5
10,000	170	9.0
15,000	190	10.0
20,000	210	11.0
30,000	230	12.0
50,000	260	14.0
70,000	280	15.0
90,000	310	16.0
100,000	320	17.0
150,000	360	19.0
300,000	440	25.0

4. Roll-On Roll-Off (RORO) Ships

4.1 RORO Ships (GT: Japanese gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
3,000	150	6.5
5,000	180	7.5
10,000	220	9.0
15,000	220	9.0

4.2 RORO Ships (GT: international gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
20,000	240	10.0
40,000	250	11.0
60,000	270	12.0

5. Pure Car Carriers (PCC)

5.1 PCC (GT: Japanese gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
3,000	150	5.5
5,000	170	7.0
40,000	260	12.0

5.2 PCC (GT: international gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
12,000	180	7.5
20,000	200	8.0
30,000	230	9.0
40,000	240	11.0
60,000	260	12.0
70,000	290	12.0

6. Passenger Ships (GT: international gross tonnage)

Gross tonnage GT (t)	Length of the berth (m)	Water depth of the berth (m)
3,000	130	5.0
5,000	150	5.5
10,000	180	7.0
20,000	220	8.0
30,000	260	8.0
50,000	310	9.0
70,000	340	9.0
100,000	360	10.0
130,000	390	10.0
160,000	410	10.0

* In setting the berth lengths and water depths for passenger ships with the maximum dimensions, it is desirable to set them after acquiring ship dimensions by using the latest Lloyd's data and Clarkson data and so on. The maximum dimensions of the existing passenger ships and those planned to be built are length of 362.1 m, molded breadth of 47.0 m, and full-load draft of 10.3 m, as shown in **Part II, Chapter 8, 1.1 Standard Values, Table 1.1.5**.

7. Ferries (GT: Japanese gross tonnage)

7-1 Intermediate- and Short-Distance Ferries (sailing distance less than 300 km in Japan)

Gross tonnage GT (t)	Case of the bow and stern side berthing -type		
	Length of the berth (m)	Length of the bow and stern-side berthing-type quaywall (m)	Water depth of the berth (m)
400	60	20	3.5
700	80	20	4.0
1,000	90	25	4.5
3,000	130	25	5.5
7,000	170	30	7.0
10,000	200	30	7.5
13,000	220	35	8.0

7-2 Long-Distance Ferries (sailing distance of the 300 km or more in Japan)

Gross tonnage GT (t)	Case of no bow- and stern side–type berthing	Case of the bow and stern side type berthing		Water depth of the berth (m)
	Length of the berth (m)	Length of the berth (m)	Length of the bow and stern-side berthing-type quaywall (m)	
6,000	190	170	30	7.5
10,000	220	200	30	7.5
15,000	250	230	40	8.0
20,000	260	250	40	9.0

8. Small Cargo Ships

Deadweight tonnage DWT (t)	Length of the berth (m)	Water depth of the berth (m)
700	70	4.0

(3) Crown Height of Wharves

① The crown height of wharves shall be appropriately set in consideration of the subsequent items:

• Safe and smooth cargo handling and embarkation and disembarkation of passengers

• Relations between the freeboards and respective full-load and unloaded draft of design ships

• Uplift force on piled pier

• Possibility of inundation due to storm surge

• Possibility of inundation due to waves

• Possibility of inundation due to tsunamis

• Possibility of the consolidation settlement of the ground and predicted consolidation settlement

• Ease of inspections, diagnoses, and repair work in the maintenance phase (particularly for piled pier)

• Possibility of ground subsidence due to crustal movement after large-scale earthquakes

• Others

② The mean monthly-highest water level can be used as the tidal level used for the datum level of the crown height of wharves.

③ In cases where the design ship cannot be identified, in general, the values shown in **Table 2.1.2** are widely used as the crown height of wharves. It should be noted that the values in this table are expressed using the mean monthly-highest water level as a datum level.

Table 2.1.2 Standard Crown Heights of Wharves

	Tidal range 3.0 m or more	Tidal range less than 3.0 m
Wharf for large vessels (water depth of 4.5 m or more)	+0.5 to 1.5 m	+1.0 to 2.0 m
Wharf for small vessels (water depth of less than 4.5 m)	+0.3 to 1.0 m	+0.5 to 1.5 m

(4) Clearance Limits of Wharves

① The shape of the wall and front toe of the wharves shall be appropriately set so as to prevent them from coming into contact with ships during berthing.

② The clearance limits of wharves shall be set so that they comply with the Sounding Procedure based on the **Implementation Procedure[1] of the Memorandum of Understanding on Hydrographic Survey Associated with Port Construction** between the Ports and Harbours Bureau and the Japan Coast Guard (March 31, 1972).

③ In the cross sections of a vessel, the bottom-corner sections are slightly rounded and have projecting bilge keels. In many cases, the radius of curvature of the corner sections and the height of the bilge keels are 1.0 to 1.5 m and 30 to 40 cm, respectively and the shape of corner sections may be assumed to be nearly 90°. The planned water depths of berths are generally deeper than the maximum draft of the design vessel by 0.3 m or more.

④ According to the **Implementation Procedure of the Memorandum of Understanding on Hydrographic Survey Associated with Port Construction**, water depth in the vicinity of wharves shall be surveyed as far as 1 m from fenders.

⑤ **Fig. 2.1.2** shows the clearance limit for wharves set in consideration of the above items and past examples.[2), 3)] The clearance limit of wharves may be determined using this figure as reference. However, care should be exercised in using the clearance limit shown in the figure because the rolling, pitching, and heaving motions of vessel at berth have not been taken into consideration in the figure.

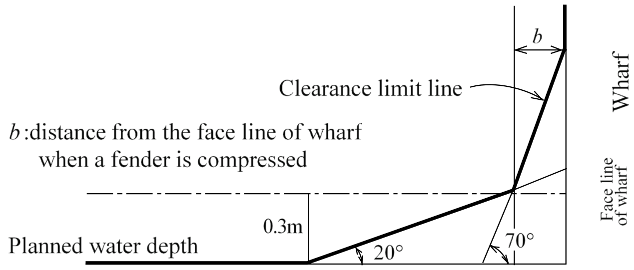


Fig. 2.1.2 Clearance Limit for Mooring Facilities

(5) Design Water Depth

① In general, the design water depth is not equal to the planned water depth. The design water depth is ordinarily obtained by adding a margin to the planned water depth to guarantee the required stability of the mooring facility. Considering that this margin will vary according to the structural type, the water depth of the site, the construction method and accuracy, and the result of scouring, it is desirable that the design water depth is carefully determined with due consideration given to these factors.

② It may be difficult to determine the depth of scouring owing to current or the berthing of specific vessels when setting the design water depth even with the risk of large scouring in front of mooring facilities. In such a case, it is advisable to provide scour prevention measures as described in **Part III, Chapter 5, 2.1.2 Protection against Scouring** instead of considering the depth of scouring as margins.

2.1.2 Protection against Scouring

(1) In cases wherein large scouring is anticipated at the front side of a wharf owing to the currents or turbulence generated by ship propellers, the front of the mooring facility shall be protected with armor stones, concrete blocks, or other materials as a measure against scouring.

(2) In the case that ships may drop anchors at the front side of a wharf, it is necessary to focus on protection area against scouring and on the appropriate selection of scouring protection materials to prevent anchors from dragging.

(3) Examples of overseas test models of the measures against scouring include armor rocks, gabions, a layer of concrete blocks on a geotextile filter, a layer of pillow-like geotextile bags filled with concrete, etc. Furthermore, there has been an inventive plan to install a deflector (curved plate) on a sea bottom to deflect the water current due to propellers toward a sea surface.[4)]

(4) When deliberating measures against scouring, it is advisable to refer to the current velocity calculation formula[5)] and the calculation formula of the required mass of foundation materials[6)] based on the Isbash formula (**Part II, Chapter 2, 6.6.3 Required Mass of Armor Stones and Blocks to Resist Currents**).

2.1.3 Green Quaywalls

(1) Green quaywalls[7)] are a type of quaywalls that contribute to the development of a favorable port environment and hospitable habitats such as tidal flats and rocky shores for organisms in accordance with natural situations wherein quaywalls are located (**Reference (Part I), Chapter 3, 2 Green Port Structures**). Green quaywalls can also be developed by adding the habitat functions to existing quaywalls concurrently with their improvement works.

(2) Environmental research and numerical model analyses shall be employed when identifying the influences on the goal of creating habitats for organisms (**Reference (Part I), Chapter 3, 2 Green Port Structures**). When verifying the performance of green quaywalls, it is necessary to confirm whether structures, cross sections, and ancillary facilities are expected to achieve the goal.

(3) The performance requirement for green quaywalls is that the quaywalls should have the habitat function, and its influences are the presence or absence of foundations related to the inhabitation of organisms; external forces such as waves, currents, etc.; and environments that are necessary for the inhabitation of organisms. The environments necessary for the inhabitation of organisms refer to water depth and water transparency, which affect the light intensity necessary for photosynthesis and water temperature, which affects the activities of organisms. Specifically,

when aiming at the cultivation of a seaweed bed, the structure and cross section of a quaywall and the texture and gradient of ancillary facilities need to allow objective seaweed and sea grass to root and maintain the structural sun light shading effects to a level that does not affect the light intensity necessary for seaweed and sea grass to grow.

(4) The performance verification of green quaywalls shall be implemented in a manner that confirms whether the environments of the locations are within the inhabitable range of objective organisms on the basis of existing knowledge. For example, in the performance verification of a quaywall with the inhabitation of seaweed, the performance verification shall be implemented in a manner that confirms whether such environments are within the inhabitable range of objective seaweed because seaweed is subject to the environments represented by light intensity affecting photosynthesis and respiration, as well as water temperature. Furthermore, in the case where changes in the environmental conditions after the installation of green quaywalls and environmental variations in the future are predictable, the performance of a quaywall can be verified by confirming whether such changes or variations in the future are within the inhabitable range of living organisms on the basis of a numerical model related to the growth of organisms.

(5) The performance verification of green quaywalls shall be implemented by referring to **Part III, Chapter 4, 4 Green Breakwaters** and **Reference (Part I), Chapter 3, 2 Green Port Structures** and the **Guidelines for the Development, Maintenance and Management of Green Port Structures**.[7)]

2.2 Gravity-type Quaywalls

[Public Notice] (Performance Criteria for Gravity-type Quaywalls)

Article 49

The performance criteria for gravity-type quaywalls shall be as prescribed respectively in the following items:

(1) The risk of sliding failure of the ground under the permanent state, in which the dominating action is self-weight, shall be equal to or less than the threshold level.

(2) The risk of failure due to the sliding or overturning of the quaywall body and the insufficient bearing capacity of the foundation ground under the permanent state, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion, shall be equal to or less than the threshold level.

[Interpritation]

11. Mooring Facilities

(3) Performance Criteria of Gravity-type Quaywalls (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance on Criteria and the interpretation related to Article 49 of the Standard Public Notice)

① The required performance of gravity-type quaywalls under the permanent state in which the dominant actions are self-weight and earth pressure and under the variable state in which the dominant actions are Level 1 earthquake ground motions shall focus on serviceability. **Attached Table 11-5** shows the performance verification items and standard indexes for determining limit values with respect to the actions.

Attached Table 11-5 Performance Verification Items and Standard Indexes to Determine the Limit Values under the Respective Design Situations of Gravity-type Quaywalls except Accidental Situations

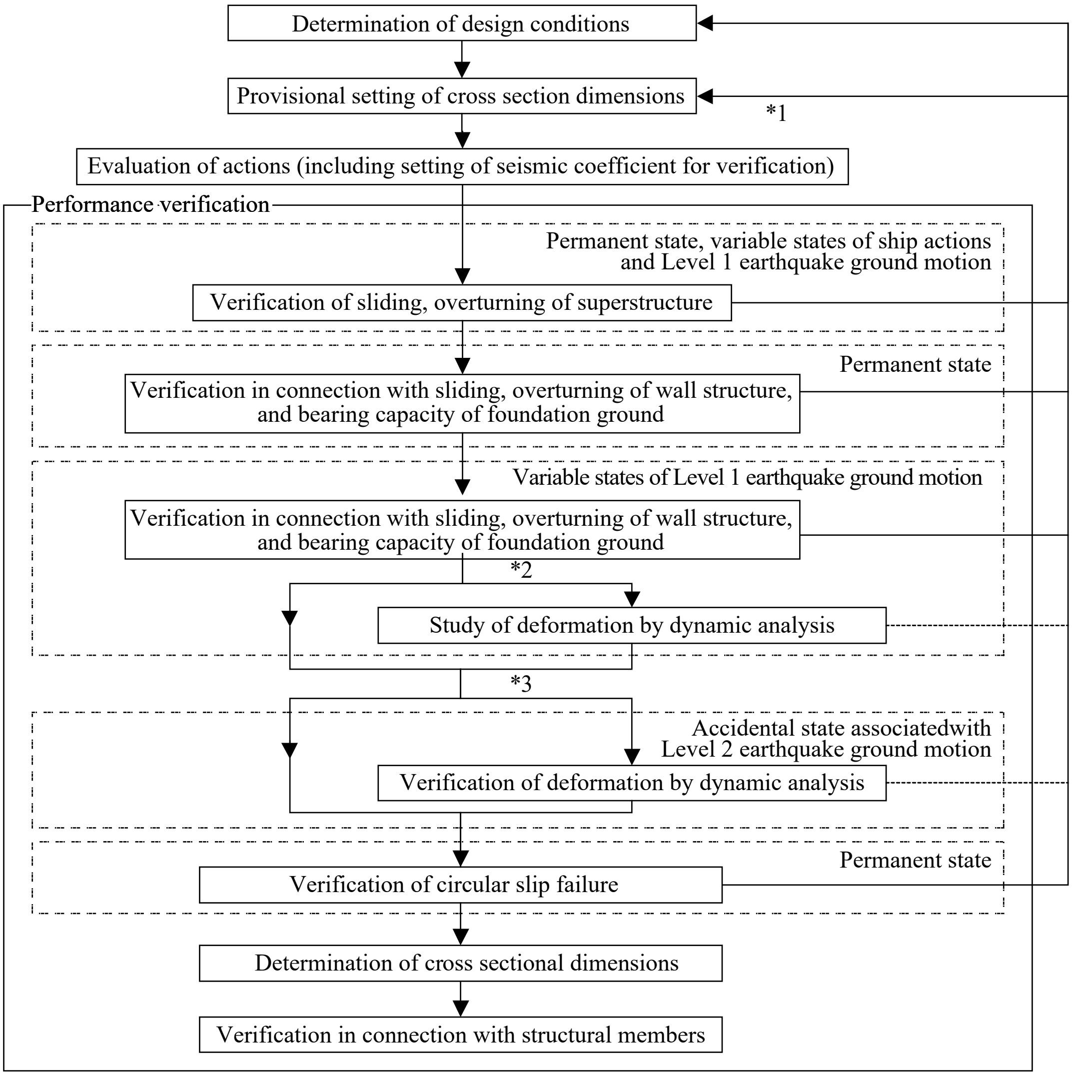
<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index to determine the limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="3">26</td><td rowspan="3">1</td><td rowspan="3">2</td><td rowspan="3">49</td><td rowspan="3">–</td><td>1</td><td rowspan="3">Serviceability</td><td rowspan="2">Permanent</td><td>Self-weight</td><td>Water pressure, surcharges</td><td>Circular slip failure of the ground</td><td>Action–resistance ratio with respect to circular slip failure</td></tr>
<tr><td rowspan="2">2</td><td>Earth pressure</td><td>Self-weight, water pressure, surcharges</td><td>Sliding, overturning of the quaywall, bearing capacity of the foundation ground</td><td>Action–resistance ratios with respect to sliding, overturning, and bearing capacity</td></tr>
<tr><td>Variable</td><td>Level 1 earthquake ground motion</td><td>Self-weight, earth pressure, water pressure, surcharges</td><td>Sliding, overturning of the quaywall, bearing capacity of the foundation ground</td><td>Action–resistance ratios with respect to sliding, overturning, and bearing capacity</td></tr>
</table>

② In addition to this code, the provisions and commentaries in connection with Paragraph 3 (Scouring and Wash Out), Article 22 and Article 28 (Performance Criteria of Armor Stones and Blocks) of the Standard Public Notice shall be applied as necessary, and the provisions and commentaries in connection with Article 23 and Article 27 of the Standard Public Notice shall be applied depending on the type of members comprising the objective gravity-type quaywalls.

2.2.1 General

(1) Depending on the types of wall structures, gravity-type quaywalls are classified as caisson-type quaywalls, L-shaped block-type quaywalls, mass-concrete block-type quaywalls, cellular concrete block-type quaywalls, cast-in-place concrete-type quaywalls, upright wave-dissipating-type quaywalls, and others. The description provided herein can be applied to the performance verification of these gravity-type quaywalls. Regarding upright wave-absorbing-type quaywalls, the performance verification method shown in **Part III, Chapter 5, 2.11 Upright Wave-absorbing-type Quaywalls** can be used as a reference.

(2) **Fig. 2.2.1** shows an example of the performance verification procedure for gravity-type quaywalls. However, because the figure does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions, when examining the effects of liquefaction, it is necessary to appropriately deliberate the possibility of and the measures against liquefaction with reference to **Part II, Chapter 7 Ground Liquefaction**. The seismic coefficient method based on the static equation of equilibrium can be used to examine gravity-type quaywalls under a variable state with respect to Level 1 earthquake ground motions. By contrast, for gravity-type quaywalls classified as high earthquake-resistance facilities, the deliberation of deformation amounts is preferably performed by nonlinear earthquake response analysis or other methods by taking into consideration the dynamic interaction between ground and structures. For those gravity-type quaywalls other than high earthquake-resistance facilities, the performance verification for an accidental situation with respect to Level 2 earthquake ground motions can be omitted.



*1 As the effects of liquefaction, subsidence etc. are not included in this procedure, separate consideration is necessary.

*2 When necessary, study of deformation by dynamic analysis for Level 1 earthquake ground motion is possible. In high-earthquake-resistance facilities, study of deformation by dynamic analysis is preferable.

*3 For high-earthquake-resistance facilities, verification for level 2 earthquake ground motion is performed.

Fig. 2.2.1 Example of Performance Verification Procedure for Gravity-type Quaywalls

(3) **Fig. 2.2.2** shows an example of a cross section of a gravity-type quaywall.

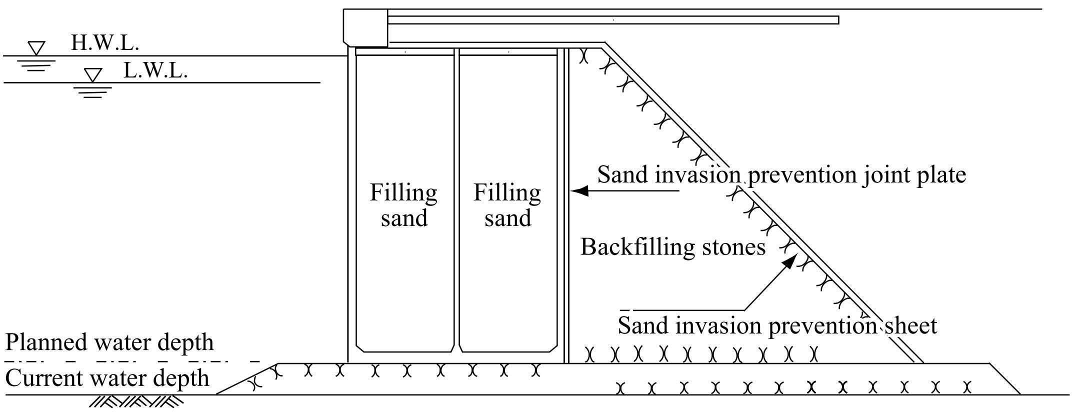


Fig. 2.2.2 Example of a Cross Section of a Gravity-type Quaywall

(4) For the gravity-type quaywalls of special structural types, it is preferable to perform performance verification by using laboratory experiments and numerical analyses.

For example, so-called caisson-type quaywalls with inclined bottom surfaces constructed on seafloors inclined landward may have a large resistance against sliding but require sufficient performance verification with respect to increased bottom reaction force and rocking due to earthquake ground motions via laboratory experiments and numerical analyses. Clearance limits should also be considered. For the performance verification of caisson-type quaywalls with inclined bottom surfaces, refer to **References 8)** and **9)**.

2.2.2 Actions

(1) Types of Actions to Be Considered in Respective Design Situations

The stability verification of gravity-type quaywalls shall consider the following actions in respective design situations. The performance verification for accidental situation can be omitted in the case where the gravity-type quaywalls to be designed are not high-earthquake-resistance facilities.

① **Permanent state**

The dominant actions shall be the self-weight of wall bodies and earth pressure acting on wall bodies.

② **Variable state**

The dominant actions shall be Level 1 earthquake ground motions. For the setting of Level 1 earthquake ground motions, refer to **Part II, Chapter 6, 1.2 Level 1 Earthquake Ground Motions Used for Performance Verification of Facilities**.

③ **Accidental state**

The dominant actions shall be Level 2 earthquake ground motions. For the setting of Level 2 earthquake ground motions, refer to **Part II, Chapter 6, 1.3 Level 2 Earthquake Ground Motions Used for Performance Verification of Facilities**.

(2) Points of Caution When Setting Actions

① Seismic coefficients for verification used in the verification of damage due to sliding and overturning of wall bodies and failures due to insufficient bearing capacity of foundation ground in variable state in respect of Level 1 earthquake ground motion[10), 11)]

(a) For the performance verification with respect to the sliding and overturning of wall bodies, as well as failures due to insufficient bearing capacity under a variable state with respect to Level 1 earthquake ground motions, the seismic coefficient method can also be used in place of the direct evaluation of deformation amounts by using nonlinear response analyses. In such a case, the seismic coefficients for verification to be used in performance verification need to be appropriately set in accordance with the deformation amounts of facilities by taking into consideration the effects of frequency characteristics and duration of earthquake ground motions.

(b) Calculation of the characteristic value of the seismic coefficient for verification

The characteristic values of the seismic coefficients for verification to be used for the performance verification of gravity-type quaywalls installed at depths of -7.5 m or deeper can be calculated with **equation (2.2.1)** by using the maximum corrected acceleration α_c stipulated in **Reference [Part III), Chapter 1, 1 Detailed Items about Seismic Coefficient for Verification** and the allowable deformation amounts D_a of the crowns of quaywalls. The seismic coefficients for verification should be expressed in numerical values rounded off to two decimal places. However, for the calculation of the seismic coefficients for verification in the case of ground improvement through the deep mixing method or the sand compaction pile (SCP) method with a replacement rate of 70% or higher, refer to **Part III, Chapter 2, 5.5 Deep Mixing Method** and **Part III, Chapter 2, 5.10 Sand Compaction Pile Method (for Cohesive Ground)**.

$$k_{h_k} = 1.78\left(\frac{D_a}{D_r}\right)^{-0.55}\frac{\alpha_c}{g} + 0.04 \qquad \textbf{(2.2.1)}$$

where

k_{h_k} : characteristic value of a seismic coefficient for verification;

α_c : maximum corrected acceleration (cm/s^2);

g : gravitational acceleration (980 cm/s^2);

D_a : allowable deformation amount of the crown of a quaywall (10 cm);

D_r : reference deformation amount (10 cm).

By contrast, for the characteristic values of the seismic coefficients for verification k_{h_k} to be used for the performance verification of gravity-type quaywalls installed at depths of less than -7.5 m, refer to the seismic coefficients for the verification of the **Reference for Design of Fishery Ports and Fishing Ground Facilities, Chapter 6, 2.2.3 Design Horizontal Seismic Coefficients That Take into Consideration Frequency Characteristics and Deformation Amounts**.[12)] For those gravity-type quaywalls installed at depths of approximately -7.5 m, the seismic coefficients are preferably set in consideration of the possible discrepancies between those calculated via **equation (2.2.1)** and the methods in **Reference 12)**.

(c) Setting of the allowable amount of deformation

It is necessary to appropriately set the allowable amount of deformation for facilities in accordance with the functions required for the facilities and circumstances in which the facilities are placed. The allowable value of the standard deformation amount of gravity-type quaywalls in Level 1 earthquake ground motion may be taken to be D_a = 10 cm. This allowable value of the standard deformation amount (D_a = 10 cm) is the average value of the amounts of residual deformation of existing gravity-type quaywalls in Level 1 earthquake ground motion, which was calculated by seismic response analyses. The standard deformation amount is determined by taking into consideration the margin of safety necessary to ensure that the seismic performance verification method is accurate enough to prevent the functions of facilities from being impaired by Level 1 earthquake ground motions. Thus, the standard deformation amount is set to be sufficiently smaller than the allowable limit deformation amounts of actual facilities.

(d) The calculation method for the characteristic value of the seismic coefficient for verification in (b) above is based on the condition allowing for no liquefaction. When applying the method to other conditions, the applicability needs to be deliberated via 2D seismic response analyses or model experiments.

(e) The calculation method for the characteristic value of the seismic coefficient for verification in (b) above is based on the allowable deformation amounts D_a in the range of 5 to 20 cm. Therefore, attention is required when applying the method to the allowable deformation amounts in other ranges.

(f) In some areas where small values have been set for Level 1 earthquake ground motions, this calculation method may produce significantly small seismic coefficients for verification. Even in those areas, the seismic coefficients for verification shall be set at the lower limit value of 0.05 by taking into consideration the uncertainty of the hazard analyses used for obtaining Level 1 earthquake ground motions, the accuracy of the calculation method for the seismic coefficients for verification, and the setting method for allowable deformation amounts.

(g) Considering that the above calculation method may result in excessively large seismic coefficients for verification, any of the following measures can be taken when the calculation produces values larger than 0.25 provided that deformation is preferably confirmed directly by dynamic analyses or other methods even when any of the measures in 2) to 4) is taken:

1) Setting of a cross section with a seismic coefficient for verification of 0.25 and evaluation of the cross section by dynamic analysis that is capable of dealing with a dynamic mutual interaction between the ground and a structure

2) Deliberation of ground improvement

3) Adoption of another structural type

4) Deliberation of the changing allowable deformation D_a of a facility without excessively enlarging it while satisfying the requirement to keep the damage due to Level 1 earthquake ground motions to a

gravity-type quaywall within a level that enables the function of the facility and prevents the impairment of its continuous use.

(h) When implementing the deep mixing method or the SCP method with a replacement rate of 70% or higher, the calculation method for the characteristic value of the seismic coefficient for verification can be applied to the performance verification of gravity-type quaywalls provided that an appropriately set reduction coefficients are used. The reduction coefficients are determined on the basis of the comparison of the 2D effective stress analysis results between unimproved and improved ground. For the details of the reduction coefficients, refer to **Part III, Chapter 2, 5.5 Deep Mixing Method** and **Part III, Chapter 2, 5.10 Sand Compaction Pile Method (for Cohesive Ground)**.

(i) When constructing quaywalls without improving very soft, normally consolidated, cohesive soil layers, the calculation method for the characteristic value of the seismic coefficient for verification may underestimate the seismic coefficients for verification from the viewpoint of deformation amounts. Therefore, in such cases, the deformation amounts should be evaluated directly by using detailed methods, such as nonlinear effective stress analyses.

(j) Soft ground is subjected to large shear strain when oscillated by strong earthquake ground motions, thus aggravating damage to quaywalls. However, in some cases, analyses do not produce large earthquake ground motions on ground surfaces and underestimate the seismic coefficients for verification. Therefore, the 1D earthquake response analysis codes to be used for verification should be able to appropriately evaluate the amplification of earthquake ground motions in soft ground, particularly the amplification of the acceleration in the frequency ranges critical for calculating the seismic coefficients for verification.

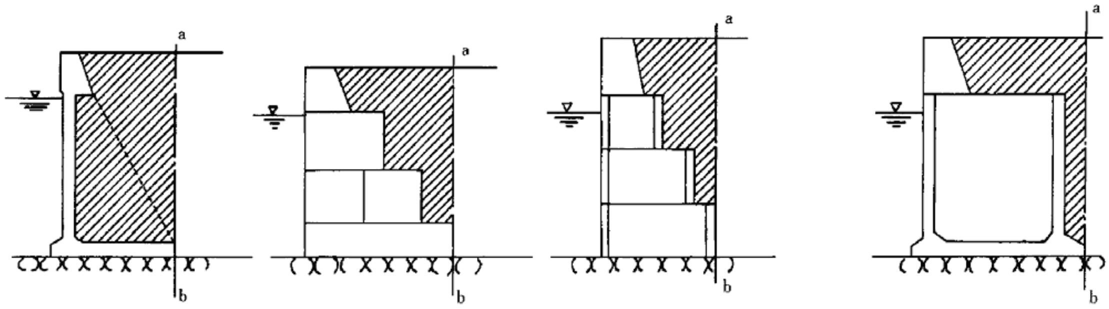
(k) When applying the seismic coefficient method to performance verification by using seismic coefficients for verification in a vertical direction, such coefficients shall be appropriately set in accordance with the characteristics of the facilities and ground.

(l) The seismic coefficients should be calculated for verification on the basis of appropriate deliberation before their use in the performance verification of structural members under an accidental situation with respect to Level 2 earthquake ground motions. Regarding the damage to caisson quaywalls in Kobe Port due to the 1995 South Hyogo Prefecture Earthquake, the failure of bottom slabs of caissons has not been reported even though they underwent large deformation. Furthermore, there is little knowledge on the seismic coefficients for verification with respect to Level 2 earthquake ground motions. Therefore, the seismic coefficients for verification to be used for the performance verification of structural members under an accidental situation with respect to Level 2 earthquake ground motions can be calculated by method (b) above by using the acceleration time history of the ground surface of the free ground area for convenience. In such a case, the allowable deformation amount D_a can be set to 50 cm. However, the seismic coefficients for verification shall be up to 0.25 and equal to or larger than the seismic coefficients for verification for Level 1 earthquake ground motions. When the seismic coefficients for verification for Level 1 earthquake ground motion are higher than 0.25, the seismic coefficients for verification for Level 2 earthquake ground motions can also be higher than 0.25.

② **Determination of wall body portions**

(a) In cases wherein stability needs to be verified by substituting inertia force for the actions of earthquake ground motions, it is necessary to assess the inertia force on the basis of the appropriate determination of the quaywall bodies. In such cases, the quaywall bodies can be set as shown below depending on the types of structures. This setting of quaywall bodies shall not be applied to cases wherein deformation amounts are assessed directly by a detailed method such as nonlinear effective stress analysis or similar methods.

(b) **Fig. 2.2.3** shows that the wall bodies of gravity-type quaywalls can be defined as the portions between the face lines of the quaywalls and the vertical planes passing through the rear toes of the quaywalls. Normally, wall bodies are provided with backfills behind them. In many gravity-type quaywalls, some parts of the backfill are positioned above the wall bodies and act as parts of the quaywall bodies. However, it is difficult to apply this concept to all cases unconditionally because the extent of backfill that is considered part of the quaywall bodies varies depending on the shapes of the quaywall bodies and the modes of failures. In general, the extent of backfill that is considered part of the quaywall bodies can be defined by the shaded area in **Fig. 2.2.3** to simplify the design calculation because modest changes in the locations of the quaywall body boundary planes do not affect the stability of the quaywall bodies significantly.



(A) L-shaped block (B) Mass concrete block (C) Cellular concrete block (D) Caisson

Fig. 2.2.3 Determination of Wall Body Portions

(c) In cases wherein quaywall structures require stability during the examination of respective horizontal strata, similar to the case for block-type quaywalls, the determination of virtual wall bodies may be performed as follows. Normally, tenons are provided between blocks for better interlocking; however, the interlocking effects of the tenons are preferably ignored in the examination of the following virtual wall bodies.

1) Examination of sliding

Fig. 2.2.4 shows that the portion in front of the vertical plane passing through the rear toe at the level under examination can be considered a wall body.

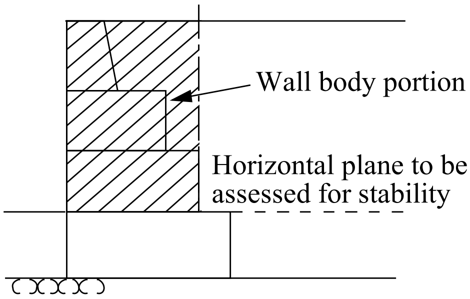


Fig. 2.2.4 Determination of Wall Body Portion for the Stability of Sliding at a Horizontal Joint

2) Examination of Overturning

The backfill in front of a vertical plane passing through the most landward side edge among blocks stacked on a block placed on the seaward side above the plane subject to stability examination may be regarded as a part of the wall body. For example, in the case of a block-type quaywall (**Fig. 2.2.5**), the weight of the portion in front of the vertical plane (shown by hatched lines) through the block placed on block Ⓒ on the seaward side can be considered to resist overturning; however, the weights of block Ⓑ and soil Ⓐ are not considered to contribute to overturning resistance.

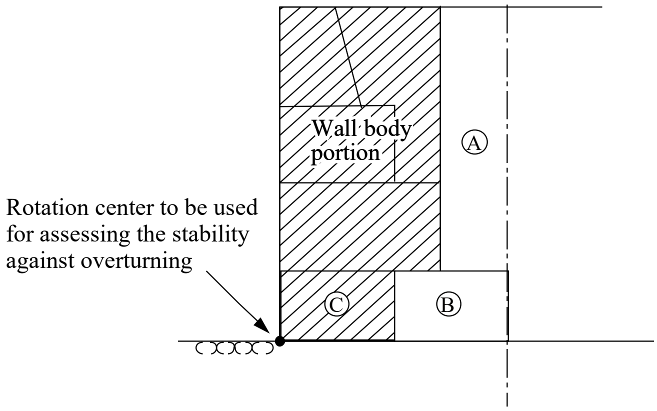


Fig. 2.2.5 Determination of Wall Body Portion for Stability against Overturning

3) Examination of Failure due to the Inadequate Bearing Capacity of Foundation Ground

In examining failures, the portion in front of the vertical plane passing through the rear toe of a wall body can be considered a virtual wall body. In the case of a cellular block wherein the wall body and a filling section appear to have different bottom reactions, the lowermost block shall preferably be constructed as an integrated block.

③ The residual water level should be set at a level one-third of the tidal range above the mean monthly lowest water level (LWL). For the design tidal levels, refer to **Part II, Chapter 2, 3.6 Design Tidal Level Conditions**. In general, the range of the residual water level difference increases as the tidal range increases and as the permeability of the wall body material decreases. Water behind the wall body permeates through voids in the wall joints, foundation mound, and backfill. The residual water level difference can be reduced by improving the permeability of these materials. On the contrary, care is necessary because this approach may result in the leakage of the backfill material.

The abovementioned value of the residual water level is applicable to cases in which long-term permeability can be secured. In cases wherein permeability is low from the initial stage or permeability reduction is expected to be reduced over a long-term, it is preferable to assume a large residual water level difference in consideration of these conditions.

A residual water level difference may occur when wave troughs hit the front face of a wall body in general; however, it is not necessary to consider the increase in the residual water level difference due to wave attacks in the performance verification of quaywalls.[13)]

④ For the wall friction angle, δ = 15° can be used. For L-shaped blocks, the shear resistance angle of the backfilling material at the virtual back plane can be used. For details, the **Technical Manual for L-Shaped Block Quaywalls**[14)] may be used as a reference.

⑤ The surcharge may be determined in accordance with **Part II, Chapter 10, 3 Surcharge**.

⑥ Buoyancy is affected by numerous indeterminate factors. Therefore, it is preferable to set buoyancy by considering the worst-case scenario for the facilities concerned. For example, as shown in **Fig. 2.2.6**, buoyancy may be calculated for the submerged portion of the wall body below the residual water level. This approach is applicable to cases in which the difference between the front water level and residual water level is within a normal level; in cases wherein the difference in water levels is remarkable, buoyancy must be set appropriately on the basis of the natural conditions of the objective facilities concerned and other relevant factors.

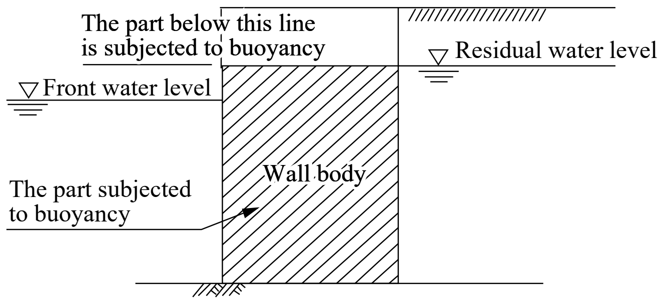


Fig. 2.2.6 Assumption for Calculating Buoyancy

⑦ To obtain earth pressure during earthquake ground motions, it is normal practice to use the equations for the calculation of earth pressure proposed by Mononobe and Okabe, which is shown in **Part II, Chapter 4, 2.3 Earth Pressure during Earthquake**. However, this is based on the concept of the seismic coefficient method, and the calculation results differ from the actual earth pressures resulting from the dynamic interaction of structures, soil, and water. Shaking table tests have shown that the inertial force of wall bodies and the earth pressure during earthquake ground motions have phase differences because they oscillate in the opposite phase when the ground is dense and in the same phase when the ground is loose, e.g., due to liquefaction. In principle, liquefaction is not considered in the performance verification under variable situation with respect to Level 1 earthquake ground motions. Therefore, it is necessary to consider that the inertial force of wall bodies and the earth pressure during earthquake ground motions have opposite phases.[15)] The seismic coefficients for verification explained above allow performance verification to be made in accordance with the deformation of quaywalls by taking into consideration the differences in phases.

⑧ **Dynamic water pressure acting on wall bodies during earthquake ground motions**

For the dynamic water pressure acting during an earthquake, refer to **Part II, Chapter 4, 3.2 Dynamic Water Pressure**.

⑨ **Earth pressure reduction effect by backfill**

In cases wherein high-quality backfill is placed (e.g., a backfill material with a shear resistance angle of 40° is used for rubble), the earth pressure reduction effect by the backfill can be obtained using an analytical method (calculation of earth pressure by discrete method) that takes into consideration the composition of the soil behind the wall body and the strength of each layer behind the quaywall.[16)] In ordinary gravity-type quaywalls, rubble or cobble stones are used as the backfill material. In this case, the earth pressure reduction effect may be assessed using the following simplified method[17)]:

(a) When the cross section of a backfill is triangular

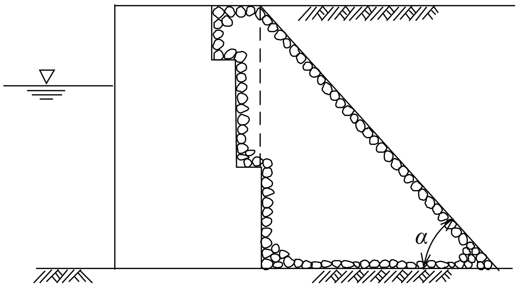
When a backfill is laid in a triangular shape from the point of intersection of the vertical line passing through the rear toe of a quaywall and the ground surface with an angle of slope less than the angle of repose α of the backfill material (**Fig. 2.2.7(a)**), the rear side of wall is entirely filled with backfill material. When the reclaiming material is slurry like cohesive soil, the application of filling-up work or the installation of sand invasion prevention sheets to the surface of the backfill shall be used to prevent the slurry cohesive soil from passing through the voids in the backfill and from reaching the quaywall.

(b) When the cross section of the backfill is rectangular

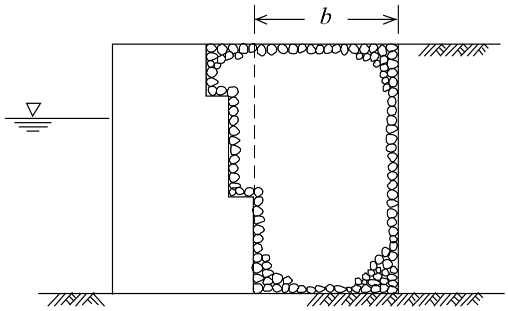
In the case of a triangular-shaped backfill with a slope steeper than the angle of repose of the backfill material or any other irregular-shaped backfill, the effect may be considered similar to a case with a rectangular-shaped backfill that has an area equivalent to the backfill in question. The effect of the rectangular backfill shown in **Fig. 2.2.7(b)** may be considered as follows.

1) When width b of the rectangular-shaped backfill is larger than the height of the wall, this case should be considered in the same manner as the case with a triangular backfill in **Fig. 2.2.7(a)**. When width b is equal to 1/2 of the height, it shall be assumed that the earth pressure is equivalent to the mean of the earth pressure due to the backfill and the reclaimed soil.

2) If the width b is 1/5 or less of the height of the wall, the earth pressure reduction effect due to the backfill shall not be considered.



(a) Triangular shape backfill



(b) Rectangular shape backfill

Fig. 2.2.7 Shapes of Backfill

2.2.3 Performance Verification

(1) Performance Verification Items

When conducting performance verification for the overall stability of structures on the basis of the static equation of equilibrium under a permanent state with respect to self-weight and under a variable state with respect to earth pressure and Level 1 earthquake ground motions as well as performance verification of structures under an accidental situation with respect to Level 2 earthquake ground motions, the necessary performance verification items shall be appropriately set with reference to **Part III, Chapter 5, 2.2 Gravity-type Quaywalls, [Interpretation], Attached Table 11-5** and to **Part III Chapter 5, 2.1 Common Items for Wharves, [Interpretation], Attached Tables 11-1 and 11-2**. When conducting the performance verification of structures under a variable state with respect to Level 1 earthquake ground motions by using nonlinear effective stress analyses, the performance verification items shall be set similar to the performance verification of structures under an accidental situation with respect to Level 2 earthquake ground motions. Furthermore, when gravity-type quaywalls to be designed are not high earthquake-resistance facilities, the performance verification for accidental situation can be omitted.

(2) Performance Verification for the Overall Stability of Structures under a Permanent State with respect to Self-weight

① Examination of the sliding failure of the ground

(a) In cases wherein the foundation ground is weak, circular slip failure from an arbitrary point behind the intersection of the vertical plane through the rear toe of the wall and the bottom plane of the rubble may be examined.

(b) The verification of the circular slip failure of the foundation ground under the permanent situation with respect to self-weight can be performed using **equation (2.2.2)**. In this equation, subscripts *k* and *d* indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.2.1**, in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

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

$$R_k = \sum \left[\{c'_k\, s + (w'_k + q_k)\cos^2\theta \tan\phi'_k\} \sec\theta \right] \tag{2.2.2}$$

$$S_k = \sum \left\{ (w'_k + q_k + q_{RWL_k}) \sin\theta \right\}$$

where

c' : undrained shear strength for cohesive soil ground or apparent adhesion under a drained condition for sandy soil ground (kN/m^2);

s : width of a segment (m);

w' : effective weight of a segment (kN/m) (atmospheric weight when above the water surface or underwater weight when below the water surface);

q : surcharge acting on a segment (kN/m);

q_{RWL} : weight of water, i.e., $\rho_w g(\mathrm{RWL} - \mathrm{LWL})s$, in a segment corresponding to the difference in water levels between the residual water level (RWL) at the back of a facility and a tidal level (LWL) in front of a facility in a case where RWL is higher than LWL (kN/m);

ϕ' : apparent shear resistance angle based on effective stress (°);

θ : angle between the bottom face of a segment and a horizontal plane (°) (Refer to **Part III, Chapter 2, 4 Slope Stability**);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.1 Partial Factors Used for the Performance Verification of the Circular Slip Failure of Foundation Ground

Verification object	Coefficient of variation of cohesive soil in a representative soil layer CV	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Circular slip failure of foundation ground (Permanent state)	Case of no cohesive soil in the layer where a circle passes through	0.83	1.01	– (1.00)
	Less than 0.10	0.86	1.05	– (1.00)
	Not less than 0.10 and less than 0.15	0.85	1.04	– (1.00)
	Not less than 0.15 and less than 0.25	0.80	1.02	– (1.00)
	Not less than 0.25	– (1.00)	– (1.00)	1.30

(c) The partial factors shown in **Table 2.2.1** have been set with reference to the safety levels in the past standards.[18] Furthermore, the *CV*s of cohesive soil in the table can be determined using the *CV*s corresponding to correction factor b_1 obtainable in the process of calculating the characteristic values of adhesion in **Part II, Chapter 3, 2.1 Estimation of the Physical Property of the Ground**. In such a case, among the soil layers (excluding thin ones) where circles can pass through, the soil layer that has the largest *CV* can be the representative soil layer.

(d) Regarding the partial factors for circular slip failure, when the objective ground is subjected to soil improvement using SCP with a replacement rate of 30–80% under the wall body, those partial factors shown in **Part III, Chapter 2, 5.10 Sand Compaction Pile Method (for Cohesive Ground)** can be used.

(3) Performance Verification of the Overall Stability of Structures under a Permanent State with respect to Earth Pressure and Variable State in respect of Level 1 Earthquake Ground Motions

① Examination of sliding of wall bodies

The examination of the stability of wall bodies against sliding can be performed using **equation (2.2.3)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.2.2** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience. The partial factors (permanent state) shown in the table have been set with reference to the safety levels in the past standards.[19]

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

$$R_k = f_k (W_k + P_{Vk} - P_{Bk}) \tag{2.2.3}$$

$$S_k = P_{Hk} + P_{wk} + P_{dwk} + P_{Fk}$$

where

f : *a* friction coefficient between the bottom face of a wall body and a foundation;

W : weight of the materials constituting a wall body (kN/m);

P_V : resultant vertical earth pressure acting on a wall body (kN/m);

P_B : buoyancy acting on wall (kN/m);

P_H : resultant horizontal earth pressure acting on a wall body (kN/m);

P_w : resultant residual water pressure acting on a wall body (kN/m)

P_{dw} : resultant dynamic water pressure acting on a wall body (kN/m) (only during earthquakes);

P_F : inertia force acting on a wall body (kN/m) (only during earthquakes);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.2 Partial Factors Used for the Performance Verification of the Sliding of Wall Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of a wall body (Permanent state)	0.87	1.06	– (1.00)
Sliding of a wall body (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.00

The characteristic values of the resultant dynamic water pressure P_{dw} in **equation (2.2.3)** can be calculated with the following equations:

$$P_{dw_k} = \frac{7}{12} k_{h_k} \rho_w g h^2 \tag{2.2.4}$$

$$P_{F_k} = k_{h_k} W_k \tag{2.2.5}$$

where

ρ_w : density of seawater (t/m^3);

g : gravitational acceleration (m/s^2);

h : water depth in front of a wall body (depth from the bottom face of a wall body to the water level in front of a wall body) (m);

k_h : seismic coefficient for verification.

Furthermore, when a caisson has footings with rectangular cross sections on both sea and land sides, the characteristic value of buoyancy can be calculated using the following equation.

$$P_{B_k} = \rho_w g \left[\left(wl_k + h' \right) B + 2h_f B_f \right] \tag{2.2.6}$$

where

ρ_w : density of seawater (t/m^3);

g : gravitational acceleration (m/s^2);

wl : residual water level (m);

h' : installation depth of a wall body (m);

B : width of a wall body (m);

h_f : height of a footing (m);

B_f : width of a footing (m).

(a) When using friction enhancement mats under the bottom faces of wall bodies to enhance the stability against sliding during the action of earthquake ground motions, particular attention is required for the inconsistency between a verification equation based on the static equation of equilibrium and deformation mechanisms.[20), 21)] Therefore, the earthquake-resistance performance of wall bodies shall be performed in accordance with **Part III, Chapter 5, 2.2.4 Performance Verification for the Deformation Amounts of Facilities during Earthquakes**. All calculation methods for the seismic coefficients for verification to be used for the verification of failures due to the sliding, overturning of the quaywall, and insufficient bearing capacity of foundation ground under variable state with respect to Level 1 earthquake ground motions have been established for the condition to obtain the widths of wall bodies without the use of a friction enhancement mat.

(b) The following vertical force acting on a wall body is normally considered in examining the sliding of a wall body:

1) The value obtained by subtracting buoyancy from the weight of the wall body excluding surcharges (e.g., load of bulk cargoes) anterior to the virtual boundary plane of the wall body

2) The vertical component of earth pressure acting on virtual boundary plane

(c) The following horizontal force acting on a wall body is normally considered in examining the sliding of a wall body:

1) The horizontal component of the earth pressure acting on the virtual boundary plane of a wall body with a surcharge applied to the surface of backfilling soil.

2) Residual water pressure

3) In addition to the above, for the performance verification during the actions of earthquake ground motions, the inertia force and dynamic water pressure acting on the wall body, the horizontal component of the earth pressure during earthquakes, and the horizontal force of cargo handling equipment acting on the wall body through the legs of the equipment

(d) The coefficient of friction can be set in accordance with **Part II, Chapter 11, 9 Friction Coefficient**.

(e) In cases of quaywalls with horizontal joints, similar to the case with block-type quaywalls, it is preferable that the quaywalls are provided with tenons that enable the respective joints to exert sufficient interlocking effects and have strength to resist the horizontal force applied to them. The structures of tenons can follow **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

(f) Even when rubble foundations or foot protection blocks are installed in front of wall bodies for the purpose of scour prevention or the protection of the foots of slopes, it is advisable that the performance verification with respect to sliding are performed without considering the resistance of the rubble foundations or foot protection blocks to the sliding of the wall bodies.

② **Examination of the stability against overturning**

The examination of the stability of a wall body against overturning can be performed using **equation (2.2.7)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.2.3** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience. The partial factors (permanent state) shown in the table have been set with reference to the safety levels in past standards.[19)]

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

$$R_k = (aW_k - bP_{B_k} + cP_{V_k}) \tag{2.2.7}$$

$$S_k = dP_{H_k} + eP_{w_k} + hP_{dw_k} + iP_{F_k}$$

where

W : weight of materials comprising a wall body (kN/m);

P_B : buoyancy acting on a wall body (kN/m);

P_V : resultant vertical earth pressure acting on a wall body (kN/m);

P_H : resultant horizontal earth pressure acting on a wall body (kN/m);

P_w : resultant residual water pressure acting on a wall body (kN/m);

P_{dw} : resultant dynamic water pressure acting on a wall body (kN/m) (only during earthquakes);

P_F : inertia force acting on a wall body (kN/m) (only during earthquakes);

a : distance from the action line of resultant weight of wall to the front toe of a wall body (m);

b : distance from the action line of buoyancy to the front toe of a wall body (m);

c : distance from the action line of resultant vertical earth pressure to the front toe of a wall body (m);

d : distance from the action line of resultant horizontal earth pressure to the bottom of a wall body (m);

e : distance from the action line of resultant residual water pressure to the bottom of a wall body (m);

h : distance from the action line of resultant dynamic water pressure to the bottom of a wall body (m) (only during earthquakes);

i : distance from the action line of inertial force to the bottom of a wall body (m) (only during earthquakes);

R : resistance term (kN·m/m);

S : load term (kN·m/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.3 Partial Factors Used for the Performance Verification of the Overturning of Wall Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of a wall body (Permanent state)	0.99	1.23	—— (1.00)
Overturning of a wall body (Variable state with respect to Level 1 earthquake ground motions)	—— (1.00)	—— (1.00)	1.10

The characteristic value of residual water pressure P_{w_k} shall be appropriately calculated by referring to **Part II, Chapter 4, 3.1 Residual Water Pressure**. In cases wherein caissons have footings with a rectangular cross section on both the sea and shore sides, **equation (2.2.6)** can be used for calculating the characteristic value of buoyancy.

③ **Examination of the bearing capacity of foundation ground**

(a) When examining the bearing capacity of shallow foundations, the force acting on the bottom of wall bodies is the resultant force of loads acting in the vertical and horizontal directions; therefore, this force can be examined using **Part II, Chapter 2, 3.2.5 Bearing Capacity for Eccentric and Inclined Actions**. For the standard partial factor used in performance verification, the values shown in **Table 2.2.4** may be used.

(b) The performance verification of the stability of the bottom of a wall body with respect to the bearing capacity of ground may be performed by using **equation (2.2.8)**. The partial factors in the equation can be selected from the values in **Table 2.2.4**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. In **Table 2.2.4**, the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience. When using **equation (2.2.8)**, an auxiliary parameter E_f needs to be determined via repeated calculation so that E_f satisfies $R_k = E_f \times S_k$ (with attention to the fact that R_k is a function of E_f), and the performance verification of bearing capacity can be performed using the R_k and S_k obtained as a result of the repeated calculation.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$F_f = \frac{R_k(F_f)}{S_k}$$
$$R_k = \sum \left[\frac{\{c'_k\, s + (w'_k + q_k)\tan\phi'_k\}\sec\theta}{1 + \tan\theta \tan\phi'_k / F_f} \right] \qquad \textbf{(2.2.8)}$$
$$S_k = \sum \{(w'_k + q_k)\sin\theta\} + \frac{aP_{H_k}}{r}$$

where

P_H : value of a horizontal action on a soil mass inside a slip failure circle (kN/m);

a : distance from an action position of P_H to the center of a slip failure circle passing through the action position (m);

c' : undrained shear strength for cohesive soil ground or apparent adhesion under drained condition for sandy soil ground (kN/m^2);

s : width of a segment (m);

w' : effective weight of a segment (kN/m) (atmospheric weight when above water surface or underwater weight when below water surface);

q : surcharge acting on a segment (kN/m);

ϕ' : apparent shear resistance angle based on effective stress (°);

θ : angle between the bottom face of a segment and a horizontal plane (°);

F_f : auxiliary parameter representing a ratio of a resistance term to a load term;

r : radius of a slip failure circle (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.4 Partial Factors Used for the Performance Verification of the Failure of Bearing Capacity of Foundation Ground

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Failure of bearing capacity of foundation ground (Permanent state)	—— (1.00)	—— (1.00)	1.20
Failure of bearing capacity of foundation ground (Variable state in respect of Level 1 earthquake ground motions)	—— (1.00)	—— (1.00)	1.00

For the characteristic values and distribution widths of the surcharge loads acting on segments, reference can be made to **Part III, Chapter 2, 3.2.5 Bearing Capacity for Eccentric and Inclined Actions**.

(c) In general, the examination of the bearing capacity of foundation ground is performed with no surcharge applied to a wall body. However, considering that a surcharge causes eccentricity to be decreased but the vertical component force to be increased, the examination may also be performed for the case with a surcharge applied to a wall body as necessary.

(d) The thickness of the foundation mound can be determined by examining the failures due to the insufficient bearing capacity of the foundation ground, the flatness of mound surfaces on which wall bodies are installed, the alleviation of local stress concentration in the ground, etc. The minimum thickness should be determined in accordance with the following guidelines:

1) For a quaywall with a water depth of less than 4.5 m, a thickness of 0.5 m or more; provided, however, that the thickness of the mound shall be at least three times the average diameter of rubbles.

2) For a quaywall with a water depth of 4.5 m or more, a thickness of 1.0 m or more; provided, however, that the thickness of the mound shall be at least three times the average diameter of rubbles.

(e) There have been few cases of gravity-type quaywall structures using foundation piles. In such cases, the performance verification can be performed in accordance with **Part III, Chapter 2, 3.4 Pile Foundations**. In cases of bearing piles driven into the ground susceptible to settlement with the bottom surfaces of wall bodies directly placed on the piles, the structures of wall bodies are destabilized below the bottom surfaces of wall bodies owing to cavities, thus causing the outflow of backfill materials. In such cases, gravity-type quaywalls need to have structures with pile heads covered by rubble mounds.

④ **Examination of settlement**

(a) Gravity-type quaywalls shall ensure their structural stability against settlement due to the consolidation of the ground in accordance with the characteristics of the ground and structures.

(b) For the foundation ground susceptible to settlement, it is important to implement sufficient soil investigation and preliminarily estimate settlement amounts in accordance with **Part II, Chapter 5, 1 Ground Settlement**. It is preferable to take measures for setting higher foundation surfaces or enabling superstructures to be used for final adjustment to achieve predetermined crown heights on the basis of the estimated settlement amounts. It is also necessary to pay attention to the possibility that uneven settlements may cause joint failures and discontinuity or superstructure and apron pavement failures.

(4) Performance Verification for Accidental Situation with respect to Level 2 Earthquake Ground Motions

The performance verification of the seismic resistance of gravity-type quaywalls for Level 2 earthquake ground motions shall be performed by specifically calculating the deformation amounts of facilities via appropriate earthquake response analyses or experiments with reference to **Part III, Chapter 5, 2.2.4 Performance Verification for the Deformation Amounts of Facilities during Earthquakes**. The standard limit values of deformation amounts under accidental situation with respect to Level 2 earthquake ground motions can be appropriately set by referring to Chapter 5, 1.5 Points of Cautions for High Earthquake-resistance Facilities.

2.2.4 Performance Verification for the Deformation Amounts of Facilities during Earthquakes

(1) Performance Verification Methods for the Deformation Amounts of Facilities during Earthquake

Performance verification methods for the deformation amounts of facilities can be broadly classified into two types: methods employing a seismic response analysis and shaking tests using a shaking table or similar apparatus. When performing the performance verification of deformation amounts via seismic response analysis, the appropriateness of the analysis methods should be confirmed by the simulation analyses of damaged cases. For example, FLIP[22)] is one of the methods that have been confirmed applicable to port facilities, such as gravity-type quaywalls. Other methods can also be used properly provided that their applicability has been confirmed by damage simulation analyses and the like.

In cases wherein there have been no damaged cases to confirm the applicability of existing analysis methods to structural types that have been newly adopted, it is necessary to confirm the applicability of the methods through the simulation analyses of appropriate shaking test results.

The general performance verification methods are explained below with the points of caution for their usage. For the details of earthquake response analyses, refer to **Reference (Part III), Chapter 1, 2 Basic Items for Earthquake Response Analyses**.

① **Methods employing seismic response analysis**

Seismic response analyses can be classified as shown in **Table 2.2.5**. In the following, the various types of seismic response analysis methods are explained in accordance with these classifications. Depending on the seismic response analysis methods, these methods may not be suitable in some cases for the purpose of verifying deformation. Therefore, it is necessary to select an analysis method that corresponds to the intended purpose on the basis of the following explanations.

Table 2.2.5 Classification of Seismic Response Analyses

Analysis method (Applicable types of saturated ground)	Effective stress analysis method, total stress analysis method (Combination of solid and liquid layers or solid layer)
Object domain of calculation (Dimensions)	1D, 2D, 3D
General calculation models	Multiple reflection model, point mass model, finite element model
Material characteristics	Linear, equivalent linear, nonlinear
Calculation domain	Time domain analysis method and frequency domain analysis method

(a) Effective stress analysis method and total stress analysis method

From the viewpoint of the prediction and determination of liquefaction and the prediction of soil deformation behavior, seismic response analyses can be classified into analyses based on the effective stress and those based on total stress. In most cases, it is necessary to consider the reduction in effective stress due to the generation of excessive pour pressure in the ground (the extreme reduction in effective stress leads to liquefaction) when predicting the deformation of port facilities during the action of earthquake ground motions. This should be considered because the deformation and response characteristics of the ground are changed as a result of the changes in the stress–strain relation and the attenuation characteristics of soil associated with change in the stress state of the soil such as the reduction in effective stress. The effective stress analysis method can directly calculate the excessive pour pressure generated in the ground. By contrast, the total stress analysis method does not calculate the change in the excessive pour pressure. Therefore, in cases wherein excessive pour pressure exceeds a certain level (approximately 0.5 or more in terms of the excessive pour pressure ratio depending on conditions), the calculation results of the total stress analysis may have large differences from the actual earthquake response of the ground.

There are many cases of using the total stress analysis in practical business because of its simplicity; however, the use of the effective stress analysis is the basic requirement in the performance verification of the deformation of port facilities that are at risk of liquefaction.

(b) Classification based on calculation domain (dimensions)

The seismic response analyses can be classified into 1D, 2D, and 3D methods depending on the calculation of object domains. The 1D method is generally applied to the seismic response analyses of ground with geological stratum structures comprising widely and horizontally accumulated planar strata. However, analysis methods with higher dimensions are required because the facilities used as objects of deformation verification have 3D structures. The 2D method is normally used for the deformation verification of facilities such as quaywalls in the structure–ground system with uniform characteristics in a depth direction. Although the 3D method is required when dealing with regions that include structures such as piles, special elements such as pile–ground cross-interaction springs are normally used to enable such regions to be analyzed by the 2D method. The 3D method is not generally used in practical business because of the necessity to deal with complicated models that require long calculation times; however, this method has been used for the deformation verification of important facilities and for experimental purposes.

(c) Types of general calculation models

1) Multiple reflection model

This calculation model considers that the ground comprises a series of horizontally accumulated soil layers and that the shear waves vertically entering the soil layers from the ground upwardly propagate while repeating transmission and reflection at the boundaries between soil layers. This model is applicable to the earthquake response analyses using linear or equivalent linear methods and is generally inapplicable to the deformation verification of structures.

2) Lumped mass model

This model deals with the ground and structures as a combination of one or more mass, springs, and attenuation mechanisms. This model can be analyzed by relatively simple calculation programs, thus enabling nonlinear deformation–restorative force relation to be incorporated into springs. Although this model enables deformation amounts to be obtained via simple calculation, it is not accurate enough to be a model that can be generally used for detailed deformation verification. This model has been frequently used in dynamic analyses to calculate stress acting on buildings and buried structures (pipes and piles).

3) Finite element model

This model divides ground and structures into a finite number of elements (**Fig. 2.2.8**) and has been used in a wide range of fields. One of the characteristics of this model is its ability to simply represent 2D changes in the layer thicknesses and physical property of the ground. Finite element analysis programs already in practical use include FLUSH[23)] and FLIP[22)] and others. It is necessary to pay attention to the fact that programs like FLUSH, which employ the equivalent linear method, are not suitable for the prediction of residual deformation. By contrast, the applicability of FLIP to the deformation verification of many port facilities has been confirmed by the analyses of damage to port facilities due to the South Hyogo Prefecture Earthquake.[24)] There has been a report that FLAC,[25)] which is one of the finite difference analysis programs based on the explicit method, can also be used for the deformation analyses in the same way as the finite element methods subject to the conformity with constitutive laws.

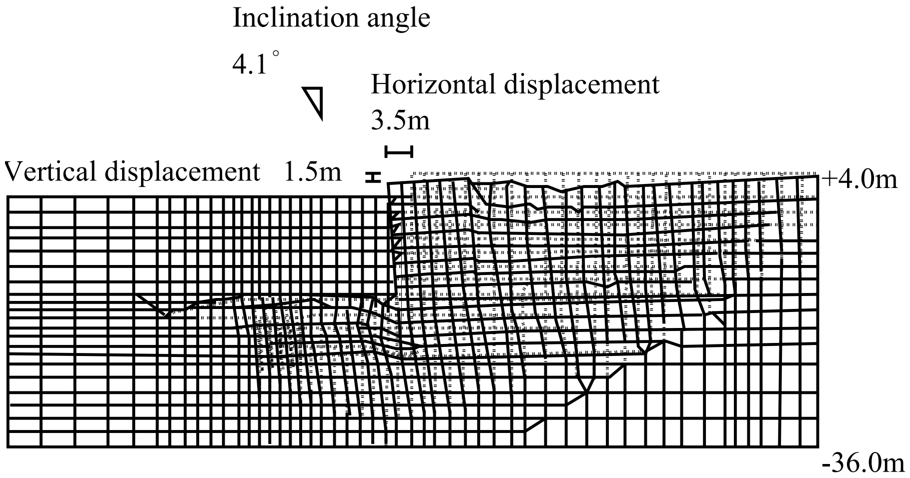


Fig. 2.2.8 Finite Element Model (Gravity-type Quaywall)

4) Individual element model

In the individual element modeling method, soil and respective structures such as wave dissipating blocks are individually modeled as granular objects, and deformation analyses are performed by calculating mutual interaction as a result of the contact among granular objects.[26] This modeling method is particularly suitable for analyzing the deformation associated with the rotation of wave dissipating blocks. There has been a proposal for a hybrid analysis method that combines the individual element method and the finite element method.[27]

(d) Modeling of material characteristics

The modeling of the nonlinear characteristics of soil material constituting the ground is important in the execution of earthquake response analyses. A mathematical model representing behavioral characteristics, such as the stress–strain relation of soil, is called a constitutive law. When shear strain during the action of earthquake ground motions is in a relatively low level, soil shows a linear stress–strain relation; however, when the shear strain is in an intermediate or high level, soil shows significantly nonlinear stress–strain relation. Therefore, depending on the levels of shear strain, it is necessary to use a constitutive law that is capable of dealing with nonlinearity in deformation verification.

Several earthquake response analysis methods have been proposed: a linear analysis method that does not consider the nonlinearity of the materials constituting the ground, an equivalent linear analysis method that performs linear analyses by using material constants depending on the strain levels that the ground receives, and a nonlinear response analysis method that considers the stress–strain relation of the soil subjected to large strain. However, in consideration of the purpose of the deformation verification to examine residual deformation, there are cases wherein linear and equivalent linear analysis methods are not always appropriate and wherein nonlinear response analysis methods based on constitutive laws and capable of dealing with nonlinearity are required.

(e) Classification by calculation domain

From the viewpoint of the calculation domain, the earthquake response analyses can be classified into time domain analysis method and frequency domain analysis method. The effective stress analysis method and calculations with nonlinear material characteristics are generally performed sequentially in the time domain.

② **Methods employing shaking tests**

The methods employing shaking tests, which apply vibrations to structures including the ground by taking into consideration mechanical similarities, are effective in assessing the overall behavior of the structures. However, these methods require high levels of experimental techniques for preparing models that adequately satisfy the condition of similarities. The shaking tests using a shaking table are classified as follows.

(a) Model shaking test in a 1G gravity field

In the model shaking test in a 1G gravity field, models are prepared in a manner that satisfies the similarity ratios by taking into consideration the shapes and mechanical characteristics of the target structures and ground. Assumed earthquake ground motions are applied to the models using a shaking table. The model shaking test generally enables large models to be prepared and is applicable to cases with complex ground and structural configurations. Furthermore, the similarity law considering the dependency of the physical property of soil on confining pressure is normally applied to the model shaking test.[28)]

(b) Model shaking test using a centrifugal loading device

In the test, the assumed earthquake ground motions generated by a shaking test device is applied to models that satisfy the similarity law, and stress states similar to actual situations are reproduced in them by the centrifugal force generated by a centrifugal loading device. The test generally requires models to be small in scale but enables the models to be tested on the basis of the dependency of the physical property of soil on confining pressure without assuming the relation between the physical property of soil and effective confining pressure. However, the test requires attention to the use of the coefficient of permeability conforming to the similarity law and the influences of the particle sizes of ground materials used in the test on test results.

(c) In-situ shaking test

In this type of test, models that are similar to or in substantially the same scale as target structures are prepared either at the location where construction is planned or under similar ground conditions. The responses of the models to artificial ground motions or natural ground motions are then observed. The methods of generating artificial ground motions include methods employing wave vibrators and blasting.[29)]

Although model and in-situ shaking tests are effective tests, they cannot accurately reproduce actual boundary conditions. For example, in the model shaking tests, models are subjected to the input of earthquake motions in rigid ground without the attenuation effect of their downward scattering; therefore, the test results are likely to be strongly influenced by the natural frequency of the ground–structure systems. Furthermore, in-situ shaking tests using blasting cannot reflect the effect of inertia force due to earthquake ground motions in the test results.

2.2.5 Performance Verification of Cellular Blocks

(1) Unlike other gravity-type quaywalls, gravity-type quaywalls that comprise cellular blocks with no bottom slabs have structures that maintain integrity with wall bodies through fillings. Therefore, in addition to the examination of stability similar to the case in other gravity-type quaywalls, overturning should be examined, with due consideration given to the extrusion of the fillings.

(2) Equation for Verifying the Stability of Cellular Blocks against Overturning

The verification of the stability of cellular blocks against overturning in considering of the extrusion of fillings can be performed using **equation (2.2.9)**. In this equation, subscripts *k* and *d* indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.2.6** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$
\begin{aligned}
& m\cdot\frac{S_d}{R_d}\leq 1.0 \quad R_d=\gamma_R R_k \quad S_d=\gamma_S S_k \\
& R_k=(aW_k-bP_{B_k}+cP_{V_k}+M_{fk}) \\
& S_k=dP_{H_k}+eP_{w_k}+hP_{dw_k}+iP_{F_k}
\end{aligned}
\qquad \textbf{(2.2.9)}
$$

where

W : weight of materials comprising a wall body (kN/m);

P_B : buoyancy acting on a wall body (kN/m);

P_V : resultant vertical earth pressure acting on a wall body (kN/m);

M : resistant moment due to friction on wall surfaces with fillings (kN·m/m);

P_H : resultant horizontal earth pressure acting on a wall body (kN/m);

P_w : resultant residual water pressure acting on a wall body (kN/m);

P_{dw} : resultant dynamic water pressure acting on a wall body (kN/m) (only during earthquakes);

P_F : inertia force acting on a wall body (kN/m) (only during earthquakes);

a : distance from the action line of resultant weight of wall to the front toe of a wall body (m);

b : distance from the action line of buoyancy to the front toe of a wall body (m);

c : distance from the action line of resultant vertical earth pressure to the front toe of a wall body (m);

d : distance from the action line of resultant horizontal earth pressure to the bottom of a wall body (m);

e : distance from the action line of resultant residual water pressure to the bottom of a wall body (m);

h : distance from the action line of resultant dynamic water pressure to the bottom of a wall body (m) (only during earthquakes);

i : distance from the action line of inertial force to the bottom of a wall body (m) (only during earthquakes);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.2.6 Partial Factors Used for the Performance Verification of the Overturning of Wall Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of the cellular block with the extrusion of fillings (Permanent state)	– (1.00)	– (1.00)	1.20
Overturning of the cellular block with the extrusion of fillings (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.10

Furthermore, when a caisson has footings with rectangular cross sections on both the sea and land sides, the characteristic value of buoyancy can be calculated using **equation (2.2.6)**.

(3) When $S_d > R_d$ in **equation (2.2.9)**, the overturning moment due to the action becomes larger than the resistant moment generated by the total vertical force, excluding fillings and the friction on wall surfaces with fillings, thus causing a cellular block to overturn with fillings left behind. In such a case, it is necessary to increase the weight of the cellular block or to provide the cellular block with partition walls.

(4) The characteristic value M_{fk} of the resistant moment generated by the friction force F_1 and F_2 on wall surfaces with fillings can be calculated as follows. The moment around Point A in **Fig. 2.2.9** can be expressed by $l_1F_1 + l_2F_2$ where $F_1 = P_1f$ and $F_2 = P_2f$, and f is a coefficient of friction between a filling material and a wall surface (P_1 and P_2 are the earth pressure of the fillings). For the concept of the earth pressure of fillings acting on wall surfaces, refer to **Part III, Chapter 2, 2.4 Cellular Blocks**. It is desirable to consider the friction resistance generated on the partition walls of a cellular block, in addition to the resistant moment.

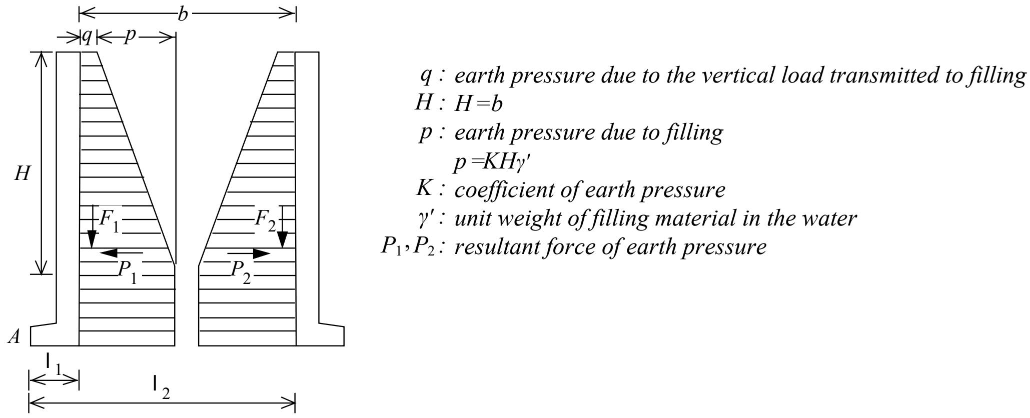


Fig. 2.2.9 Method for Obtaining Wall Surface Friction Resistance

(5) Regarding the characteristic values of the friction coefficient to be used for the performance verification of the sliding of cellular concrete blocks with no bottom slabs, 0.6 and 0.8 shall be used as the reaction force received by the bottoms of reinforced concrete sections and filling stone sections, respectively. However, for convenience, 0.7 can be used for both characteristic values.

2.2.6 Performance Verification of Structural Members

(1) For the performance verification of structural members such as caissons, cellular blocks, L-shaped blocks, refer to **Part III, Chapter 2, 2 Structural Members**. For block-type quaywalls, the blocks that constitute quaywalls should have sufficient strength because they are main sections of wall bodies. For the performance verification of blocks, refer to **Part III, Chapter 2, 2 Structural members**.

(2) The stability of the portion of a superstructure where a mooring post is installed should be examined in a manner that allows the weight of a certain concrete mass of the superstructure to resist mooring force together with the mooring post. In cases wherein the weight of a large concrete mass of a superstructure is required to ensure the stability of a mooring post, the mass shall be reinforced with rebars. In other cases wherein the stability of a mooring post cannot be achieved only by the weight of a superstructure and requires the superstructure to be connected to the main body of a quaywall through rebars, it is necessary that the action allowing the mooring force to be transferred from the superstructure to the main body via the rebars shall be considered in examining the stability of the mooring post.

(3) The performance verification of the portion of a superstructure where a fender is installed can be performed in a manner that focuses only on a certain concrete mass whose weight integrally contributes to resisting fender reaction. In cases wherein a fender is installed at the portion of a superstructure that is connected to a wall body via rebars as reinforcement to support a mooring post, the displacement of the superstructure in a direction that allows passive earth pressure to effectively resist fender reaction cannot be expected; therefore, it is desirable that fender reaction is completely borne by the rebars. In the performance verification of the cross section of a superstructure, fender reaction is assumed to be distributed as a linear load in the range of width b (**Fig. 2.2.10 (a)**) and may be considered to act as shown in **Fig. 2.2.10 (b)**. In many cases, the performance verification of the cross section of the superstructure in the vertical direction is performed by assuming a cantilever beam with the bottom edge of the superstructure as a fulcrum and that in the horizontal direction is performed by assuming either a continuous beam or a simple beam with rigid points in the wall body as fulcrums.

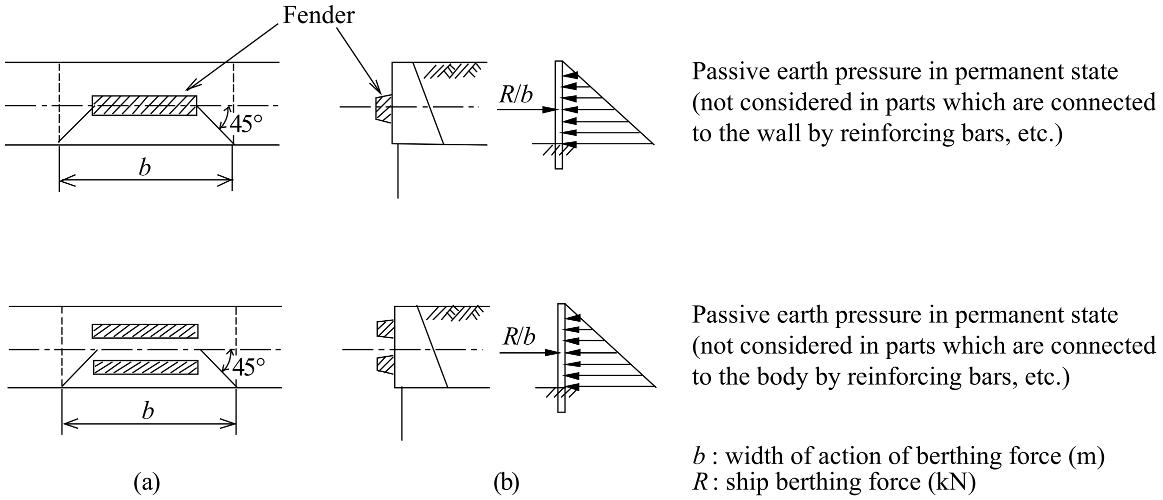


Fig. 2.2.10 Fender Reaction Acting on Superstructures

2.2.7 Structural Details

(1) In cases of gravity-type quaywalls provided with high-quality backfill, the following effects can be expected, in addition to the earth pressure reduction effect:

① Lowering of residual water levels as a result of the increase in permeability

② Protection against backfill soil from being washed out

(2) The fluctuation of residual water levels may cause backfill soil to infiltrate the gaps among backfill materials and cause the settlement of the base courses of apron pavement. Therefore, it is necessary that quaywalls are provided with measures to fill the gaps at the rear of backfilling or sand prevention sheets.

(3) In areas wherein large tidal ranges in front of quaywalls cause severe problems with residual water levels, the shapes of backfill shall be carefully studied so that the residual water levels can be effectively reduced.

(4) In cases wherein land settlement or specific site conditions may cause backfill materials to be washed out, protective measures such as sand washing-out prevention joint plates should be installed on the gaps between the rear faces of wall bodies.

(5) In many cases, the thickness of the cover concrete of caisson-type quaywalls is 20 to 30 cm; however, there may be a case that requires special measures against wave actions, similar to the case with breakwaters depending on construction conditions.

(6) Backfill soil can seep through the gaps between the blocks or rubble stones of gravity-type structures. Therefore, such structures should be provided with appropriate measures, such as sand invasion prevention sheets or plates, to prevent soil from being washed out.

(7) Blocks should be provided with tenons or rebars between them to enhance their integrity as wall bodies via increased interlocking effect. For the methods for interlocking blocks, reference can be made to **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

(8) The portions of the superstructures of quaywalls where ancillary facilities are installed should have appropriate shapes that are suitable for the facilities.

(9) The ancillary facilities generally installed on quaywalls are as follows. For the performance verification of the ancillary facilities, refer to **Part III, Chapter 5, 9 Ancillary Facilities of Mooring Facilities**.

① Fenders
② Mooring posts
③ Curbing
④ Water supply and drainage facilities
⑤ Stairs and ladders
⑥ Others

It is preferable that the joint intervals and the strength of superstructures be examined by taking into consideration the force applied to mooring posts and fenders.

2.3 Sheet Pile Quaywalls

[Public Notice] (Performance Criteria for Sheet Pile Quaywalls)

Article 50

1 The performance criteria for sheet pile quaywalls shall be as prescribed respectively in the following items:

(1) Sheet piles shall have the embedment length necessary for the structural stability and shall contain the degree of risk indicating that the stresses in the sheet piles may exceed the yield stress at the level equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(2) The following criteria shall be satisfied under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating actions are Level 1 earthquake ground motion and traction by ships:

(a) For anchored structures, the anchorage shall be located appropriately in consideration of the structural type, and the risk of losing the structural stability shall be equal to or less than the threshold level.

(b) For structures with ties and waling, the risk that the stresses in the ties and waling may exceed the yield stress shall be equal to or less than the threshold level.

(c) For structures with superstructures, the risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

(3) For structures with superstructures, the risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level under the variable situation, in which the dominating action is ship berthing.

(4) Under the permanent situation, in which the dominating action is self-weight, the risk of occurrence of slip failure in the ground below the bottom end of the sheet pile shall be equal to or less than the threshold level.

2 In addition to the provisions in the preceding paragraph, the performance criteria for cantilevered sheet piles shall indicate that the risk in which the amount of deformation of the top of the pile may exceed the allowable limit of deformation is equal to or less than the threshold level under the permanent situation, in which the dominant action is earth pressure, and under the variable situation, in which the dominant actions are Level 1 earthquake ground motion, ship berthing, and traction by ships.

3 In addition to the provisions in the paragraph (1), the performance criteria for double sheet pile structures shall be as prescribed respectively in the following items:

(1) The risk of occurrence of sliding of the structural body shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(2) The risk that the deformation of the top of the front or rear sheet pile may exceed the allowable limit of deformation shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(3) The risk of losing the stability due to the shear deformation of the structural body shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure.

[Interpretation]

11. Mooring Facilities

(4) Performance Criteria of Sheet Pile Quaywalls

① **Sheet pile quaywalls** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance on Criteria and the interpretation related to Article 50, Paragraph 1of the Public Notice)

(a) The required performance of sheet pile quaywalls under a permanent state in which the dominant action is earth pressure and a variable state in which the dominant actions are Level 1 earthquake ground motions shall be serviceability. The performance verification items and standard indexes for determining the limit values with respect to the actions shall be shown in **Attached Table 11-6** provided that those having structures comprising anchorages, those having structures comprising ties and waling, and those having copings shall comply with the provisions in (b), (c), and (d), below respectively.

Attached Table 11-6 Performance Verification Items and Standard Indexes for Determining Limit Values under the Respective Design Situations of Sheet Pile Quaywalls

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index for determining the limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="4">26</td><td rowspan="4">1</td><td rowspan="4">2</td><td rowspan="4">50</td><td rowspan="4">–</td><td rowspan="4">1</td><td rowspan="4">Serviceability</td><td rowspan="2">Permanent</td><td rowspan="2">Earth pressure</td><td rowspan="2">Water pressure, surcharges</td><td>Necessary embedded length</td><td>Embedded length required for structural stability</td></tr>
<tr><td>Yielding of the sheet pile</td><td>Design yield stress of sheet pile</td></tr>
<tr><td rowspan="2">Variable</td><td rowspan="2">L1 earthquake ground motion</td><td rowspan="2">Earth pressure, water pressure, surcharges</td><td>Necessary embedded length</td><td>Embedded length required for structural stability</td></tr>
<tr><td>Yielding of the sheet pile</td><td>Design yield stress of sheet pile</td></tr>
</table>

(b) For the permanent state in which the dominant action is earth pressure and the variable state in which the dominant actions are Level 1 earthquake ground motions and traction by ships, the performance verification items and standard indexes for determining the limit values with respect to anchorages shall be those shown in **Attached Table 11-7**.

Attached Table 11-7 Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to Anchorages under the Respective Design Situations of Sheet Pile Quaywalls

Ministerial Ordinance			Public Notice			Performance requirements	Design sate			Verification item	Standard index for determining the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	50	-	2a	Serviceability	Permanent	Earth pressure	Water pressure, surcharges	Necessary embedded length	Embedded length required for structural stability
										Yielding of the anchorage*1)	Design yield stress
										Axial force on the anchorage*2)	Action–resistance ratio with respect to the bearing force of an anchorage (press force and pullout force)
										Stability of the anchor wall*3)	Design cross-section resistance Passive earth pressure on the front face of anchor plate
							Variable	L1 earthquake ground motion [traction force of ships]	Earth pressure, water pressure, surcharges	Necessary embedded length	Embedded length required for structural stability
										Yielding of the anchorage*1)	Design yield stress
										Axial forces in the anchorage*2)	Action–resistance ratio with respect to the bearing force of an anchorage (press force and pullout force)
										Stability of the anchor wall*3)	Design cross-section resistance Passive earth pressure on the front face of the anchor plate

* [   ] indicates an alternative dominant action to be studied as design situations.
*1): Only when the structural type of the anchorage is a vertical pile anchor, a coupled-pile anchor, or sheet pile anchor
*2): Only when the structural type of the anchorage is a coupled-pile anchor
*3): Only when the structural type of the anchorage is a slab anchor

(c) For the permanent state in which the dominant action is earth pressure and the variable state in which the dominant actions are Level 1 earthquake ground motions and traction by ships, the performance verification items and the standard indexes for determining the limit values with respect to ties and waling shall be those shown in **Attached Table 11-8**.

Attached Table 11-8 Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to Ties and Waling under the Respective Design Situations of Sheet Pile Quaywalls

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for determining the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	50	–	2b	Serviceability	Permanent	Earth pressure	Water pressure, surcharges	Yielding of the tie	Design yield stress
										Yielding of the waling	
							Variable	L1 earthquake ground motion [traction force of ships]	Earth pressure, water pressure, surcharges	Yielding of the tie	
										Yielding of the waling	

* [ ] indicates an alternative dominant action to be studied as design situations.

(d) For the permanent state in which the dominant action is earth pressure and the variable state in which the dominant actions are Level 1 earthquake ground motions and traction by ships, the performance verification items and standard indexes for determining the limit values with respect to the copings of sheet pile quaywalls shall be those shown in **Attached Table 11-9**.

Attached Table 11-9 Performance Verification Items and Standard Indexes for Determining the Limit Values with respect to the Copings of Sheet Pile Quaywalls under their Respective Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for determining the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	50	–	2c	Serviceability	Permanent	Earth pressure	Surcharges	Cross-section stress of the coping	Bending compression stress
							Variable	L1 earthquake ground motion [traction force of ships] [berthing force of ships]	Earth pressure, surcharges	Failure of the cross section of the coping	Design cross-section resistance

* [ ] indicates an alternative dominant action to be studied as design situations.

(e) For the permanent situation in which the dominant action is the self-weight of sheet pile quaywalls, the performance verification items and standard indexes to determine the limit values of sheet pile quaywalls shall be those shown in **Attached Table 11-10**.

Attached Table 11-10 Performance Verification Items and Standard Indexes to Determine the Limit Values under the Permanent Situation in which the Dominant Action Is the Self-weight of Sheet Pile Quaywalls

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	50	–	4	Serviceability	Permanent	Self-weight	Water pressure, surcharge	Circular slip failure of the ground	Action–resistance ratio with respect to circular slip

(f) In the cases of using sheet piles with special joints or large-scale joints, the performance verification items and standard indexes to determine the limit values with respect to the stress on the joints shall be appropriately set as needed.

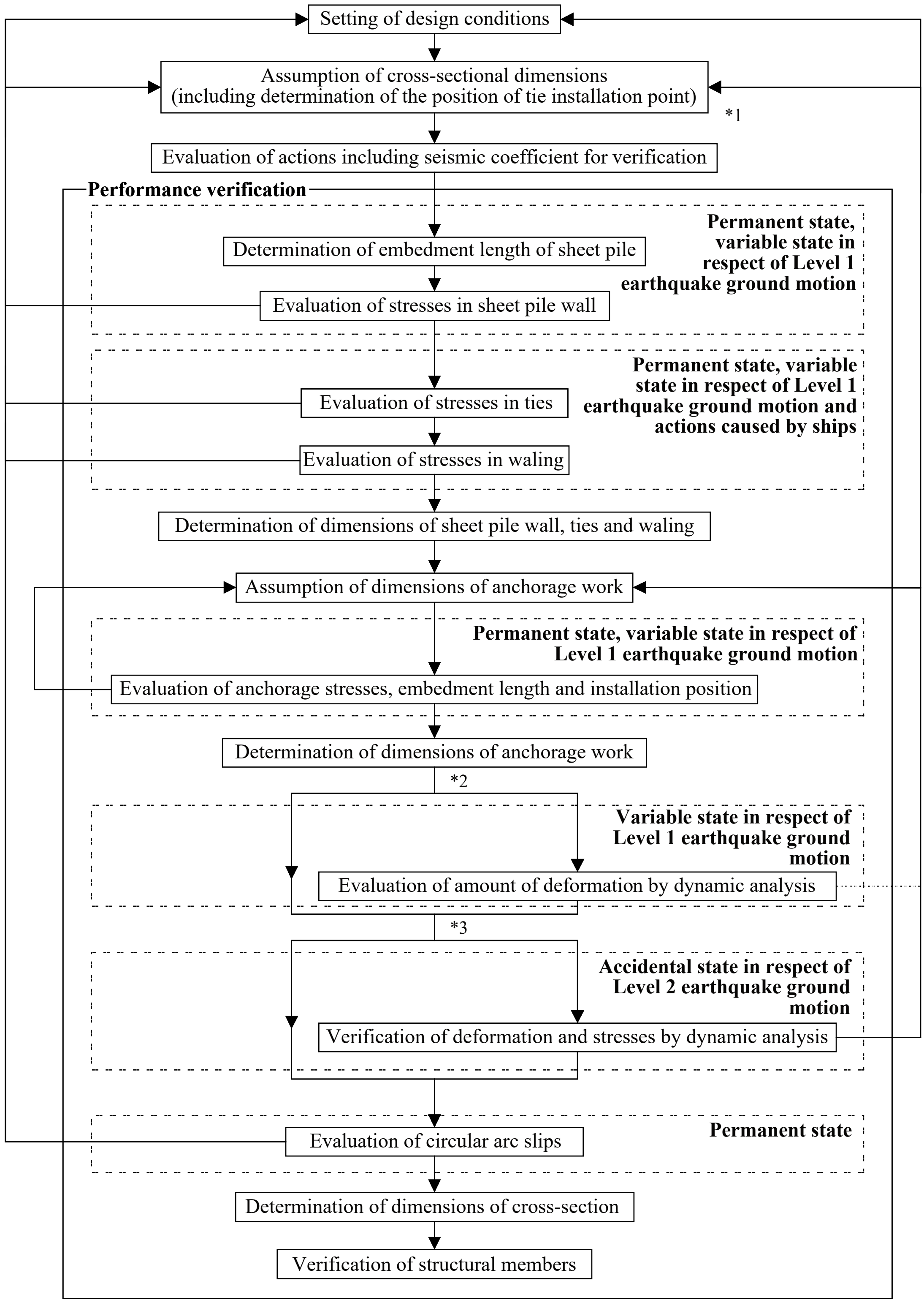
(g) In addition to the provisions in this code, sheet pile quaywalls shall comply with the provisions and commentaries in Paragraph 3 (Scouring and Wash Out), Article 22 and Article 28 (Performance Criteria of Armor Stones and Blocks) of the Standard Public Notice as needed.

2.3.1 General

(1) The provisions in this section can be applied to the performance verification of steel sheet pile quaywalls with anchorages.

(2) **Fig. 2.3.1** shows an example of the sequence of the performance verification of sheet pile quaywalls. However, **Fig. 2.3.1** does not show the evaluation of the effects of the liquefaction and settlement due to earthquakes. Therefore, it is necessary to appropriately deliberate the possibility of and countermeasures against liquefaction with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the variable state with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method on the basis of a static equation of equilibrium. However, for high earthquake-resistance facilities, it is advisable to deliberate deformation by using nonlinear seismic response analysis or other methods that take into consideration the dynamic interaction between the ground and structures. For sheet pile quaywalls other than high earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.

(3) **Fig. 2.3.2** shows an example of the cross section of the sheet pile quaywalls.



*1: The evaluation of liquefaction and settlement are not shown; therefore, it is necessary to consider these separately.

*2: When necessary, an evaluation of the amount of deformation by dynamic analysis can be performed for the Level 1 earthquake ground motion.
For high earthquake-resistance facilities, it is preferable that an examination of the amount of deformation be performed by dynamic analysis.

*3: Verification with respect to Level 2 earthquake ground motion is performed for high earthquake-resistance facilities.

Fig. 2.3.1 Example of the Sequence of the Performance Verification of Sheet Pile Quaywalls

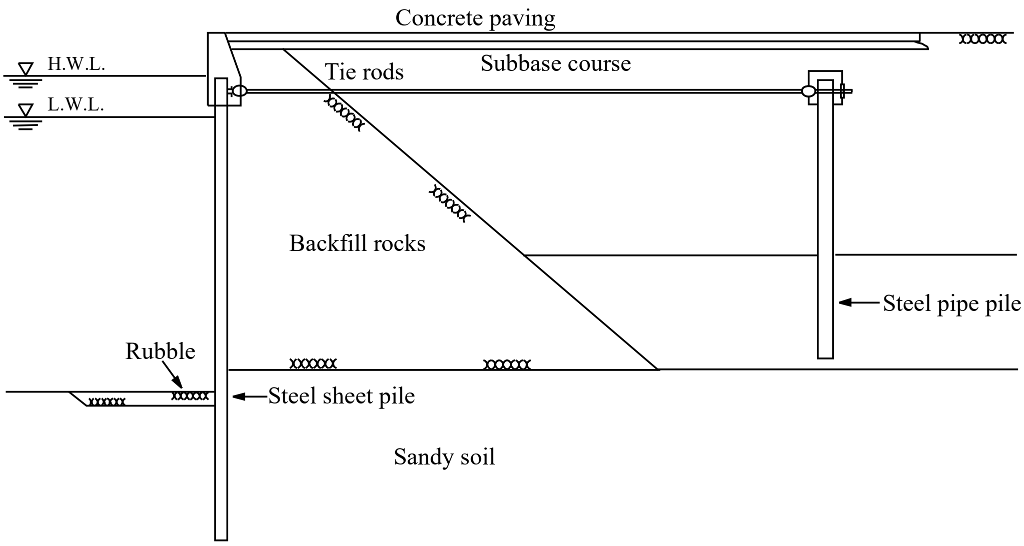


Fig. 2.3.2 Example of the Cross Section of the Sheet Pile Quaywall

2.3.2 Points of Caution When Installing Sheet Pile Quaywalls on Soft Ground

(1) The performance verification of a sheet pile wall on soft ground, such as alluvial cohesive soil on soft seabed, should preferably be conducted via comprehensive examination by using the performance verification methods shown in this section for ties and anchorages and other performance verification methods. An unexpected large deformation may occur in sheet piles constructed on soft ground owing to lateral flows that are caused by the settlement of the ground behind the sheet pile wall. Several methods for lateral flow prediction[30)] have been proposed. These effects should be taken into consideration when conducting the performance verifications.

(2) Care should be exercised in using the performance verification methods for the sheet pile quaywalls described in this section because many of these methods assume that a steel sheet pile wall is driven mainly into sandy soil ground or hard clayey soil ground. For soft ground, it is preferable to perform soil improvement work. When it is not possible to perform soil improvement work because of site conditions, it is preferable to consider using other performance verification methods, in addition to the methods described in this section, such as numerical analysis methods that can accurately evaluate the nonlinear characteristics of soil, so that a comprehensive analysis can be made.

(3) When deliberating the embedded lengths of sheet piles, the deflection curve method,[31)] which is a type of fixed earth support method based on the classical earth pressure theory that deals with the sheet piles with deep embedded lengths, can be used in addition to the method introduced in this section. The deflection curve method obtains an embedded length by solving an equation under the following conditions: the displacement and deflection angle at the lower end of embedment are zero; the displacement at a tie member installation point is zero and is under the loading conditions shown in **Fig. 2.3.3**. The deflection curve method is also applicable to soft ground.

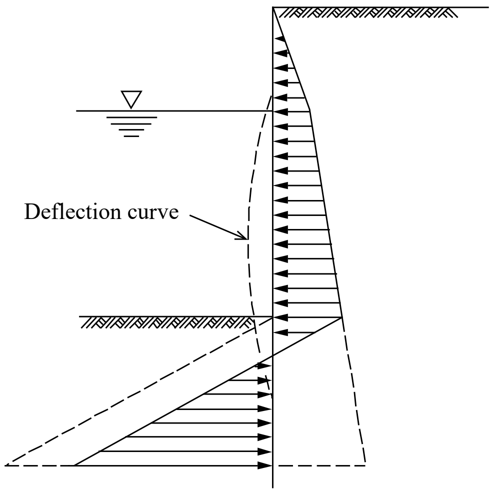


Fig. 2.3.3 Earth Pressure and Deflection Curve

(4) It is advisable to comprehensively deliberate the bending moment in sheet piles and the tensile force in tie members by using the method for bending moment and tie member installation point reaction force, which is explained in this section, and the deflection curve method mentioned above.

(5) Generally, for cohesive soil ground, the stability of embedment cannot be achieved unless **equation (2.3.1)** is satisfied. In this equation, subscript *k* refers to a characteristic value.

$$4c_k > q_k + \sum w_{i_k} + \rho_w g h_w \qquad \textbf{(2.3.1)}$$

where

c : adhesion of sea-bottom soil (kN/m^2);

q : loaded weight (kN/m^2);

w_i : weight of the *i*th soil layer above the seafloor surface or underwater weight of a soil layer if it is below a residual water level (kN/m^2);

ρ_w : density of seawater (t/m^3);

g : gravitational acceleration (m/s^2);

h_w : difference between a residual water level and a tidal level in front of a quaywall (m).

When soft seabed does not satisfy **equation (2.3.1)**, the seabed needs to be improved by an appropriate method or a sheet pile wall with a relieving platform that needs to be used as a countermeasure.

2.3.3 Setting of Cross-Sectional Dimensions

(1) Installation Positions of Tie Members

① Tie member is a collective term of materials such as tie rods and tie wires connecting sheet piles and anchorages.

② The cross sections of sheet piles and tie members will be largely influenced by the positions of the tie member installation. The positions of tie member installation should be determined by considering the difficulty of the work of tie member attachments and the costs.

③ The bending moment in sheet piles has a tendency to be reduced as the positions for the installation of tie members become lower. Generally, the bending moment is reduced by installing tie members at positions that are approximately half the heights of sheet pile walls. Thus, the cross-sectional areas of sheet piles can be reduced by lowering the tie member installation positions, and the embedded lengths of sheet pile can be reduced accordingly. By contrast, the tensile force acting on tie members has a tendency to increase as the positions to install tie members become lower, thereby increasing the cross-sectional areas of tie members and the sizes of anchorages. Therefore, it is advisable to decide the tie member installation positions to minimize construction costs by balancing the cost reduction and increase the effects above. Generally, construction costs are reduced as the tie member installation positions become lower. However, if the original ground levels before starting construction are high, decreasing the tie member installation positions may increase the construction costs because of increased excavation and backfilling costs.

④ When the wall height of a sheet pile wall is large, tie members may be provided at two levels to support the wall structure at two points to reduce the bending moments in the sheet pile.

⑤ The tie member installation position is generally set at approximately 2/3 of a tidal difference above the lowest water level (LWL).

(2) Selection of the Structural Types of Anchorages

① The structural types of anchorages are generally broadly classified as vertical pile anchorage, coupled-pile anchorage, sheet pile anchorage, and slab anchorage. The economy, construction time, and construction method differ depending on the structural types; therefore, it is necessary to determine the structural types by considering the elevation of the ground before construction and other site conditions.

② In the case where the ground in front of anchorages is saturated sandy soil and subjected to liquefaction owing to earthquake ground motions, anchorages with shallow embedment are likely to be affected by liquefaction because liquefaction occurs close to ground surfaces. Therefore, coupled-pile anchorages with deeper embedment are preferable in areas with such ground conditions. Refer to **Part II, Chapter 7 Ground Liquefaction** when studying liquefaction due to earthquake ground motions.

③ Generally, the displacement of anchorages when tie members are subjected to tensile force is smaller in the case of coupled-pile anchorage and larger in the case of sheet pile anchorage or vertical pile anchorage. Furthermore, the displacement due to earthquake ground motions is particularly increased in the case of anchorages using sheet piles and vertical piles.

④ Vertical pile anchorages or coupled-pile anchorages are generally preferable in the case where original seafloor surfaces are deep before starting construction.

⑤ Generally, a coupled-pile anchorage structure is preferable in the case where facilities at the back of quaywalls impose the constraints on the installation locations of anchorages.

⑥ It is necessary to pay attention to possible bending stress in structures, in addition to axial force in the case of coupled-pile anchorages installed in areas subjected to the settlement of backfill soil.

⑦ Whether concrete work can be executed in a dry condition is a criterion for determining the availability of the slab anchorages of sheet pile walls. In most cases, the slab anchorage for relatively large-scale sheet pile walls requires construction below groundwater level involving the temporary closing and drainage of water with pumps. For small-scale concrete walls, concrete structures that were prefabricated at factories can be transported and installed by cranes at sites similar to the case with concrete walls of dead man anchors.

⑧ The construction of sheet pile anchorages is easy and can be executed in a short period because the construction method is identical to that for sheet pile walls. The sheet pile anchorages are particularly preferable in the case where the ground levels at the back of quaywalls are high enough to enable steel sheet piles to be driven onshore.

(3) Installation Locations of Anchorages

① In principle, the location of the anchorages need to be set at an appropriate distance from the sheet pile wall to ensure the structural stability of the main body of the wall and anchorage depending on the characteristics of the anchorages. Normally, when the position of the installation of the anchorage is further from the surface of the sheet pile wall, the deformation restraint of the sheet pile wall during an earthquake will be more effective.[32)] By contrast, the cross-sectional force on the sheet pile walls increases as the level of constraint on deflection increases. The following method for setting the locations of anchorages has been used in many cases, but the application of the method shall be comprehensively determined after considering the relations explained above.

② The location of an anchorage should be determined appropriately in consideration of the structural type of the anchorage because the stability of the anchorage itself is affected by its position, and the location at which the stability is achieved varies depending on the structural type. Furthermore, the location of anchorages on soft ground shall be determined after the comprehensive deliberation of the behavior of sheet piles, tie members, and anchorages on the occurrence of earthquakes by using the method explained in this section or by using a dynamic analysis method that considers the nonlinear characteristics of the ground.

③ The location of a vertical pile anchorage is preferably determined to ensure that the passive failure plane from the point of $l_{m1}/3$ below the tie member installation point of the anchorage, and the active failure plane from the intersection of the sea bottom and sheet piles do not intersect at the level below the horizontal surface containing the tie member installation point at the anchorage (**Fig. 2.3.4**). The value of l_{m1} is the depth of the first zero point of the bending moment for a free-head pile below the tie member installation point, whereas the horizontal surface containing the installation point of the tie member at the anchorage is assumed as the ground surface.

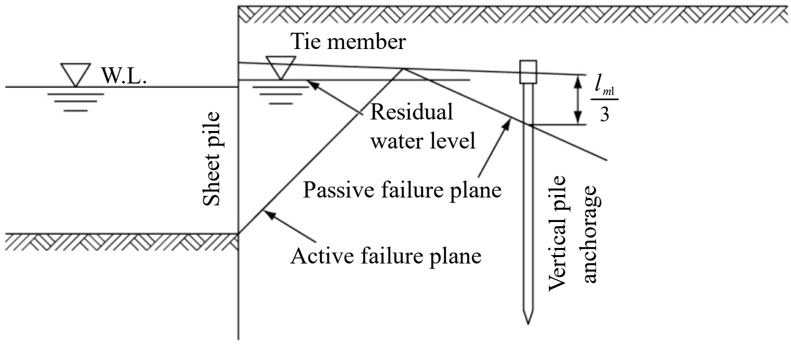


Fig. 2.3.4 Location of Vertical Pile Anchorage

④ The method for determining the location of vertical pile anchorage explained in ③ above is based on the model experiment result by Kubo et al.[33)] However, the location obtained through the method is the calculated limit distance that enables the vertical pile anchorage to obtain the predetermined resistance, and the experiment was conducted under conditions that are different from actual ones. Therefore, it is preferable to determine the location of the vertical pile anchorage so that the passive failure plane of a pile drawn from the point $l_{m1}/3$ below the tie member installation point on the pile anchorage and the active failure plane of a sheet pile drawn from the sea bottom intersect with each other at the ground surface.

⑤ The location of a coupled-pile anchorage should be behind the active failure plane of the sheet pile wall drawn from the sea bottom when it is assumed that the tension of the tie member is resisted only by the axial bearing capacity of the piles (**Fig. 2.3.5**). When the tension of the tie member is resisted by both the axial and lateral bearing capacity in consideration of the bending resistance of the piles, it is necessary to locate the anchorage in accordance with the location of the vertical pile.

⑥ Refer to **Part II, Chapter 4, 2 Earth Pressure** for the angle between an active failure plane and a horizontal plane.

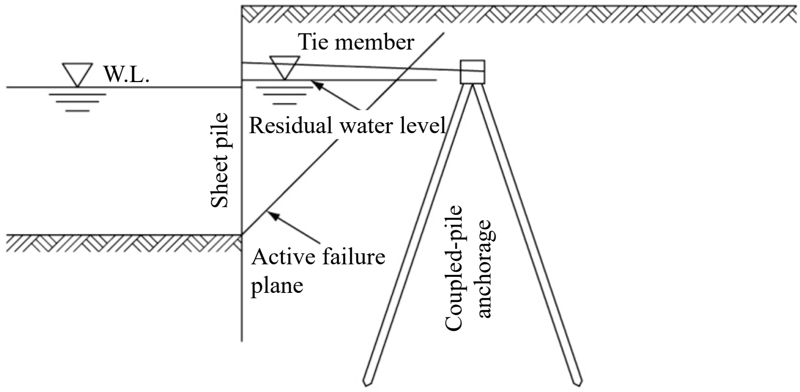


Fig. 2.3.5 Position of Coupled-Pile Anchorage

⑦ The location of a sheet pile anchorage may be determined in accordance with the location of a vertical pile when the sheet piles can be regarded as a long pile. When the sheet piles cannot be regarded as a long pile, the location of the anchorage may be determined by ignoring a part that is deeper than the level $l_{m1}/2$ below the tie member installation point at the sheet pile anchorage and by applying the location determination method for slab anchorage.

⑧ For the method to obtain the first zero point of the bending moment of the vertical pile anchorage and sheet pile anchorage and the method to determine whether a sheet pile anchorage can be considered a long pile, refer to the Port and Harbour Research Institute's method described in **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**.

⑨ For the ordinary sheet pile quaywalls with tie members that run horizontally, an angle of -15° may be used as the wall friction angle in the determination of the passive failure plane that is drawn from the vertical pile anchorage or sheet pile anchorage.

⑩ The location of the slab anchorage is preferably determined to ensure that the active failure plane starting from the intersection of the sea bottom and sheet pile wall and the passive failure plane of the slab anchorage drawn from the bottom of the anchorage do not intersect below the ground surface (**Fig. 2.3.6**).

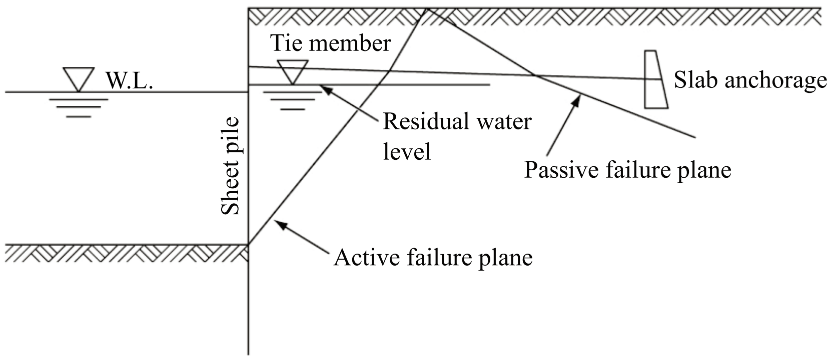


Fig. 2.3.6 Location of Slab Anchorages

2.3.4 Actions

(1) Types of Actions to Be Considered in Respective Design Situations

The stability verification of sheet pile quaywalls shall consider the following actions in respective design situations. However, the performance verification for an accidental situation can be omitted in the case where the sheet pile quaywalls to be designed are not high earthquake-resistance facilities.

① **Permanent state**

The dominant actions shall be the earth pressure acting on wall bodies and self-weight. Refer to **Part II, Chapter 4, 2 Earth Pressure** for the earth pressure.

② **Variable state**

The dominant actions shall be Level 1 earthquake ground motions and the traction force of ships. For the setting of Level 1 earthquake ground motions, refer to **Part II, Chapter 6, 1.2 Level 1 Earthquake Ground Motions Used for the Performance Verification of Facilities**. For the setting of traction force of ships, refer to **Part II, Chapter 8, 2.3 Actions due to the Oscillation of Ships** and **Part II, Chapter 8, 2.4 Actions due to Traction by Ships**.

③ **Accidental situation**

The dominant actions shall be Level 2 earthquake ground motions. For the setting of Level 2 earthquake ground motions, refer to **Part II, Chapter 6, 1.3 Level 2 Earthquake Ground Motions Used for the Performance Verification of Facilities**.

(2) Points of Caution When Setting Actions

① The active earth pressure is normally used as the earth pressure that acts on the sheet pile wall from the backside. For the front-side reaction that acts on the embedded part of the sheet pile, it is necessary to use an appropriate value such as passive earth pressure or a subgrade reaction that corresponds to modulus of subgrade reaction.

② When the free earth support method and the equivalent beam method described in this section are used in the performance verification for a sheet pile wall, the earth pressure and residual water pressure should be assumed to act as those shown in **Fig. 2.3.7**, and the pressure values can be calculated in accordance with **Part II, Chapter 4, 2 Earth Pressure** and **Part II, Chapter 4, 3.1 Residual Water Pressure**. The wall friction angle used for the calculation of the earth pressure acting on the sheet pile wall may usually be taken at 15° for the active earth pressure and -15° for the passive earth pressure when the ground is sandy soil layer.

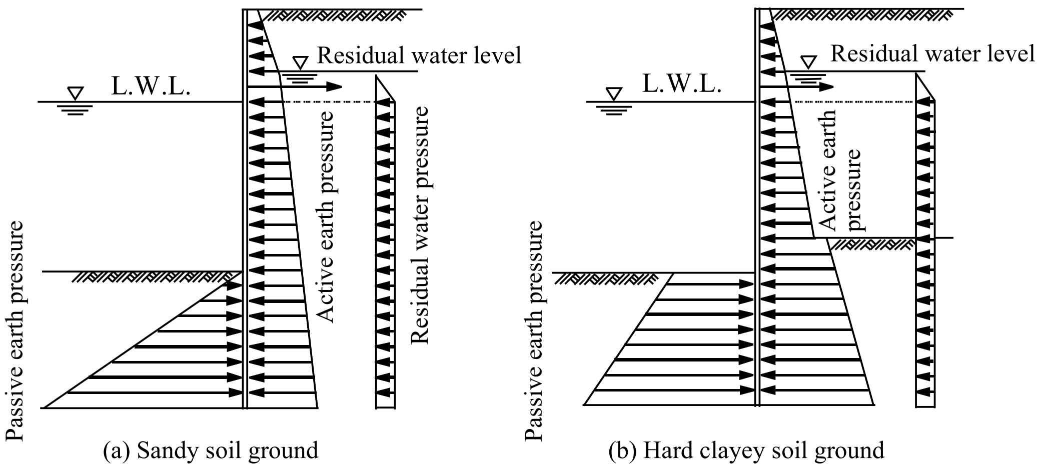


Fig. 2.3.7 Earth Pressure and Residual Water Pressure to Be Considered for the Performance Verification of Sheet Pile Walls

③ Considering that the earth pressure changes in response to the displacement of the sheet pile wall, the actual earth pressure that acts on the sheet pile wall varies depending on the following:

(i) The construction method, i.e., whether backfill is executed or the ground in front of the sheet piles is dredged to the required depth after the sheet piles have been driven in

(ii) The lateral displacement of the sheet pile at the tie member setting point

(iii) The length of the embedded part of the sheet pile

(iv) The relationship between the rigidity of the sheet pile and the characteristics of the sea-bottom ground. Therefore the earth pressure distribution is not necessary as shown in **Fig. 2.3.7**.[34), 35), 36), 37)]

④ When P.W. Rowe's method, i.e., the elastic beam analysis method, is used in a sheet pile stability calculation, it is assumed that the earth pressure and residual water pressure act as those shown in **Fig. 2.3.8**, and a reaction earth pressure that corresponds to the modulus of subgrade reaction and the earth pressure at rest act on the front surface of the sheet pile.

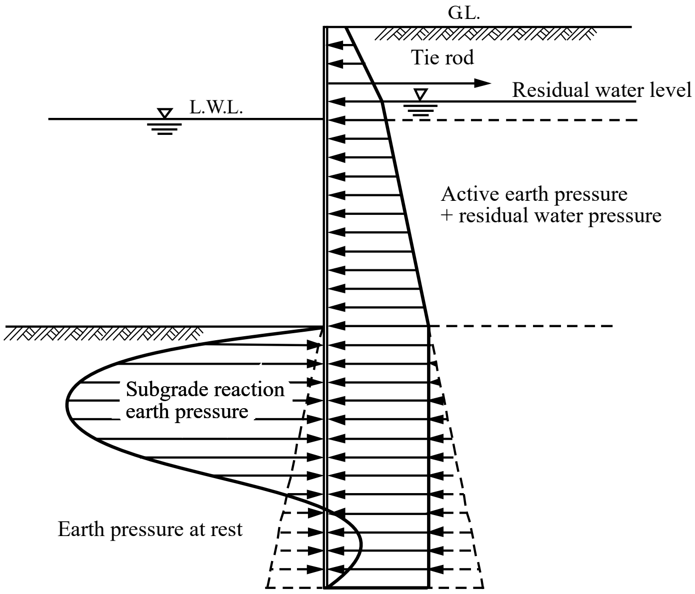


Fig. 2.3.8 Earth Pressure and Residual Water Pressure to Be Considered for the Performance Verification of Sheet Pile Walls Using P.W. Rowe's Method

⑤ When there is cargo handling equipment, such as cranes on the quaywall, it is necessary to take into consideration the earth pressure due to the self-weight and the live load of the equipment.

⑥ In the determination of the reaction force of earth pressure that acts on the front surface of the embedded part of the sheet pile, it is necessary to assume that the dredging of the sea bottom will be executed to a certain depth below the planned depth while considering the accuracy of the dredging work.

⑦ In the case of an earth retaining wall of an open-type wharf, the sea bottom in front of the sheet pile wall has a composite shape of horizontal and sloped surfaces. In such a case, the passive earth pressure may be calculated using Coulomb's method, in which the design passive earth pressure is calculated with several failure planes of different angles. The smallest value among them is adopted as the passive earth pressure.[38] However, it is necessary to consider the empirical evidence by experiments that the behavior of the ground in front of the sheet pile wall can be well predicted under the assumption of the ground being an elastic body.

⑧ The residual water level to be used in the determination of the residual water pressure needs to be estimated appropriately in consideration of the structure of the sheet pile wall and the soil conditions. The residual water level varies depending on the characteristics of the subsoil and the conditions of sheet pile joints. However, in many cases, the elevation with the height equivalent to 2/3 of the tidal range above the mean monthly LWL is used for sheet pile walls. However, in the case of a steel sheet pile wall driven into cohesive soil ground, care should be exercised in the determination of the residual water level because it is sometimes nearly the same as the high water level. When sheet piles made of other materials are used, it is preferable to determine the residual water level on the basis of the result of investigations of similar structures.

⑨ The seismic coefficient is used in the earthquake-resistance performance verification of sheet pile quaywalls for variable state in respect of Level 1 earthquake ground motion.

(a) For the performance verification of seismic-resistant of sheet piles quaywalls for the variable state in respect of Level 1 earthquake ground motion, the performance verification by the direct evaluation of the amount of deformation by a detailed method such as nonlinear effective stress analysis can be performed. However, simplified methods such as the seismic coefficient method based on the static equation of equilibrium can also be used. In this case, it is necessary to use an appropriate seismic coefficient in accordance with the deformation amounts of facilities in the performance verification and to take into consideration the effects of the frequency characteristics and duration of the ground motions.

(b) The characteristic value of the seismic coefficient for verification used in the performance verification of sheet pile quaywalls with the installation depth of deeper than -7.5 m may be calculated from **Equation (2.3.2)** by using the corrected maximum acceleration α_c and the allowable amount of deformation of the top of the quaywall D_a[10]. For the corrected maximum acceleration, refer to **Reference (Part III), Chapter 1, 1 Detailed Items about Seismic Coefficient for Verification**.

$$k_{h_k} = 1.91\left(\frac{D_a}{D_r}\right)^{-0.69}\frac{\alpha_c}{g} + 0.03 \quad \text{(vertical pile anchorage type)} \qquad \textbf{(2.3.2 (a))}$$

$$k_{h_k} = 1.32\left(\frac{D_a}{D_r}\right)^{-0.74}\frac{\alpha_c}{g} + 0.05 \quad \text{(coupled-pile anchorage type)} \qquad \textbf{(2.3.2 (b))}$$

where

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

α_c : corrected value of the maximum acceleration of the ground at the ground surface (cm/s^2);

g : gravitational acceleration (980 cm/s^2);

D_a : allowable amount of deformation at the top of the quaywall (15 cm);

D_r : standard deformation amount (10 cm).

(c) For the points of caution when using the seismic coefficient for verification, refer to **Part III, Chapter 5, 2.2.2 Actions, (2) Points of Caution When Setting Actions**.

(d) Setting of the allowable amount of deformation

1) It is necessary to appropriately set the allowable amount of deformation for a facility on the basis of the function required of the facility and the circumstances in which the facility is placed. The allowable value of the standard deformation amount of a sheet pile quaywall in Level 1 earthquake ground motion in **equation (2.3.2)** may be considered D_a = 15 cm.

2) It has been known that sheet pile walls, tie members, and vertical pile anchorage have enough cross-sectional capacity to allow them to undergo deformation of approximately 30 cm, which is the limit value from the viewpoint of serviceability. A D_a value of 15 cm does not cause the cross-sectional force to reach the yield point. However, it requires attention that the relative allowance of the cross-sectional force with respect to deformation becomes smaller as the wall heights decrease.

3) The sheet pile walls driven into very hard ground may have smaller deformation than those constructed in soft ground, but it is necessary to pay attention to the possibility that a small deformation may be the result of large cross-sectional force in members. Therefore, when setting the allowable deformation D_a at a standard value of 15 cm for sheet pile walls that have low wall heights and are driven into very hard ground, the additional deliberation of the earthquake resistance of sheet pile quaywalls shall be executed by 2D nonlinear earthquake response analysis or other appropriate methods.

(e) Considering that there may be a case that the above calculation method results in excessively large seismic coefficients for verification, any of the following measures can be taken when the calculation results in values larger than 0.25 provided that deformation is preferably confirmed directly by a dynamic analysis or other methods even when any of the following measures is used:

1) Setting of a cross section with a seismic coefficient for verification of 0.25 and evaluation of the cross section via a dynamic analysis that is capable of dealing with a dynamic mutual interaction between the ground and a structure

2) Deliberation of ground improvement

3) Adoption of another structural type

4) Deliberation of changing the allowable deformation D_a of a facility without excessively enlarging it while satisfying the requirement to prevent damage to a sheet pile quaywall due to Level 1 earthquake ground motions within a level enabling the function of the facility to be prevented from being impaired for its continuously use

⑩ The seismic coefficient for the verification of the coping of sheet pile under the accidental situation with respect to Level 2 earthquake ground motion may be conveniently calculated with **equation (2.3.2)** by using the acceleration time history of the ground surface at the free ground part. In this case, the allowable amount of deformation D_a may be considered 50 cm.

⑪ When using the method of ⑩ above, the seismic coefficient for the verification of copings shall be up to 0.25 and higher than that for Level 1 earthquake ground motions. However, when the seismic coefficient for the verification of Level 1 earthquake ground motion is higher than 0.25, the seismic coefficient for the verification of copings can also be higher than 0.25. Furthermore, the tensile strength force in tie members obtainable via dynamic analysis can be used when verifying the copings of an anchorage under an accidental situation with respect to Level 2 earthquake ground motions.

⑫ For the dynamic water pressure acting during an earthquake, refer to **Part II, Chapter 4, 3.2 Dynamic Water Pressure**.

⑬ There may be a case of sheet pile quaywalls with large copings that cause their actions on the sheet pile quaywalls due to earthquake ground motions to be too large to be omitted. In such a case, sheet pile quaywalls shall be verified with the possible action on sheet pile walls appropriately evaluated.

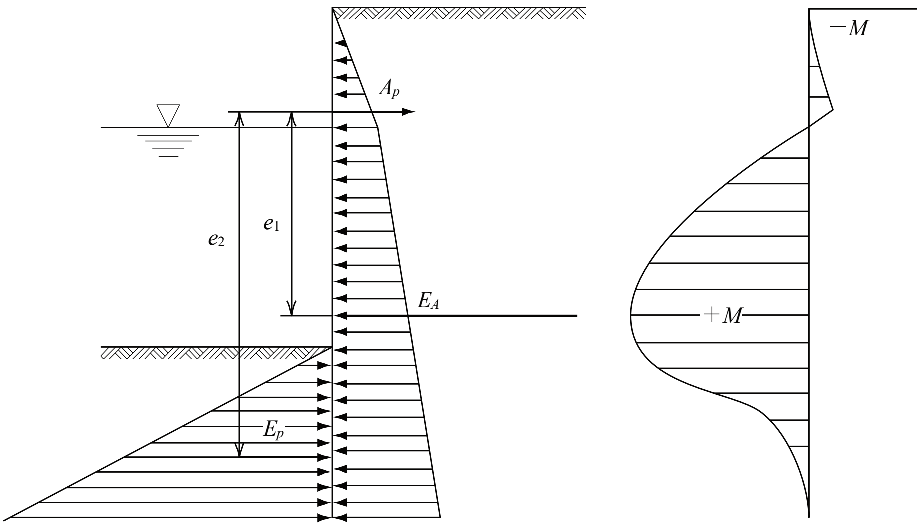
⑭ The fender reaction force is generally considered for the performance verification of the coping. The tractive force of a ship is not considered when the foundation for bollards needs to be constructed separately from the coping. However, when bollards need to be installed on the coping of the sheet pile wall, it is necessary to consider the tractive force of a ship in the performance verification of the coping, tie member, and waling.

2.3.5 Types of Performance Verification Methods for Sheet Pile Walls

(1) Free Earth Support Method[39)]

The free earth support method assumes that there is no negative bending moment in the embedded section of a sheet pile, i.e., a method for analyzing the structural stability assuming that no bending moment exists in the lower end of the embedment. In this method, the earth pressure and bending moment are generally assumed to act on a sheet pile as shown in **Fig. 2.3.9**.

The embedded length can be obtained by balancing the bending moment due to active earth pressure, passive earth pressure, and residual water pressure at the tie member installation position. The tensional force in a tie member can be obtained by subtracting the passive earth pressure from the sum of the active earth pressure and residual water pressure.



(a) Earth pressure + Residual water pressure (b) Flexural moment

$A_p = E_A - E_p \quad E_A e_1 = E_p e_2$

Fig. 2.3.9 Free Earth Support Method

(2) Equivalent Beam Method

The equivalent beam method calculates the maximum bending moment and reaction force at the tie member installation point of the sheet piles by assuming a simple beam supported at the tie member installation point and the sea bottom, with the earth pressure and residual water pressure acting as the load above the sea bottom as shown in **Fig. 2.3.10**.

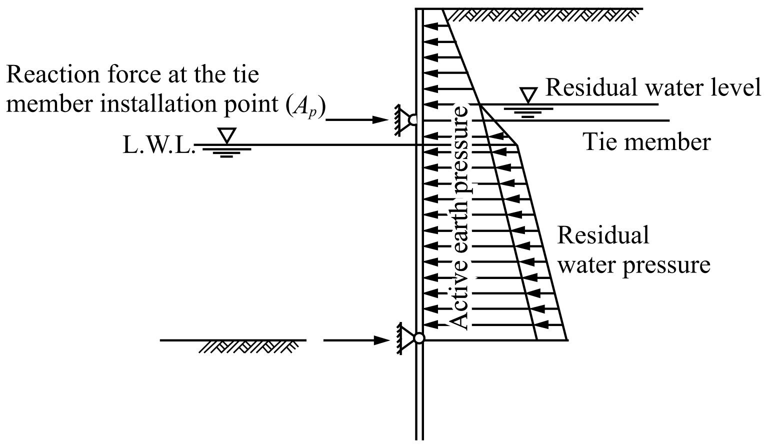
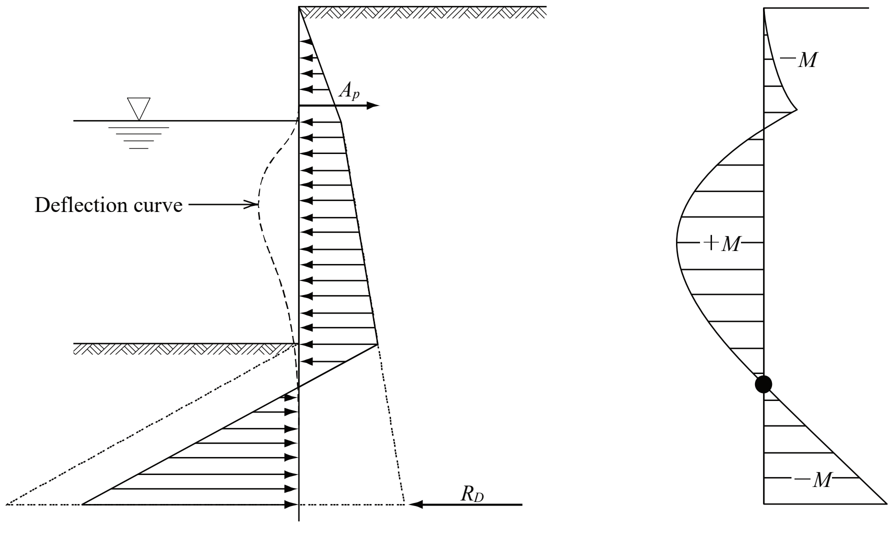


Fig. 2.3.10 Equivalent Beam Method for Obtaining Bending Moment

(3) Fixed Earth Support Methods

The fixed earth support methods analyze structural stability by assuming that a sheet pile is fixed at a certain depth of embedment in the ground. Therefore, the deflection curve of a sheet pile has a point of contrary flexure at a certain depth below a seafloor surface with negative bending moment acting on a section between the point of contrary flexure and the lower end of the sheet pile. Furthermore, these methods consider passive earth pressure in a negative direction at the lower end of the sheet pile, and the passive earth pressure is generally assumed as a concentrated load. In these methods, the earth pressure and bending moment acts on a sheet pile as shown in **Fig. 2.3.11 (b)**.

The deflection curve method is one of the typical fixed earth support methods. The deflection curve method obtains the force acting on a member by assuming an embedded length, drawing a deflection curve that approaches asymptotically to a vertical line at the lower end of the embedment, modifying the embedded length until the deflection at a tie member installation position becomes zero, and repeating the above process until the embedded length converges.



(a) Earth pressure + Deflection curve (b) Flexural moment

Fig. 2.3.11 Fixed Earth Support Method

(4) Elastic Beam Analysis Method for Sheet Piles

① The elastic beam analysis method applies the theory of a beam on elastic foundation to a sheet pile wall with an elastic modulus of subgrade reaction set for the ground where the sheet pile wall is embedded. The basic formula at the embedded section of a sheet pile is expressed as **equation (2.3.3)**.

$$EI\left(\frac{d^4 y}{dx^4}\right) = p(x) = P_{A_0} - \left(\frac{l_h}{D}\right)xy \tag{2.3.3}$$

where

E : Young's modulus of a sheet pile (MN/m^2);

I : geometrical moment of the inertia per unit width of the cross section of the sheet pile (m^4/m);

P_{A_0} : load intensity at the sea bottom generated by the active earth pressure and residual water pressure (MN/m^2);

l_h : modulus of the subgrade reaction of a sheet wall (MN/m^3);

D : embedded length of a sheet pile (m).

② Characteristic embedded lengths considering the effect of the cross-sectional stiffness of sheet piles

According to the elastic beam analysis method, the behavioral characteristics of sheet pile walls vary depending on their embedded lengths, i.e., the embedded lengths of sheet pile walls need to be longer than a certain length so that walls can be stabilized. The embedded length that places a sheet pile wall into a critically stabilized state is called a critical embedded length D_C.

When the extension of an embedded length is longer than the critical one, the bending moment on a sheet pile wall reaches peak maximum bending moment M_P in a free earth support state. The embedded length at this time is called a transitional embedded length D_P. The further extension of the embedded length causes the bending moment to reach convergent maximum bending moment M_F in a fixed earth support state. The minimum embedded length at this time is called a convergent embedded length D_F.

When calculating a critical embedded length under the conditions that all of the following partial factors in the performance verification equation with respect to embedded length in the free earth support method is set at 1.0 and that the passive earth pressure at an angle of wall friction δ is -15°, the calculation result is generally larger than a transitional embedded length D_P. This finding indicates that the sheet pile wall with the calculated embedded length has already come close to a fixed earth support state. Therefore, by considering that the free earth support method assumes the triangular distribution of reactive earth pressure similar to the case with passive earth pressure even though a sheet pile wall has already come close to the fixed earth support state and when the rigidity of a sheet pile is not considered in the calculation of embedded lengths, it can be said that the free earth support method cannot reflect actual phenomenon wherein the rigidity and the embedded length of a sheet pile affect the mechanical behavior of the embedded section and the distribution state of the reactive earth pressure of a sheet pile wall in the calculation of embedded lengths.

③ P.W. Rowe's method

Without following classical earth pressure theory, P.W. Rowe's method considers the passive earth pressure at the embedded section of a sheet pile as a subgrade reaction proportional to the lateral deflection and the depth from a seafloor surface and analyzes a sheet pile as a beam on elastic foundation[40]. P.W. Rowe's method requires complicated calculations, but it has been known that the calculation results of the method agree well with experimental results.

④ Correction of P.W. Rowe's method

(a) To simplify P.W. Rowe's method, Ishiguro[41] established the calculation charts of several coefficients necessary for a case of sheet piles embedded in sandy ground with unfixed upper ends (tie member installation position = hinge support) and unconfined lower ends. Furthermore, Takahashi and Kikuchi et al. [42), 43] used P.W. Rowe's method to analyze the behavior of a sheet pile in a fixed earth support state and established a method that enables several characteristic values to be calculated in proportion to the calculation results of the equivalent beam method by using indexes obtained by improving the flexibility numbers proposed by P.W. Rowe. The following is a method that is based on the modification of P.W. Rowe's method and can be used for solving the embedded section of a sheet pile as a beam on elastic foundation.

(b) Considering that there is no general solution to a differential equation of this form, a special technique is required to solve this equation. Broms and Rowe proposed a method for obtaining the coefficient of each term in a numerical analysis by assuming a power series as the solution. By using P.W.Rowe's method[40], Takahashi and Ishiguro[44] published details of a method that can derive a solution of the deflection curve equation of sheet pile wall and a computer-based numerical calculation method. Takahashi and Kikuchi amended this method to better reflect the behavioral characteristics of the actual sheet pile walls (see **Fig. 2.3.12**):

$$EI\left(\frac{d^4 y}{dx_4}\right) = p(x) = p_{A_0} + K_{AD}\gamma x - K_0\gamma x - \left(\frac{l_h}{D_F\gamma_f}\right)xy \qquad \textbf{(2.3.4)}$$

where

x : depth of the embedded section of a sheet pile below a ground level (m);

y : deformation of a sheet pile wall (m);

E : Young's modulus of sheet pile (MN/m^2),

I : geometrical moment of the inertia per unit width of the cross section of the sheet pile (m^4/m);

P_{A_0} : load intensity at the sea bottom generated by the active earth pressure and residual water pressure (MN/m^2);

K_{AD} : coefficient of active earth pressure in the embedded part of the sheet pile wall;

γ : unit weight of soil (MN/m^3);

K_0 : coefficient of earth pressure at rest;

D_F : convergent embedded length of sheet pile wall (m);

γ_f : ratio of the exerting depth of the primary positive reaction earth pressure acting on the front surface of the embedded part of the sheet pile to D_F.

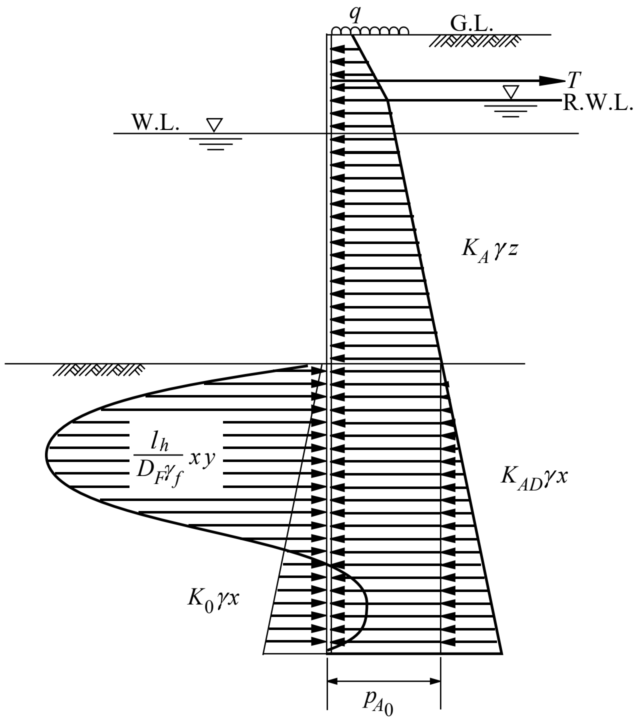


Fig. 2.3.12 Earth Pressure Distribution for the Analysis of Sheet Pile Wall

(c) Flexibility number of the sheet pile

As a measure to indicate the rigidity of a sheet pile wall as a structure, the following flexibility number proposed by Rowe is used in **equation (2.3.5)**:

$$\rho = H^4 / EI \tag{2.3.5}$$

where

ρ : flexibility number (m^3/MN);

H : total length of sheet pile (m);

E : Young's modulus of the sheet pile (MN/m^2);

I : geometrical moment of the inertia per unit width of the cross section of the sheet pile (m^4/m).

For H in **equation (2.3.5)**, P.W.Rowe uses the sum of the total height of the sheet pile wall from the sea bottom to the top of the sheet pile wall H and the embedded length D of a fixed earth support state ($H + D$) as the total length of the sheet pile.

Furthermore, Takahashi and Kikuchi et al. suggest a new index called the similarity number, which is derived using the flexibility number and ground characteristics. The height H_T from the sea bottom to the tie member installation point is used for the length H in this equation:

$$\omega = \rho l_h = (H_T{}^4 / EI) l_h \tag{2.3.6}$$

where

ω : similarity number;

ρ : flexibility number (m^3/MN);

l_h : modulus of subgrade reaction of the sheet pile wall (MN/m^3);

H_T : height from the tie installation point to the seabed surface (m);

E : Young's modulus of the sheet pile (MN/m^2);

I : geometrical moment of inertia per unit width of the cross section of the sheet pile (m^4/m).

By expressing the mechanical characteristics of a sheet pile wall with a similarity number ω, the effect of the rigidity of the sheet piles can be estimated quantitatively.

(d) Modulus of subgrade reaction of sheet piles

Little reference data provide the measured or suggested values of the modulus of the subgrade reaction of the sheet pile (l_h). Therefore, it is preferable to obtain these values by means of model tests and/or field measurements. The proposed values that have traditionally been used include the values proposed by Terzaghi and the ones proposed by Takahashi and Kikuchi et al., which have been obtained by modifying Terzaghi's values. The research conducted by Takahashi and Kikuchi et al. shows that the effect of errors in the modulus of subgrade reaction is not fatal for practical use. Therefore, the values proposed by Takahashi and Kikuchi et al. may normally be used as the coefficient of the subgrade reaction of the sheet pile wall.

1) Values proposed by Terzaghi[45)]

Table 2.3.1 shows the values proposed by Terzaghi.

Table 2.3.1 Modulus of Subgrade Reaction for Sheet Pile Wall in Sandy Ground (l_h) (MN/m^3)

Relative density of sand	Loose	Medium	Dense
Modulus of subgrade reaction (l_h)	24	38	58

2) Values proposed by Takahashi and Kikuchi et al.[43)]

Takahashi and Kikuchi et al.[43)] confirmed that the result of Tschebotarioff's model test of sheet pile wall[46)] does not contradict with the values proposed by Terzaghi. They related the modulus of subgrade reaction listed in **Table 2.3.1** with an *N* value by using the relationship between the modulus of subgrade reaction and the relative density proposed by Terzaghi, as well as the relationship between the *N* value and the relative density by Terzaghi.[47)] They then adopted the smaller value of the modulus of subgrade reaction to be on the safe side and connected the resultant values by using a smooth line (**Fig. 2.3.13**). They related the modulus of subgrade reaction with the angle of shearing resistance (**Fig. 2.3.14**) by using one equation from Dunham's equations for calculating the smaller angle of shearing resistance for a given *N* value:

$$\phi = \sqrt{12N} + 15 \tag{2.3.7}$$

where

ϕ : angle of shearing resistance (°);

N : N value.

However, it should be noted that **Fig. 2.3.14** is an expedient graph to a certain degree because Dunham's equations include cases that provide the larger angle of shearing resistance depending on the grain size of sandy soil. **Figs. 2.3.13** and **2.3.14** also show the values proposed by Terzaghi in addition to the values proposed by Takahashi and Kikuchi, at al.

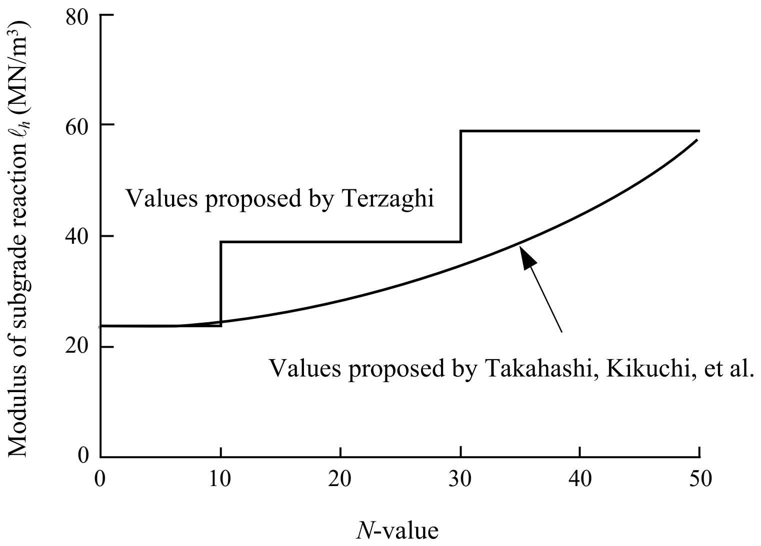


Fig. 2.3.13 Relationship between the Modulus of Subgrade Reaction (l_h) and the *N* value

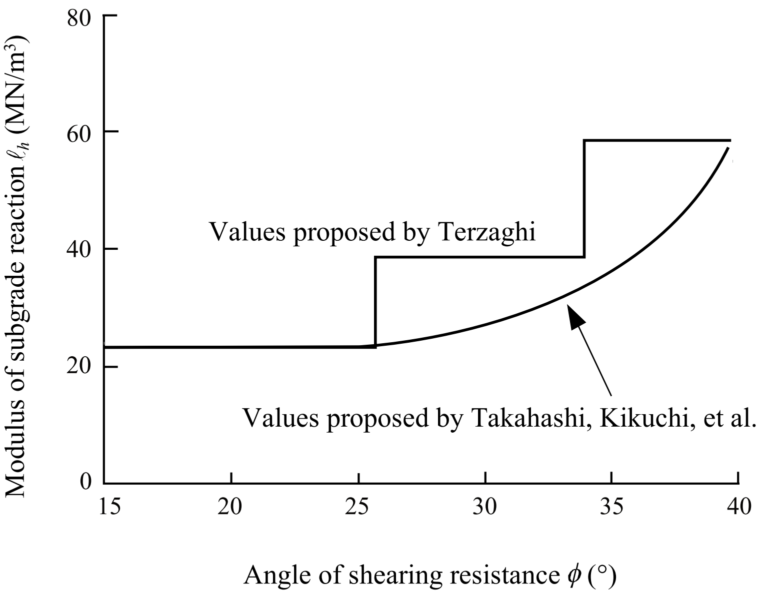


Fig. 2.3.14 Relationship between the Modulus of Subgrade Reaction (l_h) and the Angle of Shearing Resistance (ϕ)

⑤ Numerical calculation model by Morikawa et al.[48)]

By applying the spring model proposed by the Port and Harbour Research Institute to the subgrade reaction at the embedded section of a sheet pile, Morikawa et al.[48)] proposed a numerical calculation model in which the load intensity due to the active earth pressure on a rear face and earth pressure at rest on a front face that acts on the embedded section of a sheet pile wall as actions becomes p_{a0} (the load intensity on a seafloor surface due to active earth pressure and residual water pressure) on a seafloor surface and zero at the lower end of embedment. The bending moment in a sheet pile wall based on this model agrees well with experimental results.

2.3.6 Performance Verification that Takes into Consideration the Effects of the Cross-Sectional Rigidity of Sheet Pile Walls

(1) The cross section of a sheet pile shall be appropriately set by taking into consideration the cross-sectional rigidity of the sheet pile.

(2) The behavior of sheet pile walls with anchorages is strongly affected by the rigidity of sheet piles, ground characteristics, and embedded lengths. Particularly, the rigidity of sheet piles has a strong effect on the decision of embedded lengths. Therefore, it is necessary to study the effects of the cross-sectional rigidity of sheet piles when finally deciding the cross sections of sheet pile walls.

(3) Considering its simplicity and good past record, a method that combines the free earth support and equivalent beam methods has been frequently used. However, the method cannot be the method for the performance verification considering the effects of cross-sectional rigidity.

(4) In place of conventional design methods, the following are methods that take into consideration the effects of the cross-sectional rigidity of sheet pile walls: the combination or comparison of the free earth support and P.W. Rowe's methods when verifying embedded lengths; the combination or comparison of the equivalent beam and P.W. Rowe's methods when verifying the stress on sheet piles and tie members.

2.3.7 Performance Verification for the Overall Stability of Sheet Pile Walls

(1) Performance Verification Items for the Overall Stability of Sheet Pile Walls

When performing the performance verification for the overall stability of sheet pile walls under respective design situations, necessary items shall be appropriately set with reference to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls 【Interpretation】, Attached Tables 11-6 to 10** and **Part III, Chapter 5, 2.1 Common Items for Wharves【Interpretation】, and Attached Tables 11-1, 3 and 4**. In the case where the sheet pile quaywalls to be designed are not high-earthquake-resistance sheet pile quaywalls, performance verification with respect to accidental situation can be omitted.

(2) Performance Verification for the Slip Failure of the Ground under Permanent State

① The performance verification of sheet pile quaywalls with respect to the slip failure of the ground can be performed with reference to the performance verification for the slip failure of the ground in **Part III, Chapter 5, 2.2 Gravity-type Quaywalls**. In this performance verification, the subject to be generally studied is the circular slip failure of the slide planes passing below the lower end of a sheet pile wall. However, there are other cases wherein the circular slip failure of the slide planes passing through sheet pile walls is studied.[49] The values listed in **Table 2.2.1** of **Part III, Chapter 5, 2.2 Gravity-type Quaywalls** can be used as standard partial factors for the performance verification.

② When a sheet pile quaywall is determined to be unstable against circular slip failure, it is necessary to implement ground improvement by using an appropriate measure or by selecting another structural type. It is not advisable to extend the embedded lengths of sheet piles as a countermeasure against circular slip failure.

(3) Performance verification for the embedded lengths of sheet pile walls under permanent state and variable state with respect to Level 1 earthquake ground motions

① The mechanical behavior of the sheet pile wall varies depending on the embedded length. For a short embedded length, the behavior characteristics are free earth support conditions. For a long embedded length, the behavior characteristics are fixed earth support conditions. To ensure the stability of the sheet pile wall under permanent situations and variable situations, it is preferable that the bottom of the sheet pile is fixed sufficiently in the ground, i.e., fixed earth support conditions are satisfied.

② Conventionally, the embedded length was obtained by the free earth support method on the basis of classical earth pressure theory. Takahashi and Kikuchi[43] showed that the embedded length obtained with this method by considering appropriate partial factors is considered to be a fixed earth support condition. Furthermore, the equivalent beam method for obtaining the cross section of sheet piles assumes fixed earth support conditions.

③ The mechanical behavior of sheet pile walls with anchorages is largely affected by the rigidity of sheet piles, ground characteristics, and embedded lengths. Particularly, the mechanical behavior significantly differs according the embedded lengths. The performance verification methods under permanent state and variable state explained here are based on the prerequisite that the lower ends of the sheet pile walls are fixed.

④ The embedded lengths of sheet pile walls with their lower ends fixed vary depending on the rigidity of sheet piles and ground characteristics. Deciding the embedded lengths by using the free earth support method and earth pressure theory has a disadvantage in terms of determining the embedded lengths regardless of the stiffness of sheet piles and the discrepancy between the theory and actual behavior represented by the assumed distribution of passive earth pressure in disagreement with the triangle distribution of the Coulomb earth pressure. However, under certain conditions, even the embedded lengths decided via this method can achieve a fixed earth support state.

⑤ When obtaining the embedded length of sheet piles by using the free earth support method, the analysis of the embedded length of the sheet pile wall can be performed using **equation (2.3.8)** on the basis of the equilibrium of moments of the earth pressure and residual water pressure on the point of installation of the tie members (**Fig. 2.3.9**). In the following equation, subscripts *k* and *d* indicate the characteristic value and the design value, respectively. Furthermore, the partial factors in the equation can be selected from **Table 2.3.2**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience.

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

$$R_k = aP_{p_k} \tag{2.3.8}$$

$$S_k = bP_{a_k} + cP_{w_k} + dP_{dw_k}$$

where

P_p : resultant passive earth pressure acting on the sheet pile wall (kN/m);

P_a : resultant active earth pressure acting on the sheet pile wall (kN/m);

P_w : resultant residual water pressure acting on the wall structure (kN/m);

P_{dw} : resultant active water pressure acting on the wall body (kN/m) (only during earthquakes);

a to d : distance between the position of installation of the tie member and the point of action of the resultant force (m);

R : resistance term (kN·m/m);

S : load term (kN·m/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.2 Partial Factors Used for the Verification of the Embedded Lengths of Sheet Pile Walls

Verification object	Soil layer compositions	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Embedded length of sheet pile by the free earth support method (Permanent state)	Sandy ground	0.72	1.09	— (1.00)
	Soil layer with the inclusion of cohesive soil	0.77	1.11	
Embedded length of sheet pile by the free earth support method (Variable state with respect to Level 1 earthquake ground motions)	All soil layer	— (1.00)	— (1.00)	1.20

⑥ In **Table 2.3.2**, the soil layer compositions refer to the compositions of the soil layers from the ground surface to the lower end of embedment. If all soil layers are sandy ones, the partial factors for the sandy ground can be used. If cohesive soil is included even partially, the partial factors for the soil layer with the inclusion of cohesive soil can be used.

⑦ The partial factors for verifying the embedded lengths under the permanent state mentioned above are the coefficients calculated by the free earth support method on the basis of past design examples of sheet pile quaywalls with anchorages.[50)]

⑧ Embedded lengths by the P.W. Rowe's method

(a) The characteristic values of the embedded lengths of sheet pile walls using P.W. Rowe's method can be calculated to satisfy **equation (2.3.9)**. Considering that **equation (2.3.9)** considers the rigidity of sheet piles without earth pressure, attention is required to the possibility that earth pressure reduction effects do not necessarily contribute to the reduction in the embedded lengths of sheet piles when planning ground improvement methods for alleviating earth pressure on existing steel sheet pile quaywalls. Therefore, when expecting the earth pressure reduction effects, it is advisable to use the methods ① to ⑤ above in combination with P.W. Rowe's method.

$$\delta_s = \frac{D_F}{H_T} \geq 5.0916\omega^{-0.2} - 0.2591 \qquad \textbf{(2.3.9)}$$

where

δ_s : ratio of the embedded length of a sheet pile wall to the height from a tie member installation position to a seafloor surface;

D_F : embedded length of a sheet pile wall (m);

H_T : height from a tie member installation position to a seafloor surface (m);

ω : similarity number (pl_h);

ρ : flexibility number (H_T^4/EI) (m³/MN);

E : Young's modulus of a sheet pile wall (MN/m²);

I : geometrical moment of the inertia per unit width of the cross section of the sheet pile (m⁴/m);

l_h : coefficient of subgrade reaction of a sheet pile wall (MN/m³).

(b) The embedded length calculated with this equation is the convergent embedded length. According to the study conducted by Takahashi and Kikuchi et al., an increase of just 2% in the maximum bending moment occurs when an embedded length corresponding to 70% of the convergent embedded length is employed. Therefore, the use of the convergent embedded length as the design embedded length secures the safety, and there is no need to consider a margin against the safety.

(c) **Equation (2.3.9)** formulates the relationship between the ratio of the convergent embedded length D_F to the virtual wall height H_T, i.e., $\delta = (D_F/H_T)$, and the similarity number ω as shown in **Fig. 2.3.15**. This is based on the analysis performed by Takahashi and Kikuchi et al. by using a simulation model for 72 cases with a combination of conditions for the water depth of the quay (-4 to -14 m), soil conditions, seismic conditions (k_h = 0.20), and material conditions of steel sheet piles. In **Fig. 2.3.15**, δ for permanent situations and earthquake conditions are obtained as δ_N and δ_S, respectively; however, in **equation (2.3.9)**, δ_S is used for the action of earthquakes because it indicates large values.

(d) Furthermore, in the analysis by Takahashi and Kikuchi et al. the relationship between the similarity number ω, ratio μ (M_F/M_T), and ratio τ (T_F/T_T) were studied. The ratio μ is the ratio of the maximum bending moment M_F when there is convergent embedded length D_F in the bending curve analysis to the maximum bending moment M_T calculated by the equivalent beam method by assuming the tie installation point and the seabed surface as the support points. The ratio τ is the ratio of tie tension force T_F when there is convergent embedded length D_F in the bending curve analysis to the tie tension force T_T calculated from the equivalent beam method. These relationships are shown in **Figs. 2.3.16** to **2.3.17**.

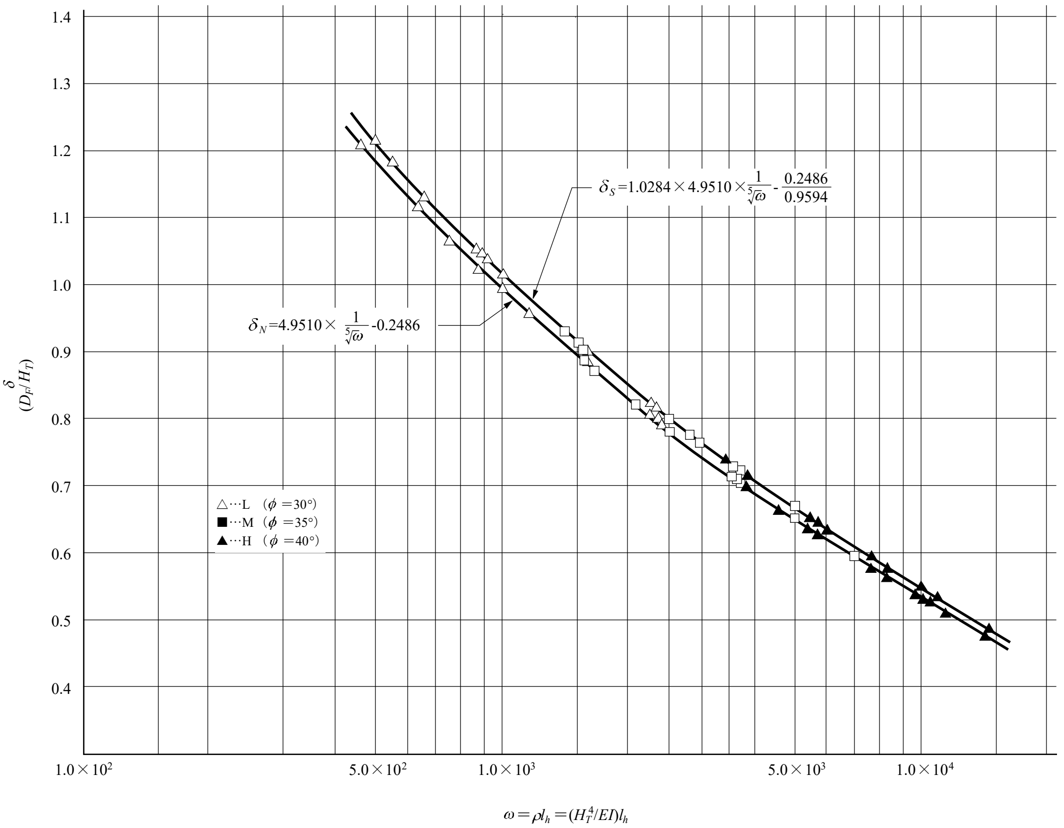


Fig. 2.3.15 Relationship between ω and δ

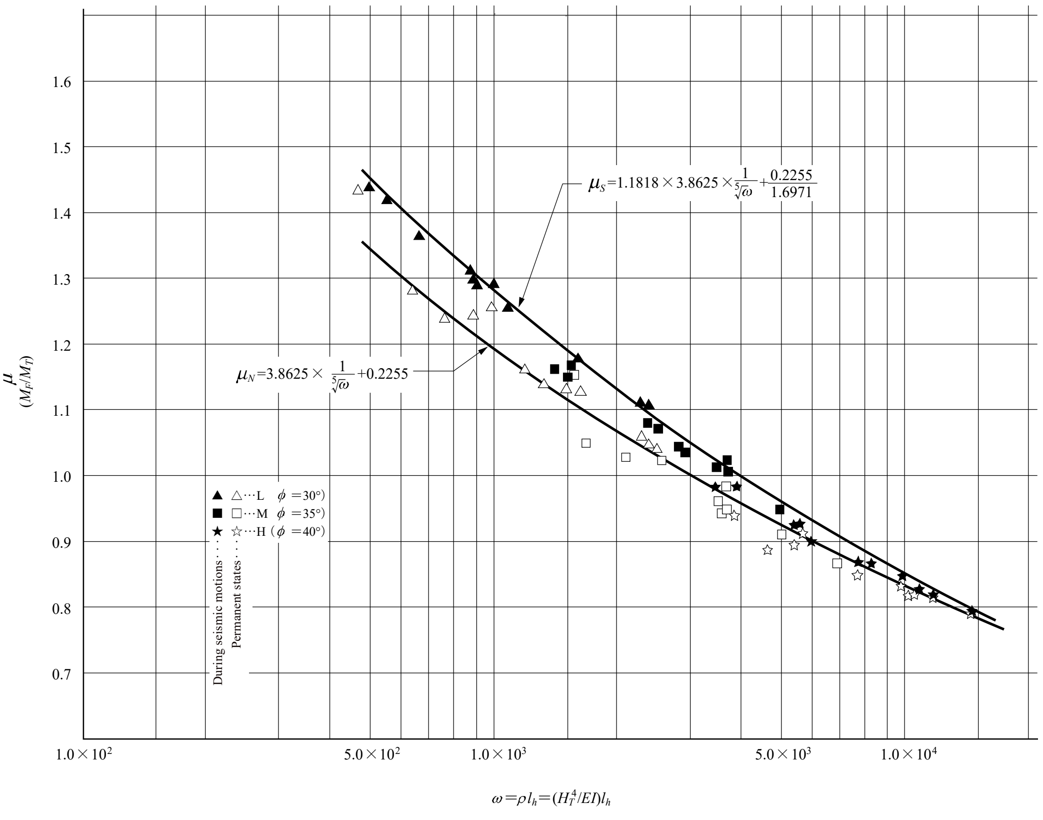


Fig. 2.3.16 Relationship between μ and ω

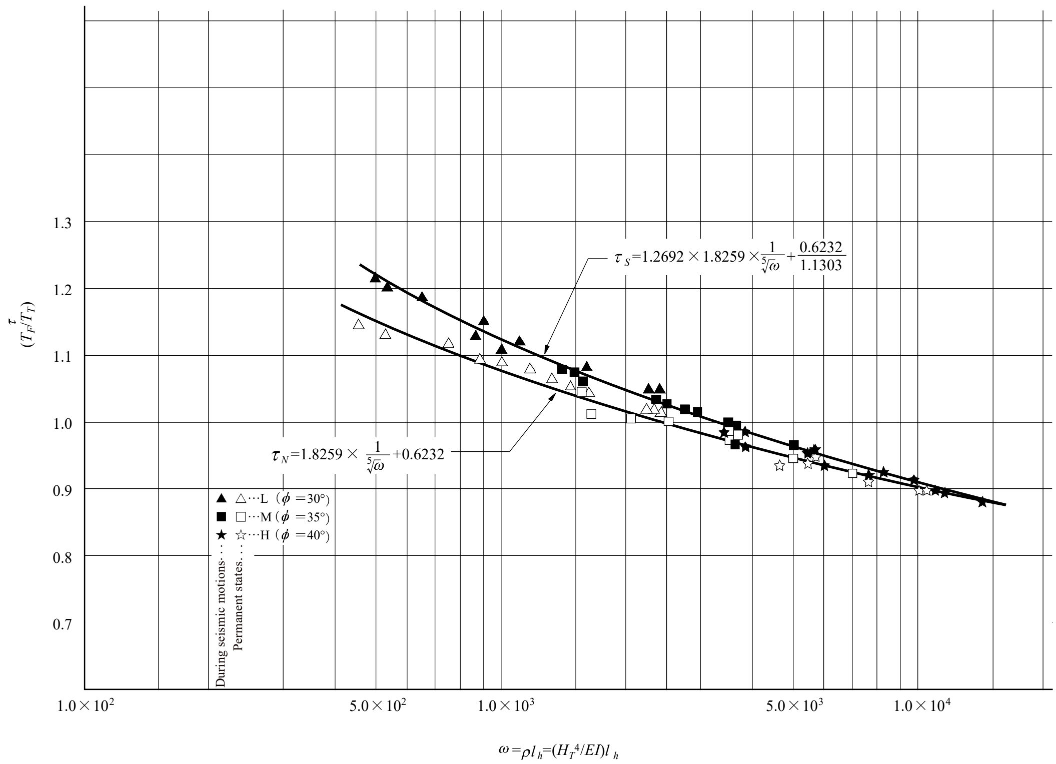


Fig. 2.3.17 Relationship between τ and ω

(e) Comparison of the embedded length by the free earth support and P.W. Rowe's methods

1) Once the fixed earth support state is achieved, the structure of a sheet pile wall is stabilized, and the further extension of an embedded length does not cause any more changes in the maximum bending moment in a sheet pile wall and an action distance D_R (refer to **Fig. 2.3.18**) of the first reaction earth pressure on the front face of the sheet pile wall. Therefore, the required embedded length for a sheet pile quaywall with an anchorage shall be the embedded length that achieves fixed earth support state. In other words, it is rational to set a minimum embedded length that achieves the fixed earth support state as the required embedded length (D_D).

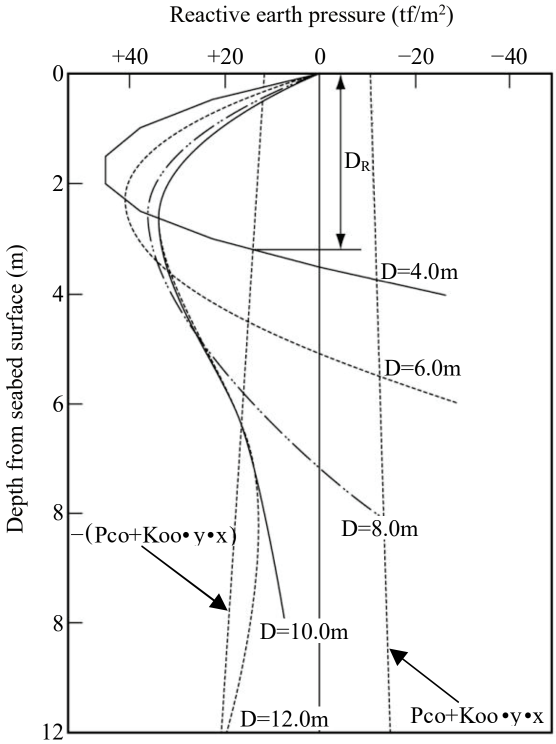


Fig. 2.3.18 Action distance D_R[43] of the First Reaction Earth Pressure

2) The relationship between the ratio v (D_D/D_t) of the required embedded length (D_D) to the embedded length (D_t) obtained by the free earth support method and the flexibility number ρ is shown in **Fig. 2.3.19**. This figure means that the embedded length obtained by the free earth support method is not sufficient enough to achieve the fixed earth support state in the region with v = 1 or larger.

3) Under any design situation, v is likely to be increased with a decrease in ρ. Therefore, it is necessary to be fully aware of the possibility that the embedded length obtained through the free earth support method may not achieve a complete fixed earth support state.

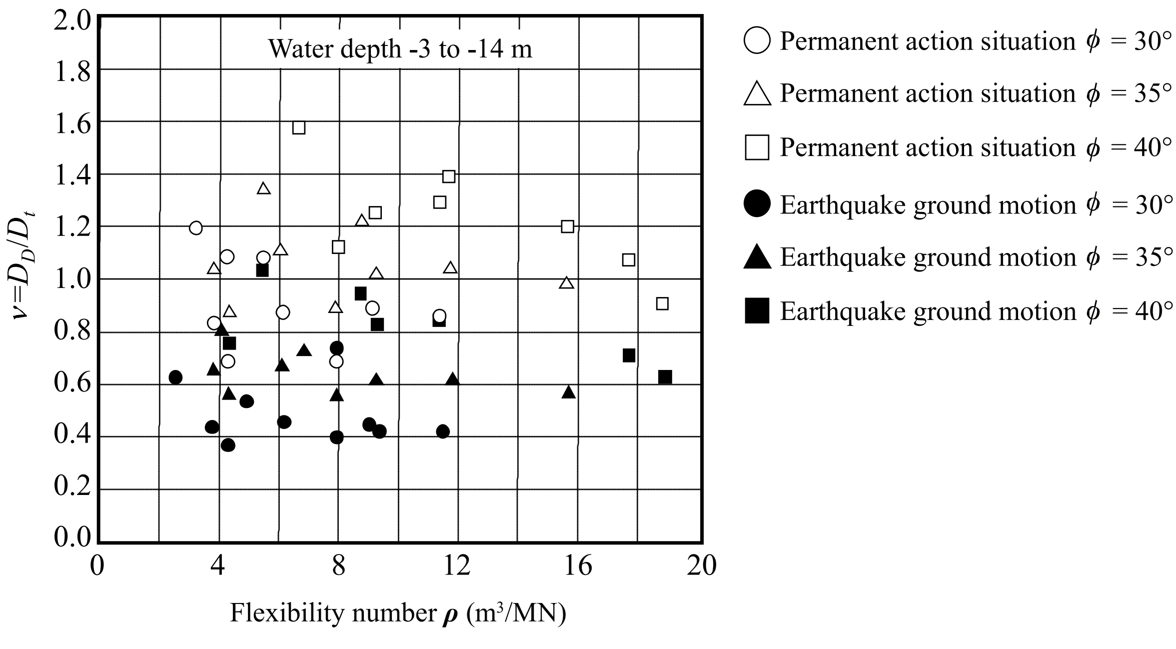


Fig. 2.3.19 Relationship between ρ and v

(4) Performance Verification of Stress on Sheet Pile Walls under Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion

① The maximum bending moment of sheet piles and reaction at the tie member installation point which are necessary for the verification of the stress on sheet pile walls and tie members shall be calculated with an appropriate method that takes into consideration the rigidity and embedded length of the sheet piles and the characteristics of the ground.

② The characteristic values of maximum bending moment in the sheet pile wall and the reaction force at the tie member installation point can normally be calculated using the **equations (2.3.10)** and **(2.3.11)**. In the following equation, subscript k indicates the characteristic value.

(a) The reaction force at the tie member installation point

$$A_{p_k} = P_{a_k} + P_{w_k} + P_{dw_k} - \frac{(aP_{a_k} + bP_{w_k} + cP_{dw_k})}{L} \qquad \textbf{(2.3.10)}$$

where

A_p : reaction force at the tie member installation point (kN/m);

P_a : resultant active earth pressure from the top of the sheet piling to the seabed surface (kN/m);

P_w : resultant residual water pressure from the top of the sheet piling to the seabed surface (kN/m);

P_{dw} : resultant dynamic water pressure acting on the sheet pile wall (kN/m) (only during earthquakes);

a to c : distance from the installation position of the tie member to the point of action of the resultant force (m);

L : distance from the installation position of the tie member to the seabed surface (m).

(b) Maximum bending moment

$$M_{\max_k} = aA_{p_k} - bP'_{a_k} - cP'_{w_k} - dP'_{dw_k} \qquad \textbf{(2.3.11)}$$

where

A_p : reaction at the tie installation point (kN/m);

P'_a : resultant active earth pressure from the top of the sheet pile to the position where the shear force S becomes 0 (kN/m);

P'_w : resultant residual water pressure from the top of the sheet pile to the position where the shear force S becomes 0 (kN/m);

P'_{dw} : resultant dynamic water pressure from the top of the sheet pile to the position where the shear force S becomes 0 (kN/m) (during an earthquake only);

a : distance from the position where the shear force S becomes 0 to the tie member installation position (m);

b to d : distance from the position where the shear force S becomes 0 to the point of action of the resultant force (m).

③ The maximum bending moment and reaction force at the tie member installation points on sheet piles may be determined using the equivalent beam method or P.W. Rowe's method. However, care should be exercised when using the equivalent beam method for sheet piles with high rigidity because the method causes the point of contraflexure of the bending moment to be deeper than a seafloor surface and may underestimate the sectional force in the sheet piles.

④ When the maximum bending moment of sheet piles is to be determined by taking the effects of the modulus of subgrade reaction and the rigidity of the sheet piles into consideration, the maximum bending moment can be obtained by getting a correction factor (**Figs. 2.3.16** and **2.3.17**) and multiplying the value of the maximum bending moment preliminarily obtained by the equivalent beam method by the correction factor.

Although the characteristic value of the seismic coefficient for performance verification purposes shown in **Figs. 2.3.16** and **2.3.17** has been set at 0.20, the values obtained from these figures may be used for the performance verification under variable state with respect to Level 1 earthquake ground motion.

⑤ The seabed surface used in calculating the bending moment should take the margin of the depth into consideration.

⑥ When the seabed surface in front of a sheet pile is not flat, it is necessary to pay attention to the possible underestimation of a bending moment calculated with the seabed surface as a point of support.

⑦ The analysis of stresses in the sheet pile wall may be performed using **equation (2.3.12)**. In this equation, subscripts *k* and *d* indicate the characteristic value and the design value, respectively. The partial factor in the equation can be selected from the values listed in **Table 2.3.3** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = \sigma_{yk} \tag{2.3.12}$$

$$S_k = \frac{M_{\max k}}{Z}$$

where

σ_y : bending yield stress of the steel material (N/mm²);

$M_{\max}$: maximum bending moment in the sheet pile wall (N·mm/m);

Z : section modulus of the steel material (mm³/m);

R : resistance term (N/mm²);

S : load term (N/mm²);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.3 Partial Factors for the Stress Verification of Sheet Pile Walls

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in the sheet pile wall (Permanent state)	0.84	1.18	– (1.00)
Stress in the sheet pile wall (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.12

⑧ The above partial factors for the stress verification of sheet pile walls under a permanent state are the values calculated on the basis of the past design examples of sheet pile quaywalls with anchorages.[50)]

⑨ The adjustment factors for the stress verification of sheet pile walls under a variable state with respect to Level 1 earthquake ground motion are the values set with reference to practical safety factors for the yield stress of steel materials in the previous design methods.

⑩ When the reaction force at the tie member installation point of sheet piles need to be determined by taking the effects of the modulus of subgrade reaction and the rigidity of the sheet piles into consideration, the reaction force at the tie member installation point can be obtained by getting a correction factor from **Figs. 2.3.16** and

2.3.17 and by multiplying the value of the reaction force at the tie member installation point that is preliminarily obtained by the equivalent beam method by the correction factor.

⑪ Refer to **Part II, Chapter 11, 2 Steel Materials** for the yield stress of steel sheet piles.

(5) Performance Verification of Stress on Tie Members under a Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion and the Tractive Forces Of Ships

① The analysis of stresses in tie members may be performed using **equation (2.3.13)**. In this equation, subscripts *k* and *d* indicate the characteristic value and the design value, respectively. The partial factor in the equation can be selected from the values listed in **Table 2.3.4** in which the symbol "**—**" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = \sigma_{yk} \qquad \textbf{(2.3.13)}$$

$$S_k = \frac{T_k}{A}$$

where

σ_y : tensile yield stress of a tie member (N/mm^2);

T : tension force on a tie member (N);

A : cross-section area of a tie member (mm^2);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

For the design value of the tension force in the tie members, refer to ④ **Tension force acting on tie members**.

② The partial factors in **Table 2.3.4** for the stress verification of tie members under a permanent are the values calculated on the basis of past design examples of sheet pile quaywalls with anchorages.[50)]

③ The adjustment factors for the stress verification of tie members under a variable state with respect to Level 1 earthquake ground motion are the values set with reference to practical safety factors for the yield stress of steel materials in the previous design methods.

Table 2.3.4 Partial Factors for the Stress Verification of Tie Members

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in the tie member (Permanent state)	0.64	1.29	– (1.00)
Stress in the tie member (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.67

④ **Tension force acting on tie members**

(a) The tension acting on a tie member can be calculated on the basis of the reaction at the tie installation point calculated in accordance with **(4) Performance Verification of Stress on Sheet Pile Walls under Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion** above.

In this case, the reaction at the tie member installation point should be calculated by taking the rigidity of the sheet pile wall cross section into consideration. Note that the reaction at the tie member installation point that is calculated in accordance with **(4) Performance Verification of Stress on sheet Pile Walls under Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion** above is the reaction per meter of quaywall length. Tie members are usually installed at fixed intervals, and tie members may be attached in some cases at a certain angle with the line perpendicular to the sheet pile wall to avoid the existing structure located behind the wall. Therefore, it is necessary to calculate the tie member tension force by considering these site conditions.

(b) The tension force that acts on a tie member is generally calculated by **equation (2.3.14)**. In the equation below, subscript *k* stands for the characteristic value.

$$T_k = A_{p_k} l \sec\theta \qquad \textbf{(2.3.14)}$$

where

T : tension force of the tie member (kN);

A_p : reaction at the tie member installation point (kN/m);

l : tie member installation interval (m);

θ : inclination angle of the tie member to the line perpendicular to the sheet pile wall (°).

(c) In some cases, bollards are installed on the coping of a sheet pile wall, and the tractive forces of ships acting on the bollards are transmitted to the tie members. Usually, the coping is assumed to be a beam with the tie members as elastic supports, and the tie member tension force may be calculated using **Equation (2.3.15)** by assuming that the tractive force is evenly shared by four tie members near a bollard. In the equation below, subscript *k* stands for the characteristic value.

$$T_k = \left(A_{p_k} l + \frac{P_k}{4} \right) \sec\theta \qquad \textbf{(2.3.15)}$$

where

T : tension force acting in the tie member (kN);

A_p : reaction force at the installation point of the tie member (kN/m);

l : spacing of installation of tie members (m);

θ : inclination angle of tie member in perpendicular to the sheet pile wall and the tie member (°);

P : horizontal component of the tractive force of a ship acting on a bollard (kN).

Refer to **Part II, Chapter 8, 2.4 Actions due to Traction by Ships** for details on the tractive forces of ships.

(d) In the case of soft ground with a risk of settlement, a certain degree of safety margin shall be considered when verifying the tension force.

⑤ Tie rods

(a) For the yield stress of tie rods, refer to **Table 2.3.5**.

Table 2.3.5 Characteristics of Tie Rod Materials

Type	Rupture strength (N/mm^2)	Yield stress (N/mm^2)	Elongation (%)	Yield stress/rupture strength
SS400	≥402	(dia. 40 mm or less) ≥ 235	≥24	0.58
		(dia. > 40 mm) ≥ 215	≥24	0.53
SS490	≥490	(dia. 40 mm or less) ≥ 275	≥21	0.56
		(dia. > 40 mm) ≥255	≥21	0.52
High tensile strength steel 490	≥490	≥325	≥24	0.66
High tensile strength steel 590	≥590	≥390	≥22	0.66
High tensile strength steel 690	≥690	≥440	≥20	0.64
High tensile strength steel 740	≥740	≥540	≥18	0.73

(b) Tie rods are subjected to bending moment at their installation positions on a sheet pile when backfill soil at the back of the sheet pile settles. When tie rods made of brittle materials with small elongation are subjected to tension force with bending moment acting on them, the tie rods undergo the reduction in rupture strength. Therefore, it is advisable that tie rods are made of steel materials that ensure characteristics corresponding to the elongation of 18% or more when tested using type 3 test pieces as stipulated in the **Test Pieces for the Tensile Test of Metallic Materials (JIS Z 2201)**.

(c) The tensile stress in the tie rod is calculated using the cross section from which the amount of corrosion has been deducted. For the amount of corrosion, refer to **Part II, Chapter 11, 2.3.3 Corrosion Rates of Steel**.

⑥ Tie wires

(a) The tie rods can be replaced by tie wires made by intertwining hardened steel wires with characteristics that are equivalent to **hardened steel wire rods (JIS G 3506)** or PC steel wires with characteristics equivalent to **piano wire rods (JIS G 3502)**. Considering that tie wires do not have clear yield points, the stress causing 0.2% permanent distortion can be considered the yield point for tie wires, and it is necessary to confirm that the ratio of the yield point to rupture strength does not become lower than 2/3.

(b) When using polyethylene materials as the corrosion protection coating for tie wires, it is necessary to pay attention to ensuring their durability and carefully implementing backfilling and other work so that damage can be prevented to the coating for tie wires.

⑦ In the case where a loose sandy surface layer behind a sheet pile wall is saturated with water, there is a risk of liquefaction on the occurrence of earthquakes and a significant increase in tension force on tie members with earth pressure increased owing to a liquefied sandy layer. In such a case, it is advisable to take necessary measures such as ground improvement to prevent liquefaction.

⑧ Although it is normal practice to select the most economic type of tie rod via a comparative study, tie rods with high yield stress in tension are advantageous when subjected to a large tension force. However, when adopting high-strength steel, it is necessary to pay attention to the fact that the ratio of yield stress to rupture strength of high-strength steel is smaller than that of ordinary steel; when subjected to bent anchorage at 15°, the rupture strength of even the high-strength steel with elongation of approximately 20% may be reduced to approximately 85% of normal anchorage depending on the manufacturing processes.[51)]

2.3.8 Performance Verification of Stresses in Waling

(1) For waling, it is necessary to perform the analysis of stresses in waling under the permanent state and the variable state with respect to Level 1 earthquake ground motion.

(2) The analysis of stresses in waling may be performed using **equation (2.3.16)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected

from the values in **Table 2.3.6** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

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

$$R_k = \sigma_{y_k} \tag{2.3.16}$$

$$S_k = \frac{M_{\max k}}{Z}$$

where

σ_y : bending yield stress in the waling (N/mm²);

$M_{\max}$: maximum bending moment in the waling (N·mm/m);

Z : section modulus of the waling (mm³/m);

R : resistance term (N/mm²);

S : load term (N/mm²);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

For the calculation of the maximum bending moment in the waling, refer to (3) below.

Table 2.3.6 Partial Factors for the Stress Verification of Waling

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in waling (Permanent state)	– (1.00)	– (1.00)	1.67
Stress in waling (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.12

(3) In general, the maximum bending moment of waling may be calculated using **equation (2.3.17)**. In the equation below, subscript k stands for the characteristic value.

$$M_{\max_k} = \frac{T_k l}{10} \tag{2.3.17}$$

where

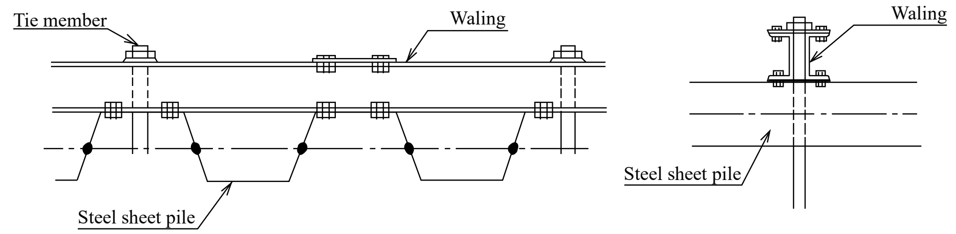
$M_{\max}$: maximum bending moment of waling (kN·m);

T : tension force of a tie member calculated in accordance to **Part III, Chapter 5, 2.3.7 (5) ④ Tension force acting on tie members** (kN);

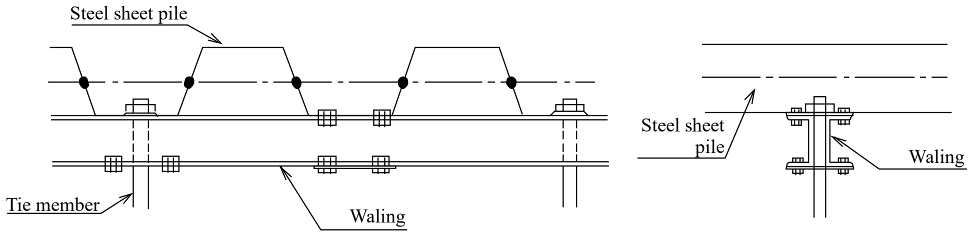
l : tie member installation interval (m).

This equation is obtained by analyzing a three-span continuous beam supported at the tie member installation points and subjected to the reaction at the tie installation point (A_p) as a uniformly distributed load.

(4) Sheet piles and tie members need to be connected via waling materials horizontally installed on the upper portions of the sheet piles. The waling materials are generally fabricated by assembling channel steel (**Fig. 2.3.20**) in general, but angle steel or H-section steel can also be used in place of channel steel.



(a) Case of the installation at the seaside of a sheet pile



(b) Case of the installation at the landside of a sheet pile

Fig. 2.3.20 Examples of Waling Installation

(5) Refer to **Part II, Chapter 11, 2 Steel Materials** for the yield stress of waling.

(6) It is advisable that waling be subjected to less stress and be embedded in copings from the viewpoint of corrosion protection. In the case of waling not embedded in copings, the analysis of the stress applied to the waling shall be based on cross sections without corrosion allowance. For the amount of corrosion, refer to **Part II, Chapter 11, 2.3.3 Corrosion Rates of Steel**.

(7) When bollards are installed on copings, it is necessary to verify the performance of waling near one of the bollards by using a tie member tension force that takes into consideration the tractive force of the ship in accordance with **Part III, 2.3.7 (5) ④ Tension force acting on tie members** above. However, when the wale is embedded in the copings, the effect of the tractive force of the ship may be ignored.

(8) Waling can be installed at either the seaside or landside of sheet piles (refer to **Fig. 2.3.20**). In the case of wales installed at the seaside of sheet piles, the number of locations to bolt the sheet piles to the waling can be reduced without forcibly bolting each sheet pile to the waling because the sheet piles naturally lean on the waling. However, it is necessary to increase the thickness of copings to embed waling fully in concrete because the corrosion on or damage to waling causes the fatal failure of sheet pile walls. In the case of waling installed at the landside of sheet piles, a larger number of bolts that have sufficient strength to fix sheet piles to waling are required. Although construction becomes complex, the thickness of copings for the waling installed at the landside of sheet piles can be reduced compared with the case of waling installed at the seaside.

2.3.9 Performance Verification of Anchorages

(1) The stability of anchorages shall be verified for permanent state and Level 1 earthquake ground motions. Ensuring the stability of anchorages is necessary by using appropriate methods on the basis of the structural characteristics of sheet pile quaywalls and anchorages.

(2) Examination of the Stability of Vertical Pile Anchorages

① The vertical pile anchorages can be verified as vertical piles to which the tension force of tie members is horizontally applied.

② For the performance verification of the vertical pile anchorages, refer to **Part III, Chapter 2, 3.4.6 Deflection of Piles Receiving Force Perpendicular to Axes**.

③ The analysis of stresses in vertical pile anchorages may be performed using **equation (2.3.18)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.3.7** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

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

$$R_k = \sigma_{y_k} \tag{2.3.18}$$

$$S_k = \frac{M_{\max k}}{Z}$$

where

σ_y : bending yield stress of a pile anchorage (N/mm^2);

$M_{\max}$: maximum bending moment in a pile anchorage (N·mm/m);

Z : section modulus of a pile anchorage (mm^3/m);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.7 Partial Factors for the Stress Verification of Vertical Pile Anchorages

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in vertical pile anchorage (Permanent state)	– (1.00)	– (1.00)	1.67
Stress in vertical pile anchorage (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.12

④ When the active failure plane of a sheet pile and the passive failure plane of a pile anchorage drawn in accordance with **Part III, 2.3.3 Setting of Cross-Sectional Dimensions, (3) Installation Locations of Anchorages** intersect each other at a position lower than a tie member installation point on a pile, the analysis of stresses in vertical pile anchorage can be generally performed by assuming a horizontal plane including the intersection point as a virtual ground surface with no soil above it.[33)]

(3) Examination of the Stability of Coupled-pile Anchorages

① The coupled-pile anchorages can be verified as coupled piles to which the tension force of tie members is horizontally applied.

② The performance verification of the coupled-pile anchorages can refer to **Part III, Chapter 2, 3.4.6 Deflection of Piles Receiving Force Perpendicular to Axes**.

③ The analysis of stresses in coupled-pile anchorages may be performed using **equation (2.3.19)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.3.8** in which the symbol "—" in a column means that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = \sigma_{y_k} \tag{2.3.19}$$

$$S_k = \left(\frac{N}{A} + \frac{M_{\max}}{Z} \right)$$

where

σ_y : bending yield stress of a coupled-pile anchorage (N/mm^2);

N : axial force acting on coupled piles (N);

A : cross-section area of coupled piles;

$M_{\max}$: maximum bending moment in a pile anchorage (N·mm/m);

Z : section modulus of a pile anchorage (mm^3);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.8 Partial Factors for the Stress Verification of Coupled-Pile Anchorages

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in coupled-pile anchorage (Permanent state)	– (1.00)	– (1.00)	1.67
Stress in coupled-pile anchorage (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.12

④ The analysis of axial force in coupled-pile anchorages may be performed using the **equation (2.3.20)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.3.9** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = R_{u_k} \tag{2.3.20}$$

$$S_k = N_k$$

where

N : axial force of a coupled-pile anchorage (N);

R_u : maximum static axial resistance force of a coupled-pile anchorage (N);

R : resistance term (N);

S : load term (N);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.3.9 Partial Factors for the Axial Force Verification of Coupled-Pile Anchorages

Verification object	Type of pile		Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Axial force acting on coupled-pile anchorage (Permanent state)	Tension pile		1.00	1.00	3.00
	Compression pile		1.00	1.00	2.50
Axial force acting on coupled-pile anchorage (Variable state with respect to Level 1 earthquake ground motions)	Tension pile		1.00	1.00	2.50
	Compression pile	Bearing pile	1.00	1.00	1.50
		Friction pile	1.00	1.00	2.00

⑤ The surface friction force on the portion of the coupled-pile anchorage above the active failure plane is not generally considered in the analysis of the bearing force of the coupled-pile anchorage when a portion of a coupled-pile anchorage is positioned above an active failure plane of a sheet pile. Furthermore, when determining the position of an anchorage in the verification of a coupled-pile anchorage by taking into consideration of pile bearing force perpendicular to pile axes, it is advisable to position the anchorage with a sufficiently safe distance behind a sheet pile in a manner that assumes the coupled-pile anchorage as a vertical pile anchorage.

(4) Examination of the Stability of Sheet Pile Anchorages

① When the sheet pile anchorage below the tie member installation point is long enough to be regarded as a long pile, the cross section of the sheet pile anchorage may be determined in accordance with **(2) Examination of the Stability of Vertical Pile Anchorages**.

② On the assumption that the earth pressure acts on a range down to $l_{m1}/2$ point below the tie member installation point (**Fig. 2.3.21**), sheet pile anchorage that cannot be regarded as a long pile may be verified in accordance with **(5) Examination of the Stability of Slab Anchorage** below. The length l_{m1} is the vertical distance from the tie member installation point to the first zero point of the bending moment of a sheet pile assuming that the sheet pile anchorage is a long pile.

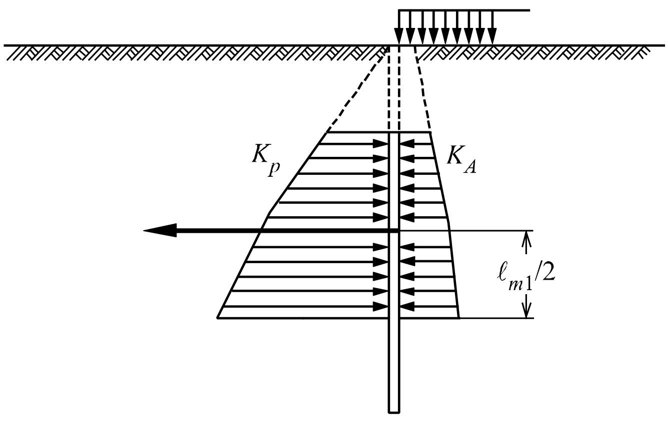


Fig. 2.3.21 Virtual Earth Pressure for Short Sheet Pile Anchorage

③ To determine if a sheet pile anchorage can be regarded as a long pile and to calculate the first zero point of the bending moment of the sheet pile anchorage, refer to the Port and Harbour Research Institute's method shown in **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**.

④ Sheet pile anchorage shall be provided with waling at the tie member installation points in a manner that equally transmits the tension force of tie members to sheet piles. For the performance verification of the waling and waling installation methods, refer to **Part III, 2.3.8 Performance Verification of Stresses in Waling**.

⑤ Short sheet pile anchorages are subjected to large bending moment at the tie member installation points; therefore, it is desirable that the stresses applied to sheet piles are calculated for cross sections without holes for installing tie members and waling fastening bolts.

⑥ When a sheet pile anchorage cannot be installed with an enough distance from the sheet pile body, a double sheet pile structure may need to be examined as an alternative to the sheet pile anchorage. In such a case, refer to **Part III, Chapter 5, 2.7 Double Sheet Pile Quaywalls**.

⑦ There have been reports indicating that the coefficients of lateral subgrade reaction of sheet piles (2D k value) are smaller than those of piles. Therefore, care should be exercised when examining the stability of sheet pile anchorage. For example, in the case of cohesive ground with c_u of 9.8 N/cm^2, the 2D k value is 14.7 N/cm^3 compared with the k value of 19.6 N/cm^3 for piles (by Yokoyama). In the case of sandy ground with an N value of 10, the 2D k value is 14.7 N/cm^3 compared with the k value of 19.6 N/cm^3 for piles.[52)]

(5) Examination of the Stability of Slab Anchorage

① The height and placing depth of slab anchorage may be determined to satisfy **equation (2.3.21)** on the assumption that the tie member tension force and the active earth pressure behind the slab anchorage are resisted by the passive earth pressure in front of the slab anchorage as shown in **Fig. 2.3.22**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 2.3.10** in which the symbol "—" in a column indicates that the value in parentheses in the column can be used for performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = Ep_k \qquad \textbf{(2.3.21)}$$

$$S_k = (A_{P_k} + E_{A_k})$$

where

E_P : passive earth pressure acting on slab anchorage (kN/m);

A_P : reaction at the tie member installation point calculated according to **Part III, 2.3.7 (5) Performance Verification of Stress on Tie Members under a Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion and the Tractive Forces Of Ships** (kN/m);

E_A : active earth pressure acting on slab anchorage (kN/m).

However, to calculate the earth pressure acting on a slab anchorage, it is normally assumed that the surcharge act shown in **Fig. 2.3.22** considers active earth pressure but not passive earth pressure.

Table 2.3.10 Partial Factors for the Stability Verification of Slab Anchorages

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stability of slab anchorage (Permanent state)	– (1.00)	– (1.00)	2.50
Stability of slab anchorage (Variable state with respect to Level 1 earthquake ground motions)	– (1.00)	– (1.00)	2.00

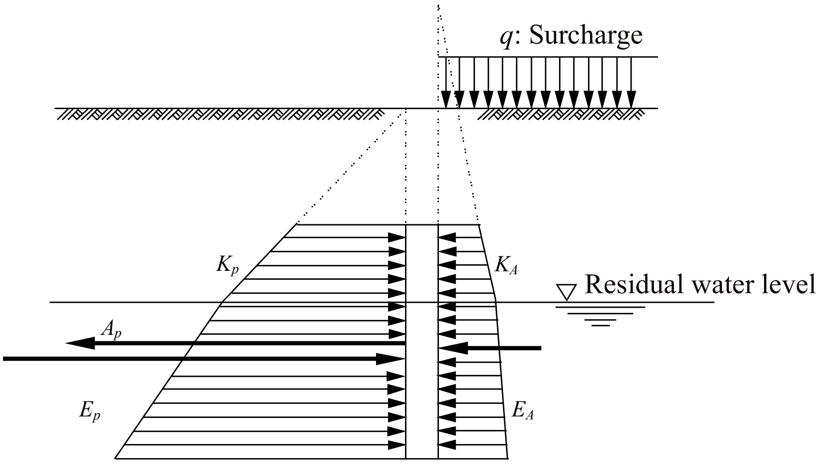


Fig. 2.3.22 Force Acting on Slab Anchorage

② For the calculation of the earth pressure acting on a slab anchorage, refer to **Part II, Chapter 4, 2 Earth Pressure**.

③ The wall surface friction angle used in calculating the earth pressure is normally assumed to be 15° in the case of active earth pressure and 0° in the case of passive earth pressure. However, in the case of a dead man anchor, an upward acting tension force acts on the slab anchorage; therefore, the wall surface friction force acts upwards, which is the opposite of the normal case of passive earth pressure. Furthermore, the passive earth pressure will be reduced. In this case the wall surface friction angle is normally assumed to be 15°.

④ There is no clear definition of a dead man anchor in terms of the inclination angle of a tie member. It has been reported that in the calculation of the sheet pile walls subjected to deformation due to the 1968 Tokachi-oki Earthquake, the calculation results by assuming the slab anchorages with tie members inclined 10° or more with respect to a horizontal plane as dead man anchors agreed well with the actual damage to the sheet pile walls.[53)]

⑤ The causes of the damage to the sheet pile walls in past earthquakes, such as the Niigata Earthquake, are mostly the insufficient resistance of slab anchorages[54)] attributable to the increase in the tension force in tie members as a result of the increase in the earth pressure around the upper part of sheet pile walls due to oscillation and the reduction in the passive resistance of slab anchorages due to the liquefaction of surface layers.

⑥ When the active failure plane of the sheet pile and the passive failure plane of the slab anchorage drawn in accordance with **Part III, Chapter 5, 2.3.3 (3) Installation Locations of Anchorages** intersect below the ground surface level, it is preferable to consider the fact that the passive earth pressure acting on the vertical surface above the intersection point does not function as a resistance force (**Fig. 2.3.23**); it should be subtracted from the design value of E_P of **equation (2.3.21)**. When the intersection point is located above the residual water level, the passive earth pressure to be subtracted may be calculated using **equation (2.3.22)**. In the following equation, subscript k indicates the characteristic value.

$$\Delta E_{P_k} = \frac{K_{P_k} w_k h_f}{2} \tag{2.3.22}$$

where

w : weight of soil (kN/m^2);

h_f : depth from the ground surface to the intersection of the failure planes (m);

K_P : coefficient of passive earth pressure.

The characteristic value w_k for the weight of soil is expressed as the product of the characteristic value for the unit weight of the soil layer under review and the depth h_f from the ground surface to the intersection of the failure planes.

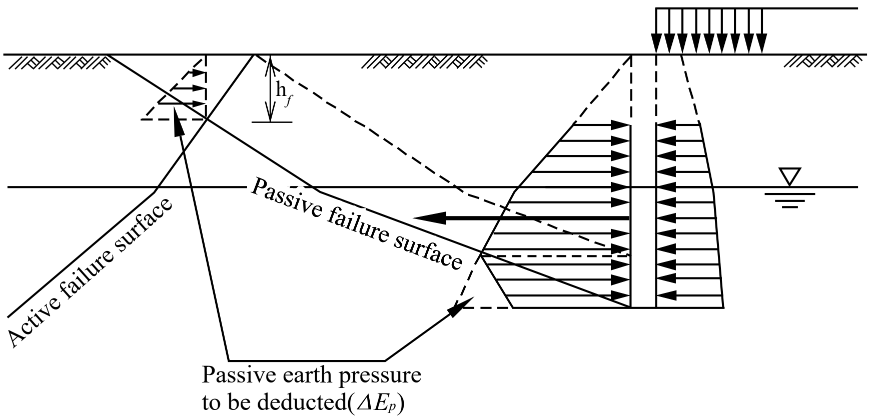


Fig. 2.3.23 Earth Pressure to Be Subtracted from the Passive Earth Pressure that Acts on Anchorage Wall when the Active Failure Plane of the Sheet Pile Wall and the Passive Failure Plane of the Slab Anchorage Intersect

⑦ When a soft cohesive soil layer exists below the area around the bottom of a slab anchorage, there is a risk that the slab anchorage does not have sufficient resistance owing to the generation of a slip surface below the lower edge of the slab anchorage. Therefore, in such a case, it is advisable to examine the stability of a slab anchorage by assuming circular or linear slip surfaces in general.

When examining circular slips, it is generally considered that a slab anchorage is unstable if the slab anchorage is positioned within a slip circle drawn on the basis of an action–resistance ratio of 1.0 or more without considering a sheet pile wall.

When examining linear slips, a slab anchorage can be determined to be stable if an action–resistance ratio with respect to the tension force of a tie member is 1.5 or lower by taking into consideration the resistance to slip of a soil mass obtainable by balancing the force acting on the soil mass defined by a slip plane passing through the lower edge of the slab anchorage, a vertical plane passing through the slab anchorage, and an active failure plane of a sheet pile wall.

⑧ Cross sections of slab anchorages

A slab anchorage should have stability against the bending moment caused by the earth pressure and the tension force of tie members. In general, the maximum bending moment may be calculated by **Equation (2.3.23)** on the assumption that the earth pressure is approximated to an equally distributed load, and the slab anchorage is a continuous slab in the horizontal direction and a cantilever slab fixed at the tie member installation point in the vertical direction. In the following equation, the subscript *k* indicates the characteristic value.

$$M_{H_k} = \frac{T_k l}{12}$$
$$M_{V_k} = \frac{T_k h}{8l} \qquad \textbf{(2.3.23)}$$

where

M_H : horizontal maximum bending moment (kN·m);

M_V : vertical maximum bending moment per meter in length (kN·m /m);

T : tie member tension according to **Part III, 2.3.7 (5) Performance Verification of Stress on Tie Members under a Permanent State and Variable State with respect to Level 1 Earthquake Ground Motion and the Tractive Forces Of Ships** (kN);

l : tie member interval (m);

h : height of slab anchorage (m).

The layout of the reinforcing bars for M_H may be determined on the assumption that the effective width of the slab anchorage is $2b$ with the tie member installation point as the center, where b is the thickness of the slab anchorage at the tie member installation point.

⑨ Slab anchorages are constructed from reinforced concrete or prestressed concrete. For the performance verification of reinforced concrete and prestressed concrete slab anchorages, refer to **Part III, Chapter 2, 2 Structural Members**.

⑩ In many cases, the installation position of a tie member on a slab anchorage is the point of resultant earth pressure or the center of the heights of slab anchorages.

2.3.10 Performance Verification of Accidental Situation with Respect to Level 2 Earthquake Ground Motion

(1) When performing the performance verification of sheet pile quaywalls that are high earthquake-resistance facilities under accidental situation with respect to Level 2 earthquake ground motions, it is necessary to examine deformation amounts via nonlinear earthquake response analysis or other methods that take into consideration the dynamic interactions of ground and structures.

(2) In sheet pile quaywalls, stress distribution in soil varies depending on construction processes, and the responses of sheet piles and anchorages during earthquakes are largely affected by the initial stress states of the ground. Therefore, it is necessary to select analysis methods that can reproduce the stress distribution in soil before being hit by earthquakes on the basis of construction processes, including the reclamation and excavation of land and the installation of sheet piles and anchorages.

(3) For the performance verification of sheet pile quaywalls against earthquake ground motions, refer to **Reference (Part III), Chapter 1, 2 Basic Items for Earthquake Response Analyses**.

(4) For the standard threshold levels used for the performance verification of deformation amounts under the accidental situation with respect to Level 2 earthquake ground motions, refer to **Part III, Chapter 5, 1.5 Points of Cautions for High Earthquake-resistance Facilities**.

(5) Calculation Method of Threshold Levels for Steel Pipe Members

The limit curvatures, which are the standard threshold levels when performing the performance verification of the damage to sheet piles and anchorages using steel pipe members under the accidental situation with respect to Level 2 earthquake ground motions, can be appropriately set by referring to **Reference (Part III), Chapter 1, 2.5.2 Pile Modeling Methods**.

It is also necessary to consider the effect of diameter–thickness ratios on the load-bearing characteristics of steel pipe members. When assuming that a beam element on the basis of the infinitesimal deformation theory has a bilinear relation between the bending moment and curvatures, the limit curvatures ϕ_u can be calculated with **equation (2.3.24)** in consideration of the diameter–thickness ratio.[56)]

$$\begin{aligned} &\text{Case of compressive axial force } (N \geq 0) && \phi_u = \mu \phi_y{}' \\ &\text{Case of tensile axial force } (N < 0) && \phi_u = \mu \phi_y \end{aligned} \qquad \textbf{(2.3.24)}$$

where

ϕ_u : limit curvature (1/mm);

ϕ_y : curvature corresponding to yield moment (1/mm);

ϕ'_y : curvature corresponding to yield moment taking into consideration the reduction in yield stress in the axial compression direction (1/mm);

μ : plasticity rate.

The limit curvatures can be calculated by multiplying the curvatures corresponding to the yield moment by the plasticity rate. However, the yield moment is subjected to the reduction in the yield stress in the axial compression direction in accordance with the diameter–thickness ratios. The plasticity rates can be calculated with **Equation (2.3.25)** for each structural type.[56)]

$$\begin{aligned} &\text{Case of steel pipe sheet pile and vertical pile anchorage} && \mu = \gamma\left(280\,t/D - 1.2\right) \\ &\text{Case of coupled-pile anchorage} && \mu = \gamma\left[\left(-5.78\,l/r + 440\right)t/D + 0.0506\,l/r - 2.55\right] \end{aligned} \qquad \textbf{(2.3.25)}$$

where

t : wall thickness (mm);

D : diameter (mm);

l : effective member length (mm);

r : radius of gyration of area (mm);

γ : correction coefficient for yield stress.

The applicability of the correction coefficients has been confirmed for yield stress up to 450 N/mm^2.[57] Furthermore, the effective member lengths that need to be set in accordance with the moment distribution of respective structural types can be appropriately set by referring to **Reference (Part III), Chapter 1, 2.5.2 Pile Modeling Methods**.

2.3.11 Performance Verification of Copings

(1) A coping may be verified as a cantilever beam that is fixed at the top of the sheet pile and subjected to the earth pressure as an action. However, it is necessary to consider the tractive forces of ships and the active earth pressure behind the wall for the parts on which bollards are installed and the fender reaction force and the passive earth pressure behind the wall for the parts on which fenders are installed. The only factor that should be considered with regard to conditions during an earthquake is the active earth pressure.

(2) The tractive forces of ships and fender reactions may be applied (**Figs. 2.3.24 (a)** and **Fig. 2.3.25 (a)**) by assuming that they are acting over the width b of the coping as shown in the figures. In this case, when considering the tractive forces, a surcharge shall be considered in the active earth pressure calculation. When applying the fender reactions, a surcharge shall not be considered in the passive earth pressure calculation. The wall surface friction angle may be taken to be 15° for active earth pressure and 0° for passive earth pressure. For the tractive forces of ships and fender reactions, refer to **Part II, Chapter 8, 2 Actions Caused by Ships**.

(3) The performance verification of copings shall be generally performed by assuming that they are constructed of reinforced concrete.

(4) Copings can be considered as beams on elastic foundations when determining the reinforcement arrangement in a horizontal direction.

(5) It is necessary to ensure the reliable transmission of bending moment acting on copings to sheet piles by sufficiently embedding the upper sections of sheet piles in the copings and by connecting the reinforcement to the sheet piles by welding.

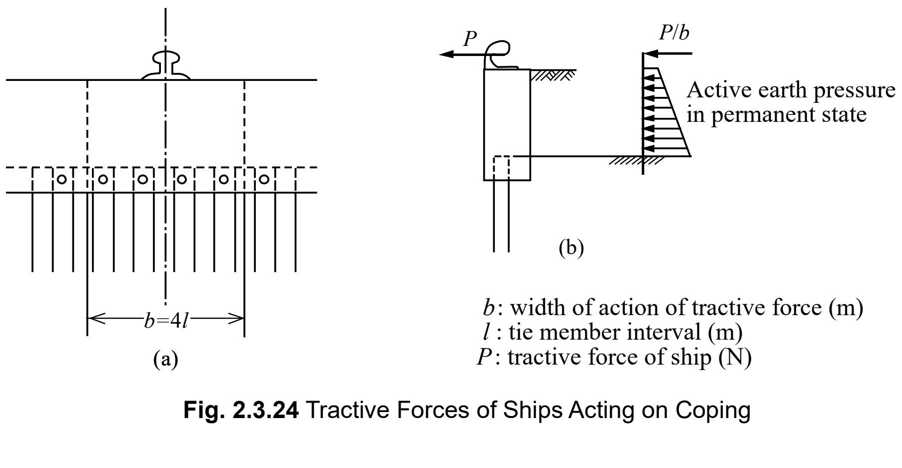


Fig. 2.3.24 Tractive Forces of Ships Acting on Coping

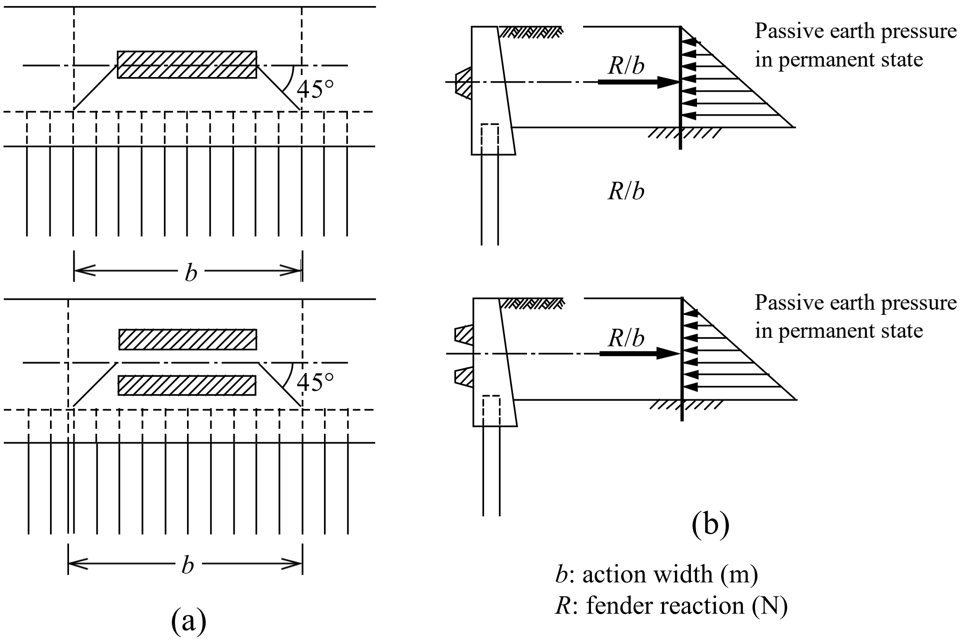


Fig. 2.3.25 Fender Reactions Acting on Coping

2.3.12 Structural Details

(1) Installation of Tie Members, and Waling on Sheet Piles

① Tie members and waling shall be installed on sheet piles so that the horizontal force acting on sheet pile walls is safely and equally transmitted to each tie member via the waling.

② The structural analyses of sheet piles, tie members, and waling are generally performed by assuming that they work integrally as a sheet pile structure. Therefore, the horizontal force acting on a sheet pile wall needs to be equally transmitted to respective tie members.

③ Tie members are generally fixed to sheet piles in a manner that allows the tie members to penetrate the sheet piles via the holes provided on the sheet piles and to be fixed to the sheet piles with washers suitable for installation angles and nuts as shown in **Part III, Chapter 5, 2.3.8 Performance Verification of Stresses in Waling** (**Fig. 2.3.20**).

④ Waling is normally installed by sandwiching tie members and is fixed to the sheet pile with bolts or similar. If waling is installed to the rear of the sheet pile, the cross section of the fastening bolts can be determined from **equation (2.3.26)**. However, if not embedded in the coping, it is necessary to consider a corrosion allowance. In the following equation, subscript k indicates the characteristic value.

$$A = m\frac{A_{p_k} l_w}{n\sigma_{y_k}} \tag{2.3.26}$$

where,

A : bolt cross-sectional area (cm^2);

A_p : reaction at tie member installation point obtained from **Part III, Chapter 5, 2.3.7 Performance Verification for the Overall Stability of Sheet Pile Walls** (N/m);

l_w : spacing of sheet pile fastened to the waling (m), when installed at one position intermediate between tie members, equivalent to a half of the tie member spacing;

n : number of bolts at one location (No.);

σ_y : tensile yield stress of bolt (N/cm^2);

m : adjustment factor.

If bolts are used, the adjustment factor may be taken to be 2.5 for permanent situations and 1.67 for variable situations with respect to the Level 1 earthquake ground motion.

(2) Tie Members

① Tie members shall safely transmit the tension force obtained in **Part III, Chapter 5, 2.3.7 (5) ④ Tension force acting on tie members**, to the anchorages. When bending stress due to the settlement of backfill soil is anticipated, tie members shall be able to cope with such bending stress.

② The performance verification of tie members shall be performed by taking into consideration the fact that tie members fulfill an important role of connecting sheet piles and anchorages and are subjected to uncertain actions.

③ Tie members are generally fixed to sheet piles in a manner that allows the tie members to penetrate the sheet piles via the holes provided on the sheet piles and to be fixed to the sheet piles with nuts installed at the tips of the tie members. The work to drill holes on sheet piles is executed after sheet piles are driven to align the holes, but it is necessary to pay attention to the difficulty in executing the hole drilling work underwater.

④ Tie rods

(a) Tie rods shall be provided with turnbuckles at their joints to make the lengths of the tie rods adjustable (**Fig. 2.3.26**).

(b) Given that tie rods have a risk of being subjected to bending stress owing to the settlement of backfill soil, they shall be provided with ring joints. Tie rods are also subjected to large bending stress at the positions where they are fixed to sheet piles and anchorages; therefore, it is advisable to install ring joints as close to the sheet piles and the anchorages as possible with half the ring joints normally embedded in coping concrete. In some cases with the risk of settlement, tie rods are supported by piles at their centers or protected by casing pipes.

(c) Given that the cross sections of tie rods are reduced when threaded, the thread sections need to have larger diameters than the other section so that the core diameters do not fall below the diameters of the other section of the tie rods.

(d) Turnbuckles, ring joints, and nuts shall not be damaged before tie rods are ruptured.

(e) The safety of turnbuckles shall be verified at the portions having the least cross-section areas.

(f) The items to be verified with respect to the interfaces between ring joints and tie rods shall ensure the safety of portions having the least cross-section areas against tension force, the upper and lower faces of pin holes against shear force applied through pins, and the pins against double shear.

(g) For the threaded portions of nuts and turnbuckles, the safety of the roots of threads against shear force shall be verified with cross-sectional force acting on them multiplied by the safety factor of 1.1 to 1.2 by taking into consideration the possible stress concentration.

(h) For the tensile and shear yield stress of steel materials, refer to **Part II, Chapter 11, 2 Steel Materials**.

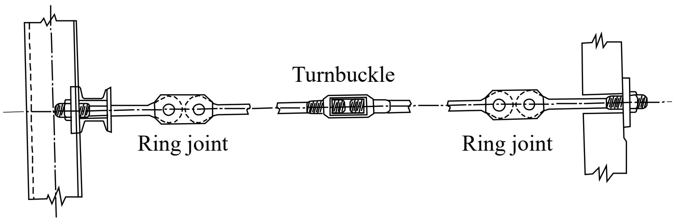
⑤ Tie wires

(a) With the compression sections on both ends directly threaded, a tie wire is configured to function as a turnbuckle. Therefore, tie wires need to be verified by taking into consideration the fixation lengths. Furthermore, similar to the case with turnbuckles used for tie rods, performance verification shall be made with respect to the portions with the least cross-section areas.

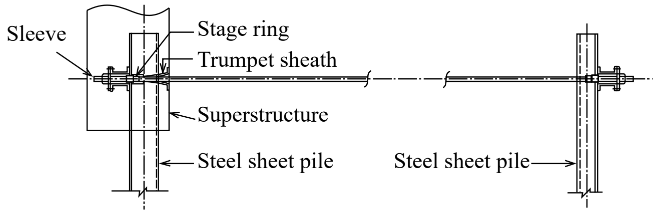
(b) When constructing coping, the end sections of tie wires shall be embedded in the coping concrete with attention not to cut sleeves.

(c) When tie wires need to be crossed at corner sections, tie wires should be prevented from coming into contact with each other by calculating the sags of tie wires.

⑥ For the materials and accessories of tie wires, refer to the Guideline for **Construction of Steel Sheet Piles**[55] (The Ports and Harbours Association of Japan).



(a) Tie rod



(b) Tie wire

Fig. 2.3.26 Installation of Tie Members

(3) Installation of Anchorages and Tie Members

① The interfaces between anchorages and tie members are structurally important. Therefore, it is necessary to ensure that the tension force in tie members calculated in accordance with **Part III, Chapter 5, 2.3.7 (5) ④ Tension force acting on tie members** can be safely and equally transmitted to anchorages.

② A continuous beam along the face line of a quaywall is usually constructed on top of a pile anchorage, and the tie members are attached to the beam. This beam may be verified for performance as a continuous beam subjected to the tie member tension force and the reaction force of the piles.

③ Tie members are generally fixed to a slab anchorage or the beam on top of pile anchorages in a manner that allows the tie members to penetrate the slab or beam via the holes provided on them and to be fixed to them with washers suitable for installation angles and nuts. The slab thicknesses and the sizes of washers need to be verified on the basis of the fact that the positions on the slabs or beams where the tie members are installed are subjected to bearing and punching stress. Furthermore, it is advisable that the slab has distribution reinforcement in the area around the tie member installation positions to enable the tension force in tie members to be distributed to the slab.

④ Refer to **Part III, Chapter 5, 2.3.12 (1) Installation of Sheet Piles, Tie Members, and Waling** for the installation of tie members to sheet pile anchorages.

(4) Items Related to the Joints of Steel Sheet Piles

① The types of joints of steel sheet piles are as follows (refer to **Fig. 2.3.27**). For the performance verification of the joint sections, it can be determined that the joint sections satisfy predetermined requirements as long as proven sheet piles walls are used. However, appropriate performance verification is required when adopting new types of joints.

(a) Hook-type joint

(b) Male–female type joint

Hook-type joints are used for hat-shaped and U-shaped sheet piles, and the male–female type joints (①, ②, and ③ in the figure) are used for steel pipe sheet piles.

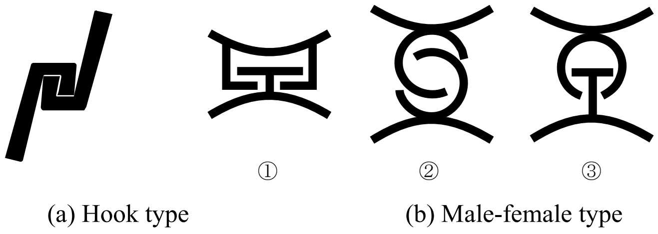


Fig. 2.3.27 Shapes of Steel Sheet Pile Joints

② The joint length of steel sheet piles should be as long as possible from the point of view of maintaining the integrity of the sheet piles. However, by taking into consideration the damage to joints during construction, the joints do not normally extend to the bottoms of the sheet piles. Normally, the bottom end of the joint is at the depth where the active earth pressure strength and the passive earth pressure strength are equal or is continuous to the virtual fixity point ($1/\beta$; refer to the virtual fixing point shown in **Part III, Chapter 5, 5.2.2 Setting of Basic Cross-section, (8) Virtual Fixed Point**) and is frequently located 2 to 3 m below the seabed surface. If the residual water level difference is large, the joint length of steel sheet piles should be determined by taking the piping phenomenon into account. The top end of the joint is often extended up to 30 to 40 cm above the bottom surface of the coping.

③ When a U-shaped steel sheet pile is subjected to bending, there is a possibility that vertical slip will occur at joints located at the center of the wall. In this case, the U-shaped steel sheet piles will not act integrally with the adjacent sheet piles. In this situation, the section modulus and geometrical moment of inertia of the cross section calculated by assuming that steel sheet piles act integrally in the wall may not be obtained. The methods for evaluating the effect of this slip in the joints include the method of reducing cross-section performance by multiplying by a joint efficiency coefficient.

(5) Corner Section

① Corner sections are subjected to complex action conditions and require full attention when verifying their performance. Furthermore, corner sections require attention in that they are particularly vulnerable to damage during the action of earthquake ground motions, i.e., they are structural weak points.

② The corner sections of sheet pile quaywalls are particularly vulnerable to damage during the action of earthquake ground motions and require sufficient reinforcement measures. For the performance verification of corner sections, refer to **Part III, Chapter 5, 9.20 Installation Sections**.

③ Special attention is required when determining the structures and installation positions of anchorages because of the possibility that the passive earth pressure regions of anchorages interfere with each other or overlap with the active earth pressure regions of sheet pile walls. Construction should also be performed in a manner that ensures the sufficient compaction of the reclaimed areas around the corner sections.

2.4 Cantilevered Sheet Pile Quaywalls

[Public Notice] (Performance Criteria of Sheet Pile Quaywalls)

Article 50

2 In addition to the provisions in the preceding paragraph, the performance criteria for cantilevered sheet piles shall indicate that the risk in which the amount of deformation of the top of the pile may exceed the allowable limit of deformation is equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating actions are Level 1 earthquake ground motion, ship berthing, and traction by ships.

[Interpretation]

11. Mooring Facilities

(4) Performance Criteria of Sheet Pile Quaywalls

② **Cantilevered sheet pile quaywalls** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance on Criteria and the interpretation related to Article 50, Paragraph 2 of the Public Notice)

(a) The performance criteria and commentary for cantilevered sheet pile quaywalls shall conform to those for sheet pile quaywalls, except the items related to tie member and anchorages, and the following provisions:

(b) The required performance of cantilevered sheet pile quaywalls under the permanent action situation in which the dominant action is earth pressure and the variable action situation in which the dominant actions are Level 1 earthquake ground motions and traction by ships shall focus on serviceability. **Attached Table 11-11** shows the performance verification items and standard indexes used for determining the limit values with respect to the actions.

Attached Table 11-11 Performance Verification Items and Standard Indexes Used for Determining the Limit Values under the Respective Design Situations of Cantilevered Sheet Pile Quaywall

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index to determine the limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="2">26</td><td rowspan="2">1</td><td rowspan="2">2</td><td rowspan="2">50</td><td rowspan="2">2</td><td rowspan="2">–</td><td rowspan="2">Serviceability</td><td>Permanent</td><td>Earth pressure</td><td>Water pressure, surcharge</td><td rowspan="2">Deformation of the normal line</td><td rowspan="2">Residual deformation amount of the crown of the quaywall</td></tr>
<tr><td>Variable</td><td>Level 1 earthquake ground motions [Traction of ships]</td><td>Earth pressure, water pressure, surcharge</td></tr>
</table>

* [ ] indicates the alternative dominant action to be studied as design situations.

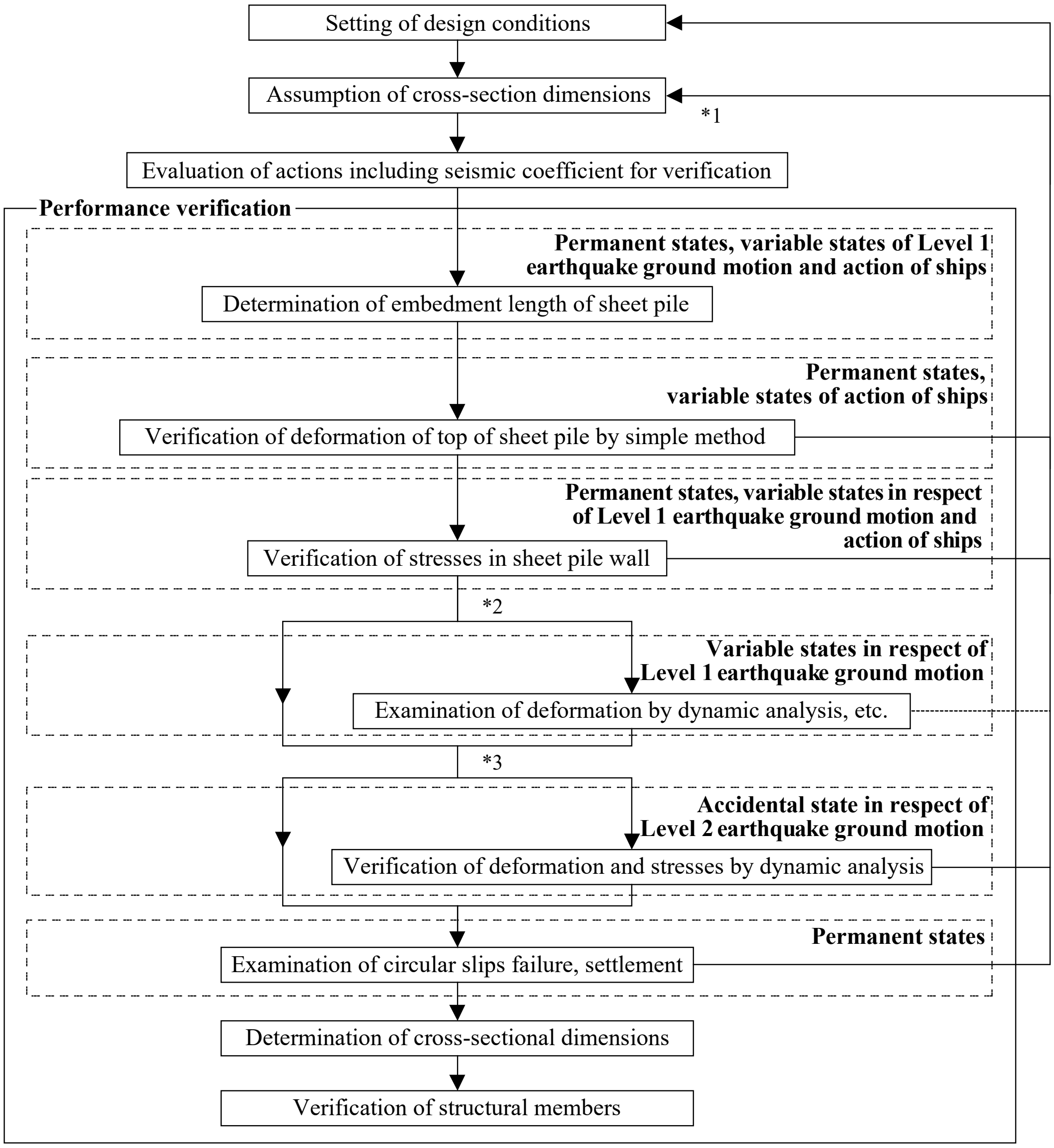
(c) In addition to the above, the requirements and the commentaries in Paragraph 3 (Scouring and Wash Out), Article 22 and Article 28 (Performance Criteria of Armor Stones and Blocks) shall be applied as needed.

2.4.1 General

(1) The descriptions in this section can be applied to the performance verification of mooring facilities that use cantilevered sheet pile walls to support the soil behind them.

(2) The performance verification methods described here are applicable to sheet pile walls driven into sandy soil ground but not to those driven into cohesive soil ground. There have been many unknown factors in the performance verification methods for cantilevered sheet pile walls driven into clayed soil ground. Furthermore, from an engineering viewpoint, because the structure of cantilevered sheet pile walls is susceptible to creep, the application of cantilevered sheet pile walls to clayed soil ground is preferably avoided.

(3) **Fig. 2.4.1** shows an example of the sequence of the performance verification of cantilevered sheet pile quaywall. However, it should be noted that **Fig. 2.4.1** does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions. Thus, regarding liquefaction, the possibility of and countermeasures against liquefaction shall be appropriately deliberated with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the performance of cantilever sheet pile walls under a variable action situation with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method on the basis of the static equation of equilibrium. However, for high earthquake-resistance facilities, it is preferable that the deformation is deliberated by a nonlinear seismic response analysis or other methods that take into consideration the dynamic interaction between the ground and structures. For cantilevered sheet pile quaywall other than high earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.



*1: Evaluation of the effect of liquefaction is not shown, so it is necessary to consider these separately.
*2: When necessary, an examination of the amount of deformation by dynamic analysis can be carried out for Level 1 earthquake ground motion.
For high earthquake-resistance facilities, it is preferable that the amount of deformation be examined by dynamic analysis.
*3: Verification in respect of Level 2 earthquake ground motion is carried out for high earthquake-resistance facilities.

Fig. 2.4.1 Example of the Sequence of the Performance Verification of Cantilevered Sheet Pile Quaywall

(4) Cantilever sheet pile walls have a structure that resists the earth and water pressure acting on the rear faces of sheet piles with the horizontal subgrade reaction at the embedded sections of the sheet piles. The flexural moment on cantilevered sheet pile walls can be calculated with reference to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(5) Sheet piles of cantilevered sheet pile quaywall are subjected to a severely corrosive environment. So the sheet piles shall be designed with appropriate corrosion protection measures on the basis of **Part III, Chapter 2, 1.3.4 Examination of Performance Deterioration with Age**.

(6) **Fig. 2.4.2** shows an example of a cross section of a cantilevered sheet pile quaywall.

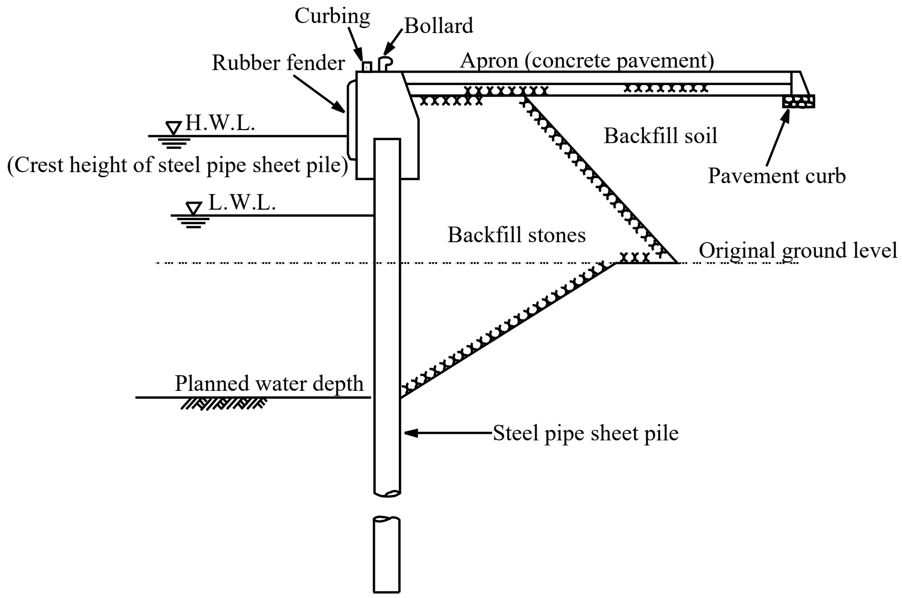


Fig. 2.4.2 Example of the Cross Section of Cantilevered Sheet Pile Quaywall

2.4.2 Actions

(1) Types of Actions to Be Considered in Respective Design Situations

For the types of actions to be considered in the stability verification of cantilevered sheet pile quaywall, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(2) Points of Caution When Setting Actions

① In the case of sandy seabed ground, the earth pressure and residual water pressure is generally considered to act on a cantilever sheet pile at a point above a virtual sea bottom (**Fig. 2.4.3**) where the sum of the active earth pressure and the residual water pressure is equal to the passive earth pressure.

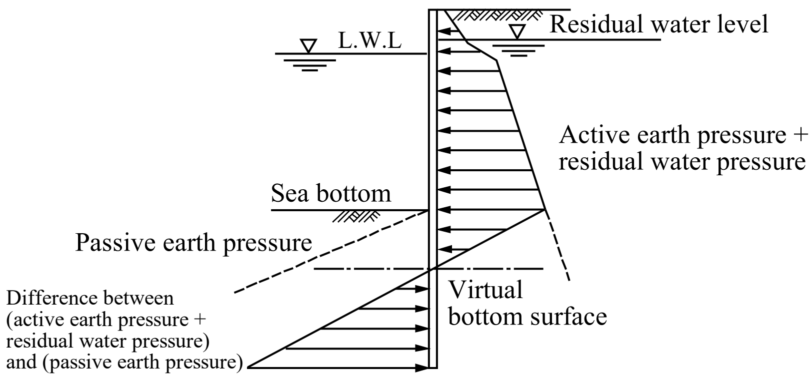


Fig. 2.4.3 Virtual Sea Bottom

② In the seabed ground just below a seafloor surface, the sum of the active earth pressure and the residual water pressure at the rear face of a sheet pile wall is larger than the passive earth pressure on the front face of the sheet pile wall. Therefore, the soil near the seafloor surface in front of the sheet pile wall has a risk of undergoing plastic deformation and will not produce elastic earth pressure, which acts as spring reaction force of the ground. Thus, with a plane called a virtual sea bottom, wherein the sum of the active earth pressure and the residual water pressure is equal to the passive earth pressure, as a boundary, the differential pressure obtained by subtracting the passive earth pressure from the sum of the active earth pressure and the residual water pressure can be considered to act on the portion of the sheet pile wall above the virtual sea bottom, and only the spring reaction force of the ground can be considered to act on the portion of the sheet pile wall below the virtual sea bottom without considering the active earth pressure acting on the rear face of the sheet pile. For performance verification during the action of earthquake ground motions, the dynamic water pressure acting on the portion of the sheet pile wall above the sea bottom should be considered.

③ **Equation (2.4.1)** can be used to calculate the characteristic value of the seismic coefficient to be used in the performance verification of cantilevered sheet pile quaywall under the variable action situation with respect to Level 1 earthquake ground motions.[58)] In this equation, the subscript k represents the characteristic value. For the filter and reduction coefficient required when reflecting the frequency characteristics into the calculation of the maximum acceleration α_c in the equation, refer to **Reference (Part III), Chapter 1, 1 Detailed Items about Seismic Coefficient for Verification**.

$$k_{h_k} = 1.40\left(\frac{D_a}{D_r}\right)^{-0.86} \cdot \frac{\alpha_c}{g} + 0.06 \qquad \textbf{(2.4.1)}$$

where

k_h : seismic coefficient for verification;

D_a : allowable deformation amount (20 cm);

D_r : reference deformation amount (10 cm);

g : gravitational acceleration (980 cm/s^2);

α_c : corrected maximum acceleration on a ground surface (cm/s^2).

④ The maximum flexural moment in a sheet pile wall shall be calculated appropriately by using an analysis method that corresponds to the mechanical behavior characteristics of the sheet pile wall. The maximum flexural moment in a sheet pile wall is normally calculated in accordance with **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**.

⑤ For the calculation of the lateral resistance of a pile, refer to **Part III, Chapter 2, 3.4.6 Deflection of Piles Receiving Force Perpendicular to Axes**.

⑥ Unlike the actions on a pile, those on a cantilevered sheet pile wall are distributed loads; therefore, the maximum flexural moment cannot be expressed by a simple equation. In the calculation of the maximum flexural moment, the distributed loads acting on a sheet pile wall can be replaced with concentrated loads acting on the center of gravity of the distributed loads.

2.4.3 Performance Verification

(1) Performance Verification of the Stress in Sheet Pile Walls

① When steel pipes are used as sheet piles, secondary stress often develops in the steel pipes of a sheet pile wall owing to the deformation of the steel pipe cross section (i.e., a circular cross section is deformed into an elliptic one) due to the earth and residual water pressures. Cantilevered sheet pile walls are structures that tend to experience large displacement, and there is a risk that a relatively high secondary stress may develop in the areas around the point where the flexural moment becomes maximum. A larger steel pipe diameter leads to a higher level of secondary stress. Therefore, in such a case, it is preferable to perform an examination of strength against the secondary stress. The secondary stress of a steel pipe is calculated using **equation (2.4.2)**:

$$\sigma_t = \alpha p \left(\frac{D}{t}\right)^2 \times 10^{-3} \qquad \textbf{(2.4.2)}$$

where

σ_t : secondary stress (N/mm^2);

p : earth pressure and residual water pressure acting on the sheet pile wall (kN/m^2);

D : diameter of a pile (mm);

t : plate thickness of a pile (mm);

α : coefficient.

The coefficient α in the equation may be defined with reference to **Fig. 2.4.4** by taking into consideration the width of action, foundation conditions, and constraint conditions.[59] In this figure, "Sliding" and "Fixed" indicate the displacement conditions of the joints of steel pipe sheet piles. "Fixed" assumes that the joints of steel pipe sheet piles are subjected to a filling treatment with concrete or other materials.

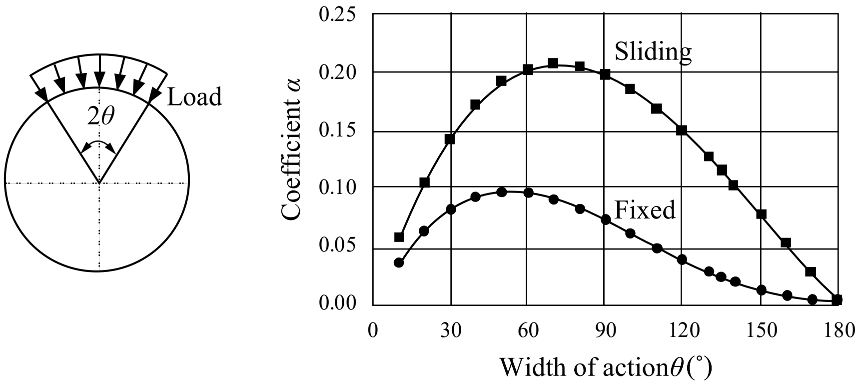


Fig. 2.4.4 Coefficient α for Secondary Stress

② The verification of stress may be performed using **equation (2.4.3)** on the basis of the axial stress σ_l in the pile obtained in accordance with **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles** and the secondary stress σ_t obtained from **equation (2.4.2)**. In the following equation, subscripts k and d indicate the characteristic value and the design value, respectively. Furthermore, the partial factors in the equation can be selected from **Table 2.4.1**. If a corresponding column in the table has the symbol "—," the value in parentheses in the column can be used for performance verification for convenience. Here, it is necessary to use positive secondary stress when the flexural yield stress is negative and negative secondary stress when the flexural yield stress is positive.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_s S_k$$

$$R_k = f_{yk} / \gamma_m \tag{2.4.3}$$

$$S_k = \gamma_b \sqrt{\sigma_{lk}{}^2 + \sigma_{tk}{}^2 - \sigma_{lk}\sigma_{tk}}$$

where

σ_l : axial stress of a pile (N/mm^2);

σ_t : secondary stress of a pile (N/mm^2);

f_{yk} : yield stress of a pile (N/mm^2);

γ_m : material coefficient (1.05);

γ_b : member coefficient (1.1);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 2.4.1 Partial Factors Used for the Verification of Stress in Sheet Pile Walls

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Stress in sheet pile wall (Permanent state)	– (1.00)	– (1.00)	1.20
Stress in sheet pile wall (Variable state of Level 1 earthquake ground motions)	– (1.00)	– (1.00)	1.00

(2) Performance Verification of the Embedded Lengths of Sheet Piles

The embedded lengths of sheet piles shall be equal to or longer than the effective length of piles calculated in accordance with **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**. Considering that cantilevered sheet piles retain earth behind them on the basis of the same mechanism as piles do, the embedded lengths of sheet piles can be calculated in the same way as that for piles. When using the PHRI method on the basis of the lateral resistance of piles in the calculation of embedded lengths, the required embedded length is expressed as 1.5 l_{m1}, where l_{m1} is the depth of the first point of zero flexural moment on a free head pile. In such a case, it should be noted that the embedded length needs to be measured not from a seafloor surface but from the virtual bottom surface.

(3) Performance Verification of the Displacement Amounts of the Tops of Sheet Pile Walls

① There are many unknown factors on the deformation of cantilevered sheet pile walls during the action of earthquake ground motions. Considering that there may be cases wherein the method described in this paragraph produces different results from those of accurate dynamic analyses, it is preferable that dynamic analyses are used for the performance verification of the displacement amounts of the tops of sheet pile walls during earthquakes.

② The displacement amount δ of the tops of a sheet pile wall can be expressed by the sum of the following three values (refer to **Fig. 2.4.5**):

(a) The deflection amount δ_1 of a sheet pile wall at a virtual bottom surface

(b) The deflection amount δ_2 of the portion of the sheet pile wall above the virtual bottom surface

(c) The deflection amount δ_3 of the top of the sheet pile wall generated as a result of the rotation of the portion of the sheet pile wall above the virtual bottom surface corresponding to the deflection angle of the sheet pile wall at the virtual bottom surface

The deflection amounts δ_1 and δ_3 above can be generally calculated by the PHRI method described in **Part III, Chapter 2, 3.4.8 Pile Deflection Calculation by the PHRI Method**. Furthermore, the deflection amount δ_2 is generally calculated as the deflection of a cantilever beam subjected to earth pressure behind the beam.

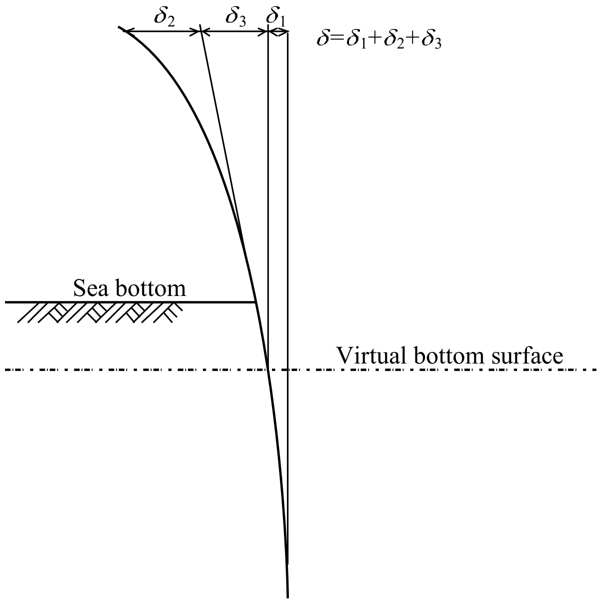


Fig. 2.4.5 Displacement Amount of the Tops of a Sheet Pile Wall

③ The deflection amount of the top of a sheet pile wall is the displacement from a state with no load applied to the sheet pile wall. Therefore, the displacement amount of the top of a sheet pile due to the surcharge after quaywall completion or the earth pressure during the action of earthquake ground motions is preferably calculated in a manner that applies such a surcharge or earth pressure to a sheet pile wall subjected to residual deflection.

④ In the calculation of the deflection amount δ_2 with a sheet pile wall assumed as a cantilever beam, the earth pressure acting on the beam can be simplified as a triangular distribution load, with the resultant earth pressure equivalent to that of the earth pressure illustrated in **Fig. 2.4.6** for convenience.

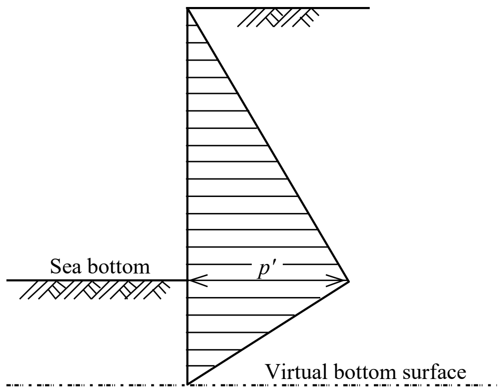


Fig. 2.4.6 Assumption of Earth Pressure

⑤ The deformation amounts of the tops of cantilevered sheet pile quaywalls shall be kept to a level that does not interfere with the use of the quaywalls. According to the past design examples, the deformation amounts of the tops of cantilevered sheet pile quaywalls have often been set at approximately 5 cm under the permanent action situation.

(4) Performance Verification of Copings

The performance verification of the superstructures of cantilevered sheet pile quaywalls can refer to **Part III, Chapter 5, 2.3.11 Performance Verification of Copings**.

(5) Examination of the Actions during Construction

The performance verification of cantilevered sheet pile quaywalls shall take into consideration the stability of the quaywalls against the actions during construction. During construction, cantilevered sheet piles are vulnerable to landward actions because there is no backfill to resist such actions. Therefore, when planning to construct cantilevered sheet pile quaywall by driving sheet pile walls into seabed ground, it is necessary to ensure that they have structures that are capable of sufficiently resisting actions, such as high waves, which are expected to occur during construction periods and may place the sheet pile walls into unstable states.

2.4.4 Performance Verification of Structural Members

For the performance verification of the structural members of cantilevered sheet pile quaywalls, refer to **Part III, Chapter 5, 2.3.12 Structural Details**.

2.5 Sheet Pile Quaywalls with Raking Pile Anchorages

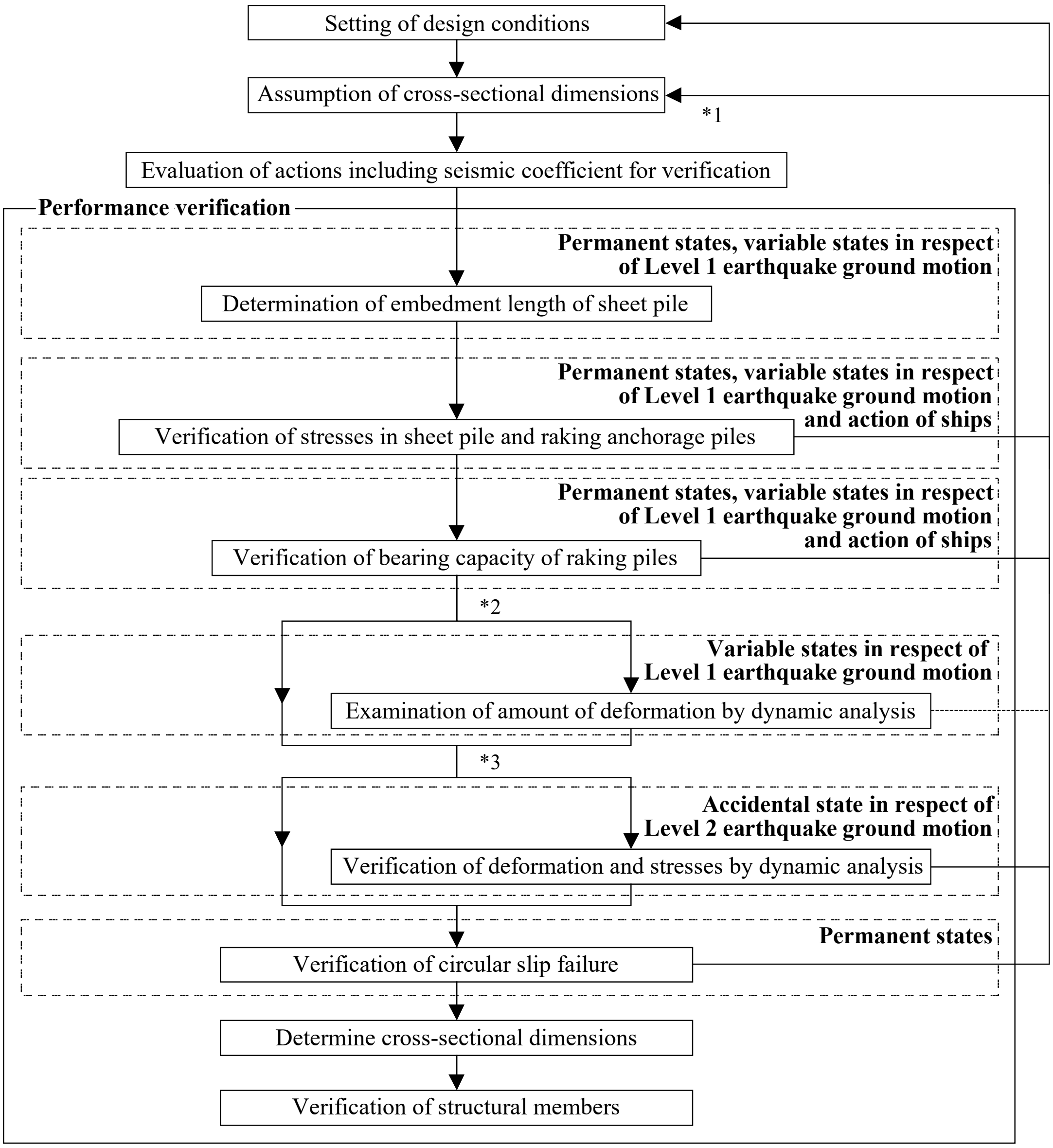
2.5.1 General

(1) The following descriptions are applicable to the performance verification of mooring facilities in which raking piles are driven behind the sheet pile walls and in which the tops of the sheet pile walls and the raking piles are connected to support the soil behind the sheet pile walls.

(2) **Fig. 2.5.1** shows an example of the sequence of the performance verification of sheet pile quaywall with raking pile anchorages. However, **Fig. 2.5.1** does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions. Thus, regarding liquefaction, the possibility of and countermeasures against liquefaction shall be appropriately deliberated with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the performance of sheet pile walls with raking pile anchorages under a variable action situation with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method based on the static equation of equilibrium. However, for high-earthquake-resistance facilities, the deformation should be deliberated by nonlinear seismic response analysis or other methods that take into consideration the dynamic interaction between the ground and structures. For sheet pile quaywall with raking pile anchorages other than high-earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.

(3) Considering that the sheet piles of sheet pile quaywall with raking pile anchorages are subjected to severely corrosive environments, the sheet piles shall be designed with appropriate corrosion protection measures on the basis of **Part III, Chapter 2, 1.3.4 Examination of Performance Deterioration with Age**.

(4) **Fig. 2.5.2** shows an example of a cross section of a sheet pile quaywall with a raking pile anchorage.



*1: As the effects of liquefaction are not shown, it is necessary to consider these separately.

*2: When necessary, an examination of the amount of deformation by dynamic analysis can be carried out for the Level 1 earthquake ground motion.
For high earthquake-resistance facilities, it is preferable that the examination of the amount of deformation be carried out by dynamic analysis.

*3: Verification in respect of Level 2 earthquake ground motion is carried out for high earthquake-presistance facilities.

Fig. 2.5.1 Example of the Sequence of the Performance Verification of Sheet Pile Quaywall with Raking Pile Anchorages

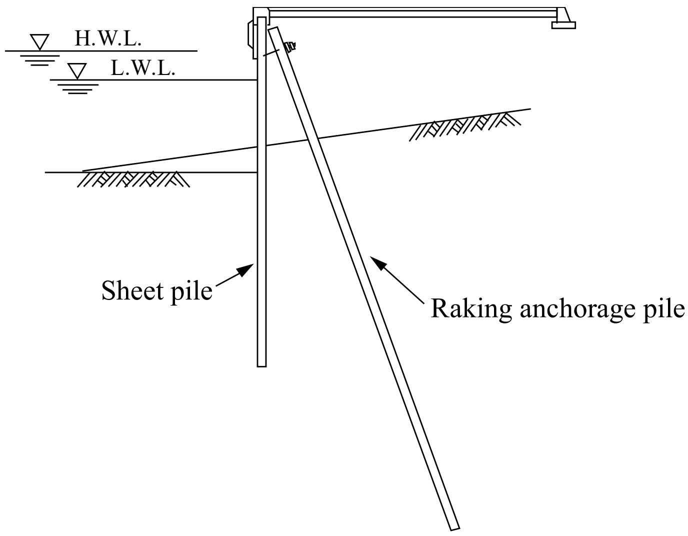


Fig. 2.5.2 Example of the Cross Section of a Sheet Pile Quaywall with Raking Pile Anchorage2.5.2 Actions

2.5.2 Actions

(1) Types of Actions to Be Considered in Respective Design Situations

For the types of actions to be considered in the stability verification of a sheet pile quaywall with raking pile anchorages, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(2) Points of Caution When Setting Actions

① The characteristic values of the seismic coefficients for verification to be used in the performance verification of sheet pile quaywalls with raking pile anchorages under the variable action situation with respect to Level 1 earthquake ground motions shall be appropriately calculated by taking into consideration the structural characteristics of the quaywalls. Here, the characteristic values of the seismic coefficient can be calculated for convenience in accordance with the equation for calculating the seismic coefficient for verification with respect to sheet pile quaywalls with vertical pile anchorages (i.e., **equation 2.3.2 (b)**) in **Part III, Chapter 5, 2.3.4 Actions**.

2.5.3 Performance Verification

(1) It is preferable that the performance verification of sheet pile quaywalls with raking pile anchorages is performed by accurate methods (such as model experiments or reliable numerical analysis methods); however, the performance verification method described in this section can be used as a simplified performance verification method.

(2) The performance verification methods proposed for sheet pile walls with raking pile anchorages include Ishiguro's formula[60] and Oshima's formula,[61] which are classified as simplified performance verification methods. The performance verification of quaywalls during the actions of earthquake ground motions is preferably performed by detailed methods, such as dynamic analyses.

(3) In Ishiguro's formula, the flexural moment and axial force are theoretically calculated on the assumptions that the distance between a sheet pile wall and a raking anchor pile does not change and that the embedded sections of the sheet pile wall and the raking anchor pile are beams placed on elastic supports. In Oshima's formula, the maximum flexural moment and axial force are calculated on the assumptions that earth pressure is equally distributed between a sheet pile wall and a raking anchor pile and that the sheet pile and the ranking anchor pile are beams with respective first fixed points defined as anchorage points. A published report compared the calculation results by using two formulas with the actual stress and earth pressure acting on existing sheet pile walls and raking anchor piles measured on their completion.[62] The report indicates that the calculated stress in sheet pile walls and raking anchor piles correspond relatively well to the measured stress. However, the distribution ratio of earth pressure still needs to be reconsidered because the measured earth pressure acting on sheet piles is remarkably lower than that acting on the raking anchor piles.

(4) Verification of the Stress in Sheet Piles and Raking Anchor Piles

① For sheet pile quaywall with raking pile anchorages, verification may be performed for the resistance of the sheet pile and the piles against the actions in the horizontal and vertical directions at the connection point, earth pressure, and residual water pressure.

② The horizontal and vertical forces acting on the connection point between a sheet pile and a raking pile can be calculated by assuming that the connection is a pin structure.

(5) Determination of the Embedded Lengths of Sheet Piles and Raking Anchor Piles

The embedded length of the sheet pile or raking anchorage pile that is required to resist the forces acting in the axial direction and the direction perpendicular to the axis can be calculated in accordance with **Part III, Chapter 2, 3.4 Pile Foundations**. However, the bearing capacity in the axial direction of the sheet pile and that of the raking anchorage pile is preferably examined via loading and pulling tests.

2.5.4 Performance Verification of Structural Members

The performance verification of sheet pile quaywalls with raking pile anchorages can be performed in accordance with the performance verification of sheet pile quaywalls and open-type wharves on vertical piles with reference to **Part III, Chapter 5, 2.3.12 Structural Details** and **Part III, Chapter 5, 5.2.6 Performance Verification of Structural Members**.

2.6 Open-type Quaywall with Sheet Pile Wall Anchored by Forward Batter Piles

2.6.1 General

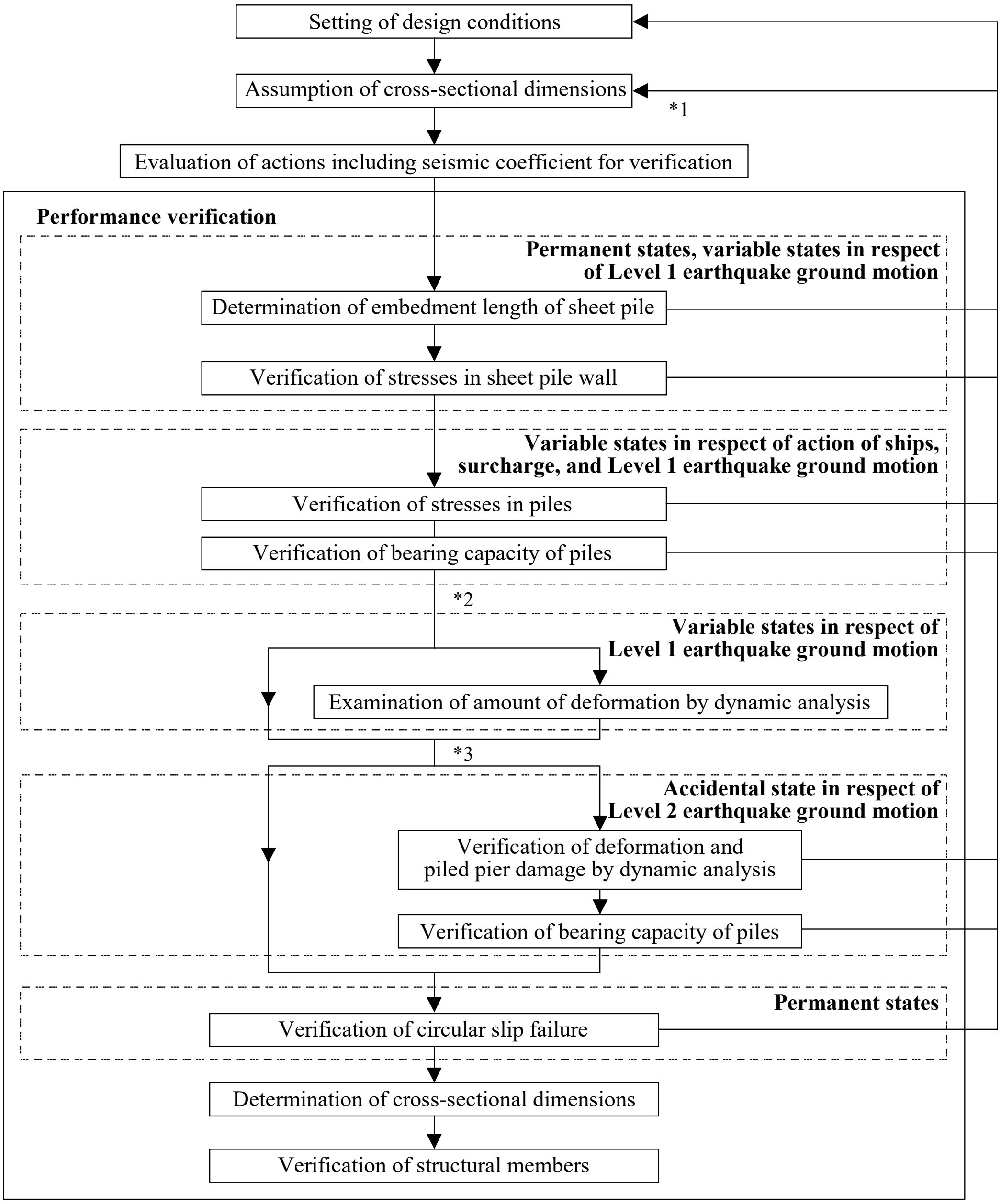
(1) The provisions in this section shall be applied to the performance verification of mooring facilities that retain the earth pressure behind them by sheet pile walls and raking anchor piles driven in front of the sheet pile walls with respective top sections coupled together.

(2) An open-type quaywall with sheet pile wall anchored by forward batter piles is normally constructed with open-type wharves in front of the sheet pile walls. The open-type wharf may be integrated into or separated from the sheet pile walls. This section provides guidelines for cases in which open-type wharves and sheet pile walls are integrated. For cases in which open-type wharves are separated from sheet pile walls, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**; **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**; and **Part III, Chapter 5, 5.3 Open-type Wharves on Coupled Raking Piles**. The performance verification method described in this section is based on the equivalent beam method applied to the performance verification of sheet piles. Therefore, the structural types covered by this section are steel sheet pile walls driven into sandy soil ground or hard clayed soil ground.

(3) **Fig. 2.6.1** shows an example of the sequence of the performance verification of an open-type quaywall with sheet pile wall anchored by forward batter piles. However, **Fig. 2.6.1** does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions. Thus, regarding liquefaction, the possibility of and countermeasures against liquefaction shall be appropriately deliberated with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the performance of open-type quaywall with sheet pile wall anchored by forward batter piles under a variable action situation with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method based on the static equation of equilibrium. However, for high earthquake-resistance facilities, the performance verification should be performed by nonlinear seismic response analysis or other methods that take into consideration the 3D dynamic interaction between piles and the ground. For open-type quaywall with sheet pile wall anchored by forward batter piles other than high earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.

(4) An open-type quaywall with sheet pile wall anchored by forward batter piles is subjected to severe environments and have structures that are susceptible to the performance reductions of members due to material degradation such as chloride-induced corrosion to concrete members and corrosions of steel pipe piles. Therefore, open-type quaywalls with sheet pile wall anchored by forward batter piles can be designed in accordance with open-type wharves on vertical piles with reference to **Part III, Chapter 5, 5.2.1 General**.

(5) **Fig. 2.6.2** shows an example of the cross section of an open-type quaywall with a sheet pile wall anchored by forward batter piles.



*1: The evaluation of the effect of liquefaction is not shown because it is necessary to consider these separately.

*2: When necessary, an examination of the amount of deformation by dynamic analysis can be performed for the Level 1 earthquake ground motion.
For high earthquake-resistance facilities, the examination of the amount of deformation should be performed by dynamic analysis.

*3: Verification with respect to Level 2 earthquake ground motion is performed for high earthquake-resistance facilities.

Fig. 2.6.1 Example of the Sequence of the Performance Verification of Open-type Quaywall with Sheet Pile Wall Anchored by Forward Batter Piles

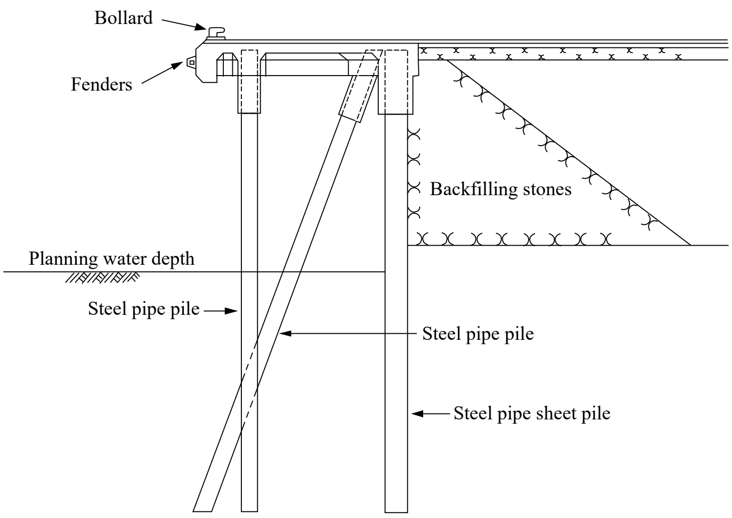


Fig. 2.6.2 Example of Cross-section of Open-type Quaywall with Sheet Pile Wall Anchored by Forward Batter Piles

2.6.2 Layouts and Dimensions

(1) For the size of one block of a superstructure and the pile layout of an open-type wharf, refer to the size of open-type wharf block and pile layouts in **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

(2) It is preferable that the layouts and inclinations of batter piles are determined with due consideration given to their positional relationship with other piles and construction work-related constraints such as those concerning the capacity of pile driving equipment. A pile inclination of approximately 20° is normally used for batter piles.

(3) For the dimensions of the superstructures, refer to the dimensions of superstructures in **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

2.6.3 Actions

(1) For the actions on open-type wharves, refer to **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

(2) For the actions of the sheet piles, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(3) The self-weight of the reinforced concrete of the superstructures of open-type wharves can be calculated with a unit weight of 21 kN/m^2 in the performance verification of piles and sheet piles in accordance with **Part III, Chapter 5, 5.3 Open-type Wharves on Coupled Raking Piles**.

(4) The fender reaction force can be calculated using the calculation methods described in **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

(5) The characteristic values of the seismic coefficients for verification used in the performance verification of an open-type quaywall with a sheet pile wall anchored by forward batter piles for the variable action situation with respect to Level 1 earthquake ground motions shall be appropriately calculated by taking the structural characteristics into consideration. For convenience, the characteristic values of the seismic coefficients for verification used in the performance verification of open-type quaywalls with sheet pile wall anchored by forward batter piles may be calculated using **equation (2.3.2 (a))** in **Part III, Chapter 5, 2.3.4 (2) Points of Caution When Setting Actions** for sheet pile walls and in accordance with **Part III, Chapter 5, 5.2 Open-type Wharf on Vertical Piles** for open-types wharves.[63)]

2.6.4 Performance Verification

(1) The performance verification of open-type quaywalls with sheet pile wall anchored by forward batter piles should be performed by using accurate methods (such as model experiments or reliable numerical analysis methods).[63), 64)] Given that an open-type quaywall with sheet pile wall anchored by forward batter piles has a structure that allows sheet pile walls to retain earth pressure behind them, the sheet pile walls are subjected to deformation during the action of earthquake ground motions. Therefore, the performance verification methods should be able to

appropriately evaluate the effects of the deformation. The following methods are simplified ones that are applicable to be the performance verification of an open-type quaywall with sheet pile wall anchored by forward batter piles.

(2) Performance Verification of Sheet Piles and Other Types of Piles

① The performance verification of sheet pile walls may be performed by assuming the connection points between batter piles and sheet piles as fulcrums in accordance with the performance verification in **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

② On the basis of the reaction force at the connection points between batter piles and sheet piles as the horizontal force acting on the superstructures of open-type wharves, the axial force in sheet piles and piles can be calculated in accordance with the performance verification of open-type wharves on coupled raking piles.

(3) Performance Verification of Open-type Wharves

① For the performance verification of open-type wharves, refer to **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-type Wharves on Coupled Raking Piles**.

② For the assumptions regarding the seabed, refer to **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**. The estimation of the behavior of piles, including the lateral resistance capacity of piles, may be performed using Chang's method.

③ Generally, the vertical loads distributed to respective pile heads can be calculated as the fulcrum reaction force under the assumption that the superstructures of open-type wharves are simple beams supported at the positions of pile heads. The axial force on raking piles and sheet piles may be calculated using the horizontal force on open-type wharves and the vertical loads distributed to pile heads according to **equation (3.4.52)** in **Part III, Chapter 2, 3.4.9 Bearing Capacity of Coupled Piles**. As a compression force acting on vertical, the vertical loads distributed to pile heads can be used.

④ Generally, the flexural moment at the connection points between batter piles and sheet piles may be calculated as the flexural moment acting on a rigid frame fixed at the virtual fixed points of the batter and sheet piles subjected to the earth pressure, residual water pressure, dynamic water pressure during the actions of earthquake ground motions, and other horizontal forces.

⑤ The performance verification of open-type wharves shall be performed with due consideration to the rotation of open-type wharf blocks as needed.

(4) The examinations of the embedded lengths of piles with respect to axial force and lateral resistance can be made in accordance with **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

2.6.5 Performance Verification of Structural Members

(1) The performance verification of structural members on sheet pile walls anchored by foreword batter piles can be made by referring to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls** and **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles**.

(2) The connecting points between sheet pile walls and batter piles shall have structures that are capable of fulfilling their full function to transmit loads.

(3) The superstructures of open-type wharves shall have structures that are capable of fully withstanding the flexural moment transmitted from the sheet pile walls.

(4) Considering that damage to the connecting points could lead to the collapse of the entire quaywalls, the connecting points between sheet pile walls and batter piles must have sufficient reinforcement. The flexural moment generated in the heads of the sheet piles is transmitted to the superstructures of open-type wharves. Therefore, the flexural moment needs to be taken into consideration in the performance verification of the superstructures of open-type wharves.

2.7 Double Sheet Pile Quaywalls

[Public Notice] (Performance Criteria for Double Sheet Pile Quaywalls)

Article 50

3 In addition to the provisions in the paragraph (1), the performance criteria for double sheet pile structures shall be as prescribed respectively in the subsequent items:

(1) The risk of occurrence of sliding of the structural body shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(2) The risk that the deformation of the top of the front or rear sheet piles may exceed the allowable limit of deformation shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(3) The risk of losing the stability due to the shear deformation of the structural body shall be equal to or less than the threshold level under the permanent situation in which the dominating action is earth pressure.

[Interpretation]

11. Mooring Facilities

(4) Performance Criteria of Sheet Pile Quaywall

③ **Double sheet pile quaywall** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance on Criteria and the interpretation related to Article 50, Paragraph 3 of the Public Notice)

(a) The performance criteria and commentaries for a double sheet pile quaywall shall conform to those for sheet pile quaywall and the following provisions:

(b) The required performance of a double sheet pile quaywall under the permanent action situation in which the dominant actions are self-weight and earth pressure and the variable action situation in which the dominant action is Level 1 earthquake ground motions shall focus on serviceability. **Attached Table 11-12** shows the performance verification items and standard indexes used for determining the limit values with respect to the actions.

Attached Table 11-12 Performance Verification Items and Standard Indexes Used for Determining the Limit Values under the Respective Design Situations of a Double Sheet Pile Quaywall

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index to determine limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non dominating action</th></tr>
<tr><td rowspan="3">26</td><td rowspan="3">1</td><td rowspan="3">2</td><td rowspan="2">50</td><td rowspan="2">3</td><td rowspan="2">1</td><td rowspan="3">Serviceability</td><td>Permanent</td><td>Earth pressure</td><td>Self-weight, water pressure, surcharge</td><td rowspan="2">Sliding of wall body</td><td rowspan="2">Action–resistance ratio with respect to the sliding of the wall body</td></tr>
<tr><td>Variable</td><td>L1 earthquake ground motion</td><td>Self-weight, earth pressure, water pressure, surcharge</td></tr>
<tr><td>50</td><td>1</td><td>4</td><td>Permanent</td><td>Self-weight</td><td>Water pressure, surcharge</td><td>Circular slip failure of ground</td><td>Action–resistance ratio with respect to circular slip failure</td></tr>
</table>

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non dominating action		
			50	3	2		Permanent	Earth pressure	Self-weight, water pressure, surcharge	Deformation of the crown of the front and back sheet piles	Residual deformation amount of the crown of the quaywall
							Variable	L1 earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge		
					3		Permanent	Earth pressure	Water pressure, surcharge	Shear deformation of wall body	Action–resistance ratio with respect to shear deformation of wall body

(c) In addition to the above information, the requirements and the commentaries in Paragraph 3 (Scouring and Wash Out), Article 22 and Article 28 (Performance Criteria of Armor Stones and Blocks) of the Public Notice shall be applied as needed.

2.7.1 General

(1) The following is applicable to the performance verification of mooring facilities that use double sheet pile structures.

(2) A double sheet pile quaywall is a mooring facility that has an earth-retaining wall structure constructed in a manner that drives two rows of sheet pile walls, connects the two walls through tie members or similar materials, and fills the space between the two walls with soil.

(3) **Fig. 2.7.1** shows an example of the cross section of a double sheet pile quaywall.

(4) **Fig. 2.7.2** shows an example of the sequence of the performance verification of a double sheet pile quaywall. There may be another case of the performance verification of the shear deformation of double sheet pile walls based on virtual cross sections that assume sheet pile wall structures have sheet pile anchorages with the embedded lengths of sheet piles and the distance between two sheet piles equivalent to those of double sheet pile walls. However, **Fig. 2.7.2** does not show the evaluation of the effects of liquefaction and settlement due to earthquake ground motions. Therefore, regarding liquefaction, the possibility of and countermeasures against liquefaction shall be appropriately deliberated with reference to **Part II, Chapter 7 Ground Liquefaction**. Here, the performance of double sheet pile walls under a variable action situation with respect to Level 1 earthquake ground motions can be verified by the seismic coefficient method based on the static equation of equilibrium. However, for high earthquake-resistance facilities, the deformation is preferably deliberated by nonlinear seismic response analysis or other methods by taking into consideration the dynamic interaction between the ground and structures. For double sheet pile quaywalls other than high earthquake-resistance facilities, the verification of the accidental situation with respect to Level 2 earthquake ground motions can be omitted.

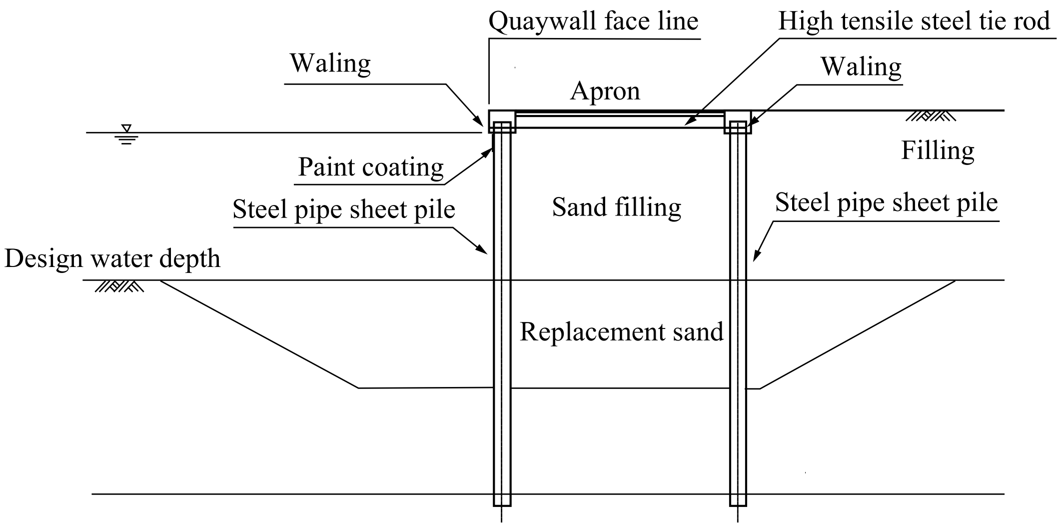
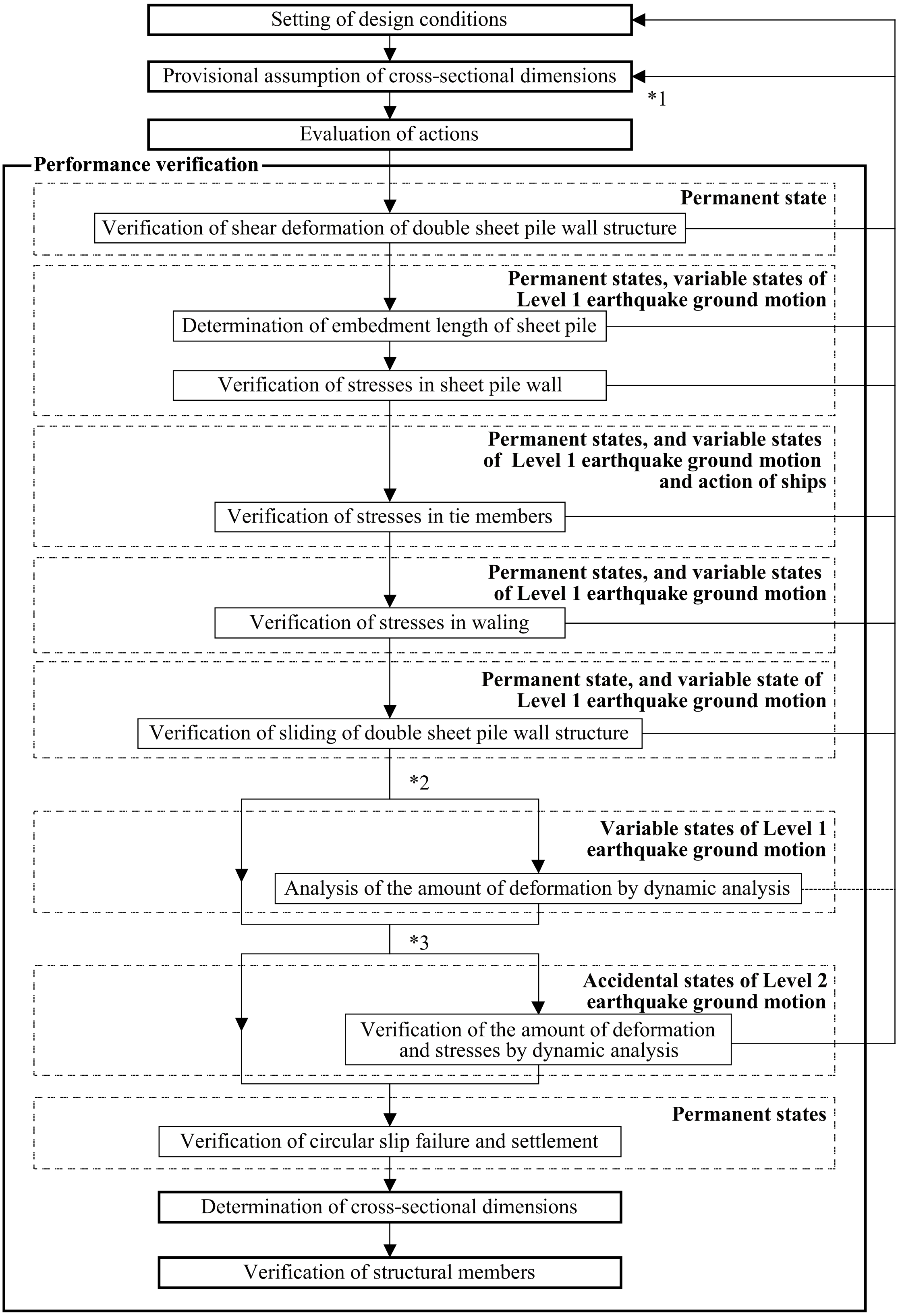


Fig. 2.7.1 Example of the Cross Section of a Double Sheet Pile Quaywall



*1: The evaluation of the effect of liquefaction is not shown because this must be separately considered.

*2: Analysis of the amount of deformation due to Level 1 earthquake ground motion may be carried out by dynamic analysis when necessary.
For high earthquake-resistance facilities, analysis of the amount of deformation by dynamic analysis is desirable.

*3: For high earthquake-resistance facilities, verification is carried out for Level 2 earthquake ground motion.

Fig. 2.7.2 Example of the Sequence of the Performance Verification of Double Sheet Pile Quaywalls

(5) The performance verification of double sheet pile quaywalls has conventionally been performed in accordance with the performance verification methods for steel pile cellular-bulkhead quaywalls or sheet pile quaywalls with sheet pile anchorages. Therefore, the performance verification methods described in this section can be applied to double sheet pile quaywall under conditions that are similar to those frequently applied to existing quaywalls.

(6) The performance verification of deformation is important when applying double sheet pile structures to large-scale permanent structures. The methods for evaluating the deformation of double sheet pile structures include Sawaguchi's method[65] and Ohori's method,[66] which was established by modifying Sawaguchi's method so that the deformation of double sheet pile walls can be comprehensively evaluated. Considering that these methods are simplified ones, it is preferable that detailed methods, such as dynamic analyses, are used for the performance verification of the deformation during the actions of earthquake ground motions.

(7) Similar to the case with a cellular-bulkhead structure, a double sheet pile structure can ensure structural stability after the completion of filling work but has a risk of collapsing when hit by small waves during construction with no materials filled in between two sheet piles. Therefore, filling work is preferably implemented as soon as sheet piles are driven. To facilitate the early implementation of filling work, it is common practice to install supplemental sheet piles that function as bulkheads between the rows of sheet piles at the intervals depending on the wave height, the types of infill materials to be used, and the construction site conditions. It is also common practice to use tie members in combination with rigid beams to brace sheet piles during construction.

(8) In constructing a double sheet pile quaywall, the installation of a double sheet pile wall (two rows of sheet piles with filling sand placed in between) is generally implemented before backfilling work. Therefore, sheet piles with identical shapes and dimensions are normally used for both rows.

(9) When used for purposes other than mooring facilities such as enclosing bunds, breakwaters, or revetments, the performance of double sheet pile structures shall be verified by appropriate methods. For example, the performance verification of those used as temporary enclosure bunds or earth-retaining walls shall be performed for the embedded lengths followed by seepage control effects (seepage path lengths) and the heaving and piping prevention. Furthermore, the performance verification of double sheet pile walls can be performed with reference to **references 67)** and **68)**.

(10) Considering that the sheet piles of double sheet pile quaywall are subjected to severely corrosive environments, the sheet piles shall be designed with appropriate corrosion protection measures on the basis of **Part III, Chapter 2, 1.3.4 Examination of Performance Deterioration with Age**.

2.7.2 Actions

(1) For the actions on double sheet pile quaywalls, refer to **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections**.

(2) **Equation (2.7.1)** can be used for calculating the characteristic value of the seismic coefficient to be used in the performance verification of double sheet pile quaywalls under the variable action situation with respect to Level 1 earthquake ground motions.[58] In this equation, the subscript k indicates the characteristic value. For the filter and reduction coefficient required when reflecting the frequency characteristics into the calculation of the maximum acceleration α_c in the equation, refer to **Reference (Part III), Chapter 1, 1 Detail Items about Seismic Coefficient for Verification**.

$$k_{h_k} = 1.91\left(\frac{D_a}{D_r}\right)^{-0.69} \cdot \frac{\alpha_c}{g} + 0.03 \qquad \textbf{(2.7.1)}$$

where

k_h : seismic coefficient for verification;

D_a : allowable deformation amount (15 cm);

D_r : reference deformation amount (10 cm);

g : gravitational acceleration (980 cm/s^2).

2.7.3 Performance Verification

(1) The examination to determine the distance between two sheet pile walls to achieve the required strength against shear deformation can be made in accordance with **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections**.

(2) The calculation of the deformation moment can be made in accordance with **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections**.

(3) The calculation of the resistance moment can be made in accordance with **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections** provided that the moment due to the tension force between the joints on bulkhead sheet piles is not generally considered to contribute to the resistance moment.

(4) The embedded length of sheet piles is generally the following value, whichever is larger: the value calculated by the method for sheet piles with an anchorage (the examination of the embedded lengths of sheet piles in **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**) or the value satisfying the allowable horizontal displacement of the top of cellular bulkhead (the examination of the stability of a wall body as a whole and the examination of displacement of the top of cellular bulkhead in **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections**). However, for important facilities, the performance verification is preferably performed by accurate methods (such as model experiments or numerical analyses capable of reproducing mechanisms). When using numerical analyses, refer to **Reference (Part III), Chapter 1, 2 Basic Items for Earthquake Response Analyses**.

(5) The flexural stress in sheet piles of double sheet pile quaywalls is considered to be caused by the actions of waves on the series of sheet piles during construction without any support, the earth pressure after the completion of filling work, the earth pressure of the soil reclaimed behind double sheet pile walls, and seaward water pressure as a result of the lowering of water levels in front of double sheet pile walls. The cross sections of sheet piles shall be determined by using the most severe flexural stress after examining all the cases described above.

(6) For the calculation of the tension force in tie members, refer to the tension force in tie members in **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(7) For the performance verification of waling, refer to the verification of waling in **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**.

(8) A double sheet pile quaywall can be considered a kind of gravity-type quaywall. Therefore, it is necessary to verify the stability of quaywalls against the sliding and the circular slip failure including a wall body as is the case with a cellular-bulkhead type quaywall. The performance verification of double sheet pile quaywalls can be performed with reference to the performance verification described in **Part III, Chapter 5, 2.2 Gravity-type Quaywalls**. The examination of sliding failure shall generally be performed on both failure surfaces: one is a seafloor surface that is virtually set as the bottom face of a sheet pile wall, and the other is a surface that passes through the toe of the sheet pile wall. In the former case, the resistance of the sheet pile wall below the virtual seafloor surface should be ignored. In the examination of the overall slope stability, including the double sheet pile wall with the embedded length of the double sheet pile wall equal to or longer than the required embedded length calculated for a sheet pile quaywall with an anchorage, the portion of a sheet piles below the required embedded length shall not be considered to contribute to resisting circular slip failures that have failure surfaces passing below the required embedded length.

(9) For the performance verification of the slabs and upright sections of a superstructure, refer to the performance verification of the relieving platforms in **Part III, Chapter 5, 2.8 Quaywalls with Relieving Platforms**. Foundation piles are driven into the filling material to support the superstructure according to the circumstances. These piles should have sufficient safety against the horizontal and vertical force transmitted from the superstructure. Here, it is assumed that the vertical force transmitted from the superstructure is entirely borne by the piles, and the vertical bearing capacity of the piles is calculated by ignoring the skin friction between the piles and filling material. The horizontal force that acts on the superstructure is transmitted to a double sheet pile wall partly through the piles and partly through the sheet piles. Therefore, the appropriate shares of the horizontal force between the piles and sheet piles should be determined.

2.8 Quaywalls with Relieving Platforms

[Public Notice] (Performance Criteria of Quaywalls with Relieving Platforms)

Article 51

The performance criteria of quaywalls with relieving platforms shall be as prescribed respectively in the following items:

(1) Sheet piles shall have the embedment length as necessary for the structural stability and shall contain the degree of risk that the stresses in the sheet piles may exceed the yield stress at a level equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(2) The risk of occurrence of sliding or overturning of the wall body shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(3) The following criteria shall be satisfied under the permanent situation in which the dominating action is self-weight:

(a) The risk that the axial forces acting in the relieving platform piles may exceed the resistance force based on the failure of the ground shall be equal to or less than the threshold level.

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

(4) The following criteria shall be satisfied under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating actions are Level 1 earthquake ground motions, ship berthing, and traction by ships:

(a) The risk that the axial forces acting on the relieving platform piles may exceed the resistance force based on the failure of the ground shall be equal to or less than the threshold level.

(b) The risk that the stress acting on the relieving platform piles may exceed the yield stress shall be equal to or less than the threshold level.

(c) The risk of impairing the integrity of the members of the relieving platform shall be equal to or less than the threshold level.

(5) The risk of occurrence of a slip failure in the ground that passes below the bottom end of the sheet piling shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is self-weight.

[Interpretation]

11. Mooring Facilities

(5) Performance Criteria of Quaywalls with Relieving Platforms (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance and the Interpretation related to Article 51of the Public Notice,)

① Serviceability shall be the required performance for quaywalls with relieving platforms under the permanent situation in which the dominating action is self-weight or earth pressure and under the variable situation in which the dominating actions are Level 1 earthquake ground motions, ship berthing, and traction by ships. Performance verification items for those actions and standard indexes for setting limit values shall be in accordance with **Attached Tables 11-13** and **11-14**. In **Attached Table 11-13**, the wall body is equivalent to the wall body in the case of a gravity-type quaywall in the verification of the stability of the structure of quaywalls with relieving platforms.

Attached Table 11-13 Performance Verification Items and Standard Indexes for Setting Limit Values for Sheet Piles and Structural Stability of Quaywalls with Relieving Platforms under Different Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	51	–	1	Serviceability	Permanent	Earth pressure	Water pressure, surcharges	Necessary embedment length	Embedment length necessary for structural stability
										Yielding of sheet pile	Design yield stress of sheet pile
							Variable	Level 1earthquake ground motion	Earth pressure, water pressure, surcharges	Necessary embedment length	Embedment length necessary for structural stability
										Yielding of sheet pile	Design yield stress of sheet pile
					2		Permanent	Earth pressure	Self-weight, water pressure, surcharge	Sliding/overturning of wall body	Action-to-resistance ratio for sliding and overturning of wall body
							Variable	Level 1earthquake ground motion	Self-weight, earth pressure, water pressure, surcharge	Sliding/overturning of wall body	Action-to-resistance ratio for sliding and overturning of wall body
					5		Permanent	Self-weight	Water pressure, surcharge	Circular slip failure of ground	Action-to-resistance ratio for circular slip failure

Attached Table 11-14 Performance Verification Items and Standard Indexes for Setting Limit Values for Relieving Platforms and Relieving Platform Piles of Quaywalls with Relieving Platforms under Different Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	51	–	3 (a)	Serviceability	Permanent	Self-weight	Surcharging, water pressure	Axial forces on relieving platform piles	Action-to-resistance ratio for bearing capacity of anchorage work (pushing, pulling)
					3 (b)				Earth pressure, water pressure, surcharge	Sectional stress of cross-section of relieving platform	Bending compressive stress
					4 (a)		Permanent	Earth pressure	Self-weight, water pressure, surcharge	Axial forces on relieving platform piles	Action-to-resistance ratio for bearing capacity of piles (pushing, pulling)
							Variable	Level 1earthquake ground motion Traction of ships	Self-weight, earth pressure, water pressure, surcharge		

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index for setting limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="7"></td><td rowspan="7"></td><td rowspan="7"></td><td rowspan="7"></td><td rowspan="7"></td><td rowspan="3">4
(b)</td><td rowspan="7"></td><td>Permanent</td><td>Earth pressure</td><td>Water pressure, surcharge</td><td rowspan="3">Yielding of relieving platform piles</td><td rowspan="3">Design yield stress</td></tr>
<tr><td rowspan="2">Variable</td><td>Level 1earthquake ground motion</td><td rowspan="2">Self-weight, earth pressure, water pressure, surcharge</td></tr>
<tr><td>Traction of ships</td></tr>
<tr><td rowspan="4">4
(c)</td><td>Permanent</td><td>Earth pressure</td><td>Water pressure, surcharge</td><td>Sectional stress of cross-section of relieving platform</td><td>Bending compressive stress</td></tr>
<tr><td rowspan="3">Variable</td><td>Level 1earthquake ground motion</td><td rowspan="3">Self-weight, earth pressure, water pressure, surcharge</td><td rowspan="3">Failure of cross-section of relieving platform</td><td rowspan="3">Design cross-sectional resistance force</td></tr>
<tr><td>Ship berthing, traction of ships</td></tr>
<tr></tr>
</table>

② In addition to these provisions, provisions regarding the **Public Notice**, **Article 22**, **Paragraph 3** (Scouring and Sand Washing Out), and **Article 28** (Performance Criteria for Armor Stones and Blocks) and their interpretations shall apply, as needed.

2.8.1 General

(1) The provisions in this section may be applied to the performance verification of a quaywall with a relieving platform comprising a relieving platform, a sheet pile wall in front of the relieving platform, and relieving platform piles.

(2) A quaywall with a relieving platform normally comprise a relieving platform, a sheet pile wall in front of the relieving platform, and relieving platform piles. The relieving platform is in many cases constructed as an L-shaped structure of cast-in-place reinforced concrete and the upper part of the platform is usually buried under landfill material, but sometimes a box-shaped platform is used to reduce the weight of the platform and the earthquake forces that act on it (see **Figs. 2.8.1** and **2.8.2**.).

(3) The performance verification of a quaywall with a relieving platform can be conducted separately for the sheet piles, relieving platform, and relieving platform piles. For the sheet piles, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**. For the relieving platform and the relieving platform piles, refer to **Part III,** , **Chapter 2**, **3.4 Pile Foundations**. For sliding and overturning of a quaywall with a relieving platform as a whole, refer to **Part III, Chapter 5**, **2.2 Gravity-type Quaywalls**. For circular slip failure, refer to **Par III, Chapter 2**, **4.2.1 Stability Analysis by Circular Slip Failure Plane**. Analysis of circular slip failure is often omitted for ground of relatively good quality, such as sandy ground.

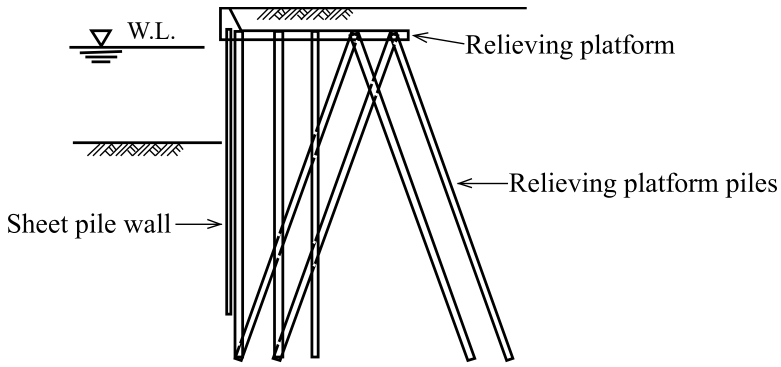


Fig. 2.8.1 Structure of Quaywall with Relieving Platform (L-Shaped Platform)

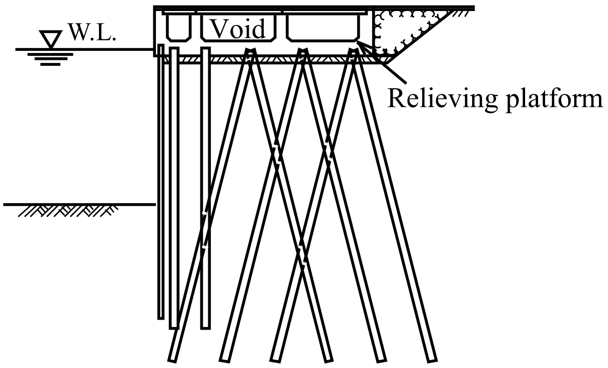


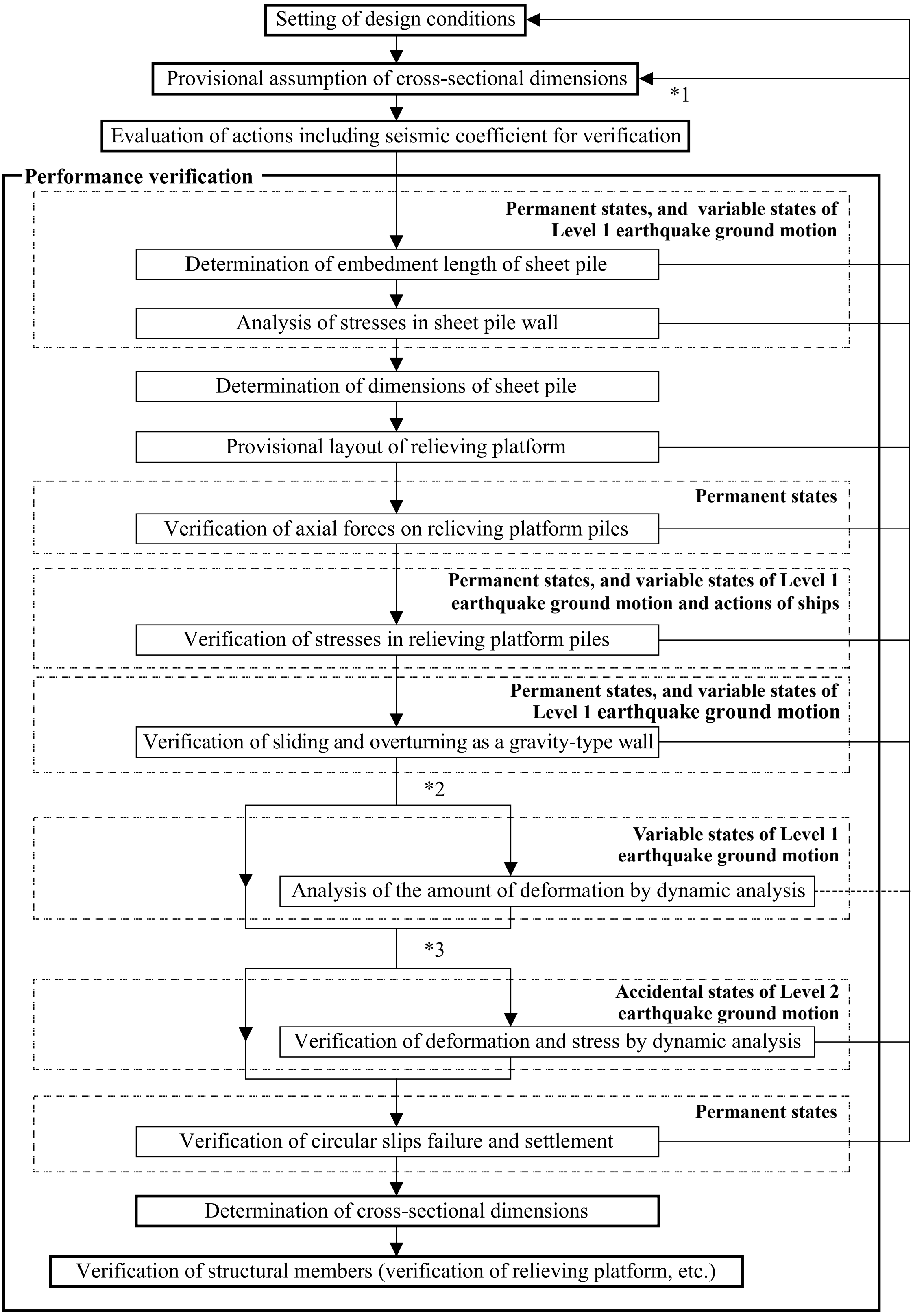
Fig. 2.8.2 Structure of Quaywall with Relieving Platform (Box-Shaped Platform)

(4) The approaches given in **Part III, Chapter 5**, **2.8.3 Actions** and **Part III, Chapter 5**, **2.8.4 Performance Verification** use simplified techniques and thus caution is advised when adopting these approaches. Since quaywalls with relieving platforms have complex structures, highly precise methods (model tests and numerical analysis techniques that can simulate mechanisms) should be used for verification.

(5) An example of the sequence of performance verification of quaywalls with relieving platforms is shown in **Fig. 2.8.3**. Note that the evaluation of the effects of liquefaction, settlement, and other phenomena caused by earthquake ground motions is not shown in **Fig. 2.8.3**. Therefore, for liquefaction, as an example, it is necessary to determine whether there is liquefaction and to consider measures against it appropriately by reference to **Part II, , Chapter 7 Ground Liquefaction**. It is possible to verify the variable situation associated with Level 1 earthquake ground motions using the seismic coefficient method. For high earthquake-resistance facilities, however, it is desirable to analyze the amount of deformation, for example, using the nonlinear seismic response analysis in consideration of dynamic interaction between the ground and structure. For quaywalls with relieving platforms that are not categorized as high earthquake-resistance facilities, verification of the accidental situation associated with Level 2 earthquake ground motions can be omitted.

(6) For a quaywall that has a retaining sheet pile wall on the back of the relieving platform, performance verification of the sheet pile wall and relieving platform can be conducted with reference to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls** and **Part III, Chapter 5**, **5.2 Open-Type Wharves on Vertical Piles**, respectively. In the performance verification of such a quaywall, the earth pressure on the back of the relieving platform and the reaction in the upper part of sheet piling shall be taken into consideration as forces acting on the relieving platform.

(7) Sheet piles of quaywalls with relieving platforms are placed in severely corrosive environments. Therefore, the sheet piles shall be designed with appropriate corrosion protection measures in accordance with **Part III, Chapter 2**, **1.3.4 Examination of Change in Performance Over Time**.



*1: The evaluation of the effect of liquefaction is not shown; therefore, it must be considered separately.

*2: Analysis of the amount of deformation due to Level 1 earthquake ground motion may be carried out by dynamic analysis when necessary.
For high earthquake-resistance facilities, analysis of the amount of deformation due to Level 1 earthquake ground motion should also be carried out by dynamic analysis.

*3: For high earthquake-resistance facilities, verification is conducted for Level 2 earthquake ground motion.

Fig. 2.8.3 Example of the Sequence of Performance Verification of a Quaywall with a Relieving Platform

2.8.2 Setting of Cross-sectional Dimensions

(1) Determining the height and width of a relieving platform

Appropriate installation height and shape of a relieving platform shall be determined by considering conditions of actions, economic efficiency and constructability, and paying particular attention to the following points:

① The earth pressure acting on a sheet pile wall can be reduced by increasing the height of the relieving platform and placing its bottom at a lower level, which makes it possible to design the sheet pile wall with a smaller sectional area and shorter embedment length. At the same time, however, this makes it necessary to increase the quantity and length of reliving platform piles to support the increased weight of the relieving platform and resist a greater force of Level 1 earthquake ground motion acting on it.

② There is a possibility that the ground sinks at the bottom of the relieving platform and causes a gap below it. It is desirable to placing the bottom of the relieving platform at the level equal to or lower than the residual water level because piles may corrode.

③ To reduce the earth pressure acting on the sheet pile wall, it is common to determine the width of the relieving platform so that it intersects the sheet pile active failure plane extending from the seabed. When determining the width of the relieving platform, it is necessary to ensure that the required number of relieving platform piles can be arranged appropriately.

2.8.3 Actions

(1) The earth pressure and residual water pressure acting on the sheet pile wall vary according to the structural characteristics. Therefore, they shall be calculated appropriately in consideration of the height and width of the relieving platform as well as support conditions.

(2) When the relieving platform intersects the sheet pile active failure plane extending from the seabed, the active earth pressure acting on the sheet pile wall can be calculated assuming that the bottom of the relieving platform is the virtual ground plane and no surcharge is on it as shown in Fig. 2.8.4.

(3) Normally, the residual water pressure acting on the sheet pile wall may be considered as the residual water pressure acting on the bottom of the relieving platform and below on the assumption that the water pressure distribution is the same as that of a quaywall with no relieving platform (see **Fig. 2.8.4**.).

(4) As for passive earth pressure in front of the embedded section of a sheet pile wall, refer to **Part III, Chapter 5, 2.3 Sheet Pile quaywalls**.

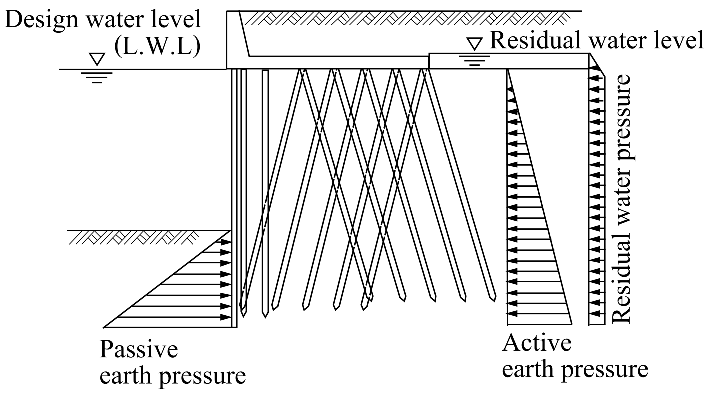


Fig. 2.8.4 Earth Pressure and Residual Water Pressure Acting on a Sheet Pile Wall

(5) The characteristic value of seismic coefficient for verification used in the performance verification of quaywalls with relieving platforms for the variable situations associated with Level 1 earthquake ground motion shall be appropriately calculated taking the structural characteristics into consideration. For convenience, the characteristic value of seismic coefficient for verification of quaywalls with relieving platforms may be calculated in accordance with **Part III, Chapter 5**, **2.2 Gravity-type Quaywalls**.

(6) For calculations of the earth pressure and residual water pressure acting on a sheet pile wall, refer to **Part II, Chapter 4**, **2 Earth Pressure,** and **Part II, Chapter 4**, **3.1 Residual Water Pressure**. In this case, the angle of wall friction of the sheet pile wall may be taken to be 15° for active earth pressure, and −15° for passive earth pressure. For the residual water level, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**.

(7) It is desirable that the width of the relieving platform be extended to the range where it intersects with the active failure plane extending from the seabed. However, if the use of a narrow relieving platform is unavoidable, the following method can be used as the method of calculating the active earth pressure acting on the sheet pile wall.

As shown in **Fig. 2.8.5**, the earth pressure acting on the sheet pile wall is calculated as the earth pressure acting in the case that there is no relieving platform below the intersection point of the active failure plane extending from the rear end of the relieving platform and the sheet pile, and as the earth pressure acting in (2) above, above the point of intersection of the natural failure plane during Level 1 earthquake ground motion extending from the rear end of the relieving platform and the sheet pile. Between these two, it may be assumed that the earth pressure varies linearly.

The angle (α) formed between the natural failure plane and the horizontal during an earthquake can generally be obtained from **equation (2.8.1)**.

$$\alpha = \phi - \tan^{-1} k_h' \qquad \textbf{(2.8.1)}$$

where

ϕ : angle of shearing resistance of the soil (°)

k_h' : apparent seismic coefficient.

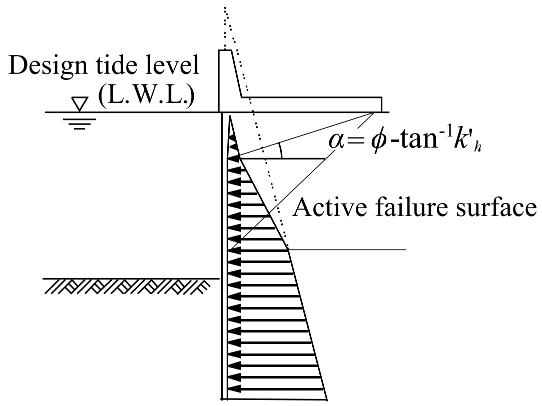


Fig.2.8.5 Earth Pressure Acting on Sheet Pile with Narrow Relieving Platform

(8) Relieving platform piles driven behind sheet piles bear a part of the earth pressure acting on the sheet piles and thereby have the effect of reducing the earth pressure acting on the sheet piles. Since there are many uncertainties in this effect, it is common that this effect is not taken into consideration for performance verification. There are some proposed methods, including a method of determining the distribution of the earth pressure based on the ratio of the flexural rigidity *EI* of the sheet piles to that of the relieving platform piles[69] and method of calculating the earth pressure acting on the sheet pile based on the ratio of the pile diameter to the center interval of piles[70].

(9) The horizontal force transmitted from the sheet piles and acting on the relieving platform may be calculated with the same method as that for the reaction at tie member installation point obtained in accordance with **Part III, Chapter 5**, **2.3.7 Performance Verification of Stability of Sheet Pile Walls as a Whole** by regarding the bottom of the relieving platform as a tie member installation point.

(10) The tractive force by ship and fender reaction force also act on the relieving platform. These external forces should be considered as necessary.

(11) The external forces transmitted from the sheet piles to the relieving platform include the horizontal force and bending moment. However, the transmission of the bending moment may be ignored for the sake of safety because the fixing of the sheet piles to the relieving platform may not be rigid enough.

(12) The earth pressure and residual water pressure acting on the back of the relieving platform can be calculated in accordance with **Part II, Chapter 4, 2 Earth Pressure** and **Part II, Chapter 4, 3.1 Residual Water Pressure**. In the calculation of earth pressure, surcharge should be taken into consideration. In the part below the bottom of relieving platform, the difference between the passive earth pressure acting on the rear and that acting on the front acts as the active earth pressure down to the depth where the two pressures are balanced. This should be added as shown in **Fig. 2.8.6**. The angle of wall friction can be taken to be 15° for active earth pressure and −15° for passive earth pressure.

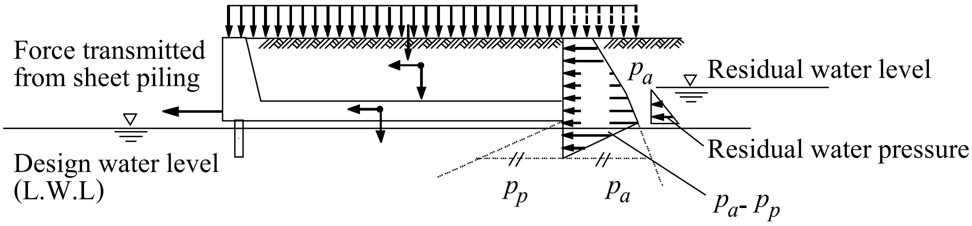


Fig. 2.8.6 External Forces to be considered for Performance Verification of a Relieving Platform

(13) For the self-weight of the relieving platform and the weight of soil and surcharge on the relieving platform, refer to **Part II, Chapter 10**, **2 Self-Weight** and **Part II, Chapter 10**, **3 Surcharge**.

(14) For the dynamic water pressure during action of Level 1 earthquake ground motion, refer to **Part II, Chapter 4**, **3.2 Dynamic Water Pressure**.

(15) When the relieving platform is an L-shaped structure, the earth pressure and residual water pressure acting on the upright section may be calculated in accordance with **Part II, Chapter 4, 2 Earth Pressure** and **Part II, Chapter 4, 3.1 Residual Water Pressure**. The angle of wall friction can be taken to be 15°.

2.8.4 Performance Verification

(1) Performance verification of sheet pile walls

① The embedment length of the sheet pile wall can be examined by assuming that the joint between a sheet pile and the relieving platform is a hinge support, replacing the bottom of the relieving platform with a tie member installation point and applying **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls.**

② Verification of stresses in the sheet pile wall may be conducted in accordance with **Part III, Chapter 5, 2.3 Sheet Piled Quaywalls**, replacing the bottom of the relieving platform with the tie member installation point.

③ In addition to the bending moment due to earth pressure, the bending moment and vertical force transmitted from the relieving platform act on the sheet piles of a sheet pile wall. Normally, the bending moment transmitted from the relieving platform is not taken into consideration because it usually acts in a direction opposite to that of the maximum bending moment that acts on the sheet piles and thus reduces the maximum bending moment. Furthermore, the vertical force transmitted from the relieving platform to the sheet pile wall is normally not taken into consideration when the front row of relieving platform piles is driven in as close to the sheet piles as possible and this significantly reduces the vertical force acting on the sheet piles.

(2) Performance verification of the relieving platform

① A relieving platform should be verified for performance as continuous beams that run in the direction of the quaywall alignment and in the direction perpendicular to the alignment and are supported at heads of relieving platform piles (see **Fig. 2.8.7**). In this case, loads cannot be distributed in the two directions. When the relieving platform is an L-shaped structure, the upright section should be verified for performance as a cantilever beam supported at the bottom slab.

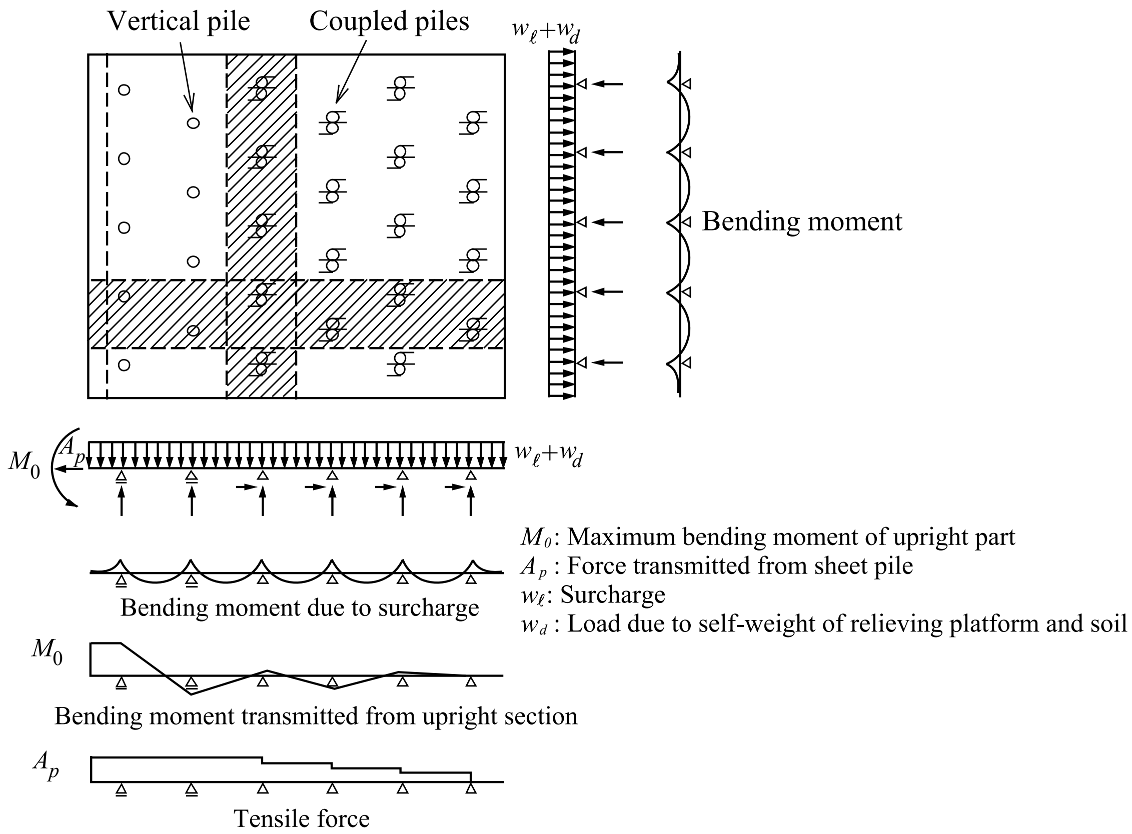


Fig.2.8.7 Continuous Beams Assumed in Performance Verification of a Relieving Platform

② When the relieving platform is an L-shaped structure, the continuous beams that run in the direction of quaywall alignment and in the direction perpendicular to the alignment are subjected to not only the bending moment due to the vertical action alone but also the bending moment transmitted from the upright section of the relieving platform. Therefore, the bending moment at the bottom slab of the relieving platform shall be calculated in consideration of the sum of these bending moments. A convenient way to calculate the bending moment transmitted from the upright section of the relieving platform is to transmit the maximum bending moment at the upright section using the moment distribution method.

③ A great horizontal force from the sheet pile wall acts on the relieving platform but this force can be directly transmitted to relieving platform piles by connecting the sheet piles and the relieving platform piles using tie members. Such a structure may be advantageous in the performance verification of the relieving platform. For the performance verification of tie members used for this purpose, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**. For the calculation of the horizontal force at heads of piles, refer to **(3) Performance Verification of Relieving Platform Piles** below.

④ Normally, relieving platform piles are arranged at short intervals. In principle, reinforcing bars for members of a relieving platform should be arranged at regular intervals without distributing the bending moment to column strips and middle strips.

(3) Performance verification of relieving platform piles

① Performance of relieving platform piles can be verified in accordance with **Part III, Chapter 2, 3.4 Pile Foundations.**

② In principle, relieving platform piles should consist of a combination of coupled piles and vertical piles. The horizontal external force may be borne by the coupled piles alone, and the vertical external force may be borne by the vertical piles alone. It may be assumed that each of the coupled piles bears the horizontal force equally.

③ In the performance verification of relieving platform piles, assessment should be made for the most dangerous state of each pile by varying the surcharge, direction of seismic forces, and tide level within the design condition ranges.

④ In calculating the axial resistance of each of the relieving platform piles, it is desirable to assume that in the ground above the sheet pile active failure plane extending from the seabed, the skin friction does not contribute as the resistance force of the relieving platform piles.

⑤ If it is unavoidable that the relieving platform piles are all composed of vertical piles and the horizontal force is borne by the vertical piles, it is normally assumed in calculating the resistance force normal to their axes that there is no soil above the sheet pile active failure plane extending from the seabed.

⑥ It is desirable to arrange relieving platform piles in such a way as to minimize the vertical force that comes from the relieving platform and acts on the sheet pile wall.

⑦ It is desirable to determine the length of each of relieving platform piles so that they reach almost the same depth.

(4) Analysis of the Stability as Gravity-type Walls

① The examination of the stability of a quaywall with a relieving platform as a whole can be made by assuming that the quaywall with a relieving platform is a kind of gravity-type wall.

② For analyzing the stability of the assumed gravity-type wall, refer to **Part III, Chapter 5, 2.2 Gravity-type Quaywalls**. In this case, the passive earth pressure to the front of the sheet pile can be taken into consideration.

③ A quaywall with a relieving platform may be considered as a gravity-type wall of which the rectangular cross-section is defined by a vertical plane containing the rear face of the relieving platform and a horizontal plane containing the bottom ends of the front side batter piles of the coupled piles, as shown in **Fig. 2.8.8**.

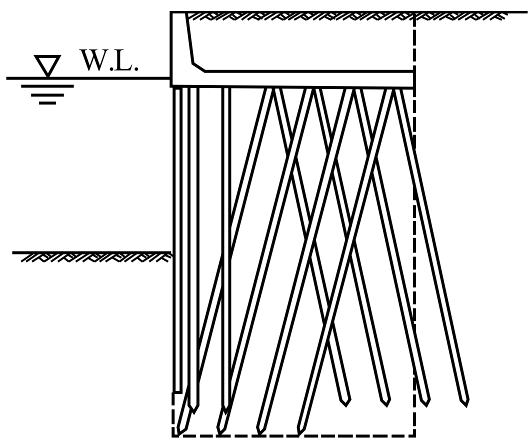


Fig.2.8.8 Virtual Wall as Gravity-type Wall

④ In principle, the frictional resistance acting on the bottom of the gravity-type wall can be assumed to be the product of the total vertical force acting on the wall and tan ϕ when the ground at the wall bottom is made of sandy soil or the product of the cohesion of clayey soil and the area of the wall bottom when the ground at the wall bottom is made of clayey soil. The total vertical force acting on the wall is a weight of the wall not including surcharges and calculated by subtracting buoyancy, and ϕ is the angle of shear resistance of sandy soil.

⑤ In principle, the angle of wall friction to be used in the calculation of earth pressure may be taken to be 15° for active earth pressure, and −15° for passive earth pressure. For clayey soil at the seabed or below, the apparent seismic coefficient to be used in the calculation of earth pressure during earthquake may be calculated in accordance with **Part II, Chapter 4**, **2 Earth Pressure**.

(5) Verification of circular slip failure

① For analysis of circular slip failure, refer to **Part III, Chapter 2, 4.2.1 Stability Analysis by Circular Slip Failure Plane**. In this case, analysis is conducted for circular slip failure passing under the bottom end of the sheet piling. Also, for setting the water level, refer to **Part II, Chapter 2, 3 Tide Levels**.

② When a quaywall with a relieving platform is considered unstable against circular slip failure, it is necessary to take appropriate measures, such as soil improvement and adoption of other type of structure. It is undesirable to increase the embedment length of sheet piles for the purpose of preventing circular slip failure.

2.8.5 Performance Verification of Structural Members

(1) It is necessary to connect a sheet pile wall and relieving platform piles to a relieving platform in such a way as to assure the required safety against stresses that occur at the connections.

(2) In principle, the top of a sheet pile shall be connected to a relieving platform either by embedding the top of the sheet pile into the relieving platform to a sufficient depth and welding reinforcing bars to the sheet pile or by attaching tie rods to convey the horizontal force from the sheet pile to relieving platform piles (see **Fig. 2.8.9**). When attaching tie members, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**.

(3) In principle, relieving platform piles shall be embedded into the bottom slab of the relieving platform to a sufficient depth so that the pile head reaction can be conveyed to the relieving platform. It is advisable to secure the heads of coupled piles by bolting or other means so that the piles can work as a unit (see **Fig. 2.8.9**).

(4) For details of the performance verification of structural members of quaywalls with relieving platforms, refer to those of open-type wharves on vertical piles described in **Part III, Chapter 5**, **5.2.6 Performance Verification of Structural Members**. In particular, for piles subjected to pulling force, it is necessary to thoroughly examine how to fix them to a relieving platform.

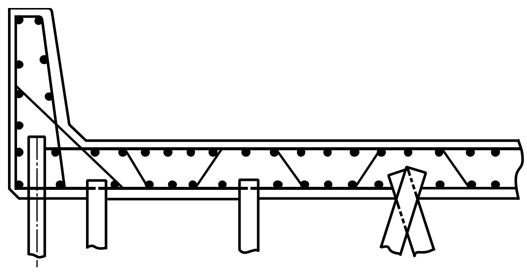


Fig. 2.8.9 Connections of Sheet Pile and Piles to a Relieving Platform

2.9 Embedded-Type Cellular-Bulkhead Quaywalls

[Public Notice] (Performance Criteria of Cellular-Bulkhead Quaywalls)

Article 52

1 The performance criteria of cellular-bulkhead quaywalls shall be as prescribed respectively in the following items:

(1) The following criteria shall be satisfied under the permanent situation, in which the dominating action is earth pressure:

(a) The risk of losing stability due to the shear deformation of the wall body shall be equal to or less than the threshold level.

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

(2) The following criteria shall be satisfied under the permanent situation, in which the dominating action is earth pressure, and under the variable situation, in which the dominating action is Level 1 earthquake ground motion.

(a) The risk of occurrence of sliding of the wall body and failure due to the insufficient bearing capacity of the foundation ground shall be equal to or less than the threshold level.

(b) The risk that the amount of deformation of the top of the cells may exceed the allowable limit of deformation shall be equal to or less than the threshold level.

(3) The risk of occurrence of slip failure in the ground shall be equal to or less than the threshold level under the permanent situation, in which the dominating action is self-weight.

(4) The following criteria shall be satisfied by the superstructure of cellular-bulkhead quaywalls under the permanent situation in which the dominating action is earth pressure, and under the variable situation in which the dominating actions are Level 1 earthquake ground motions, ship berthing, and traction by ships.

(a) The risk that the axial force acting in a pile may exceed the resistance force on the basis of the failure of the ground shall be equal to or less than the threshold level.

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

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

2 In addition to the provisions of the preceding paragraph, for the performance criteria of placement-type cellular-bulkhead quaywalls, the risk of occurrence of overturning under the variable situation, in which the dominating action is Level 1 earthquake ground motions, shall be equal to or less than the threshold level.

[Interpretation]

11. Mooring Facilities

(6) Performance Criteria of Cellular-Bulkhead Quaywalls

① **Embedded-type cellular-bulkhead quaywalls** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 52, Paragraph 1 of the Public Notice

(a) Serviceability shall be the required performance for embedded-type cellular-bulkhead quaywalls under the permanent state in which the dominating action is self-weight and earth pressure and under the variable state in which the dominating action is Level 1 earthquake ground motions, ship berthing or traction by ships. The performance verification items for those actions and standard indexes for setting the limit values shall be in accordance with **Attached Tables 11-15** and **11-16**.

Attached Table 11-15 Performance Verification Items and Standard Indexes for Setting Limit Values for the Structural Stability of Cells and the Integrity of Members of Embedded-type Cellular-bulkhead Quaywalls under Different Design Situations

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirements</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Index for setting limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="10">26</td><td rowspan="10">1</td><td rowspan="10">2</td><td rowspan="10">52</td><td rowspan="10">1</td><td>1 (a)</td><td rowspan="10">Serviceability</td><td rowspan="4">Permanent</td><td rowspan="4">Earth pressure</td><td rowspan="4">Water pressure, surcharges</td><td>Shear deformation of wall</td><td>Action-to-resistance ratio for the shear deformation of the wall body</td></tr>
<tr><td rowspan="3">1 (b)</td><td>Yielding of cell</td><td>Design yield stress</td></tr>
<tr><td>Yielding arc</td><td>Design yield stress</td></tr>
<tr><td>Joint yielding</td><td>Design yield stress</td></tr>
<tr><td rowspan="2">2 (a)</td><td>Permanent</td><td>Earth pressure</td><td>Self-weight, water pressure, surcharges</td><td rowspan="2">Wall sliding, bearing capacity of foundation ground</td><td>Action-to-resistance ratio for the sliding of the wall body and the bearing capacity of the foundation ground</td></tr>
<tr><td>Variable</td><td>Level 1 earthquake ground motion</td><td>Self-weight, earth pressure, water pressure, surcharges</td><td>Action-to-resistance ratio for the sliding of the wall body and the bearing capacity of the foundation ground</td></tr>
<tr><td rowspan="2">2 (b)</td><td>Permanent</td><td>Earth pressure</td><td>Water pressure, surcharges</td><td rowspan="2">Deformation of cell top</td><td rowspan="2">Residual deformation amount at the top of the quaywall</td></tr>
<tr><td>Variable</td><td>Level 1 earthquake ground motion</td><td>Self-weight, earth pressure, water pressure, surcharges</td></tr>
<tr><td>3</td><td>Permanent</td><td>Self-weight</td><td>Water pressure, surcharges</td><td>Circular slip failure of ground</td><td>Action-to-resistance ratio for circular slip failure</td></tr>
</table>

Attached Table 11-16 Performance Verification Items and Standard Indexes for Setting Limit Values for Superstructure of Embedded-type Cellular-bulkhead Quaywalls under Different Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	52	1	4 (a)	Serviceability	Permanent	Earth pressure	Self-weight, water pressure, surcharges	Axial forces acting on superstructure piles*1)	Action-to-resistance ratio for the supporting capacity of the piles (pushing and pulling)
							Variable	Level 1 earthquake ground motion	Self-weight, earth pressure, water pressure. surcharges		
								Traction of ships			
					4 (b)		Permanent	Earth pressure	Water pressure. surcharges	Yielding of superstructure piles*1)	Design yield stress
							Variable	Level 1 earthquake ground motion	Self-weight, earth pressure, water pressure. surcharges		
								Traction of ships			
					4 (c)		Permanent	Earth pressure	Water pressure. surcharges	Sectional stress of the superstructure cross section	Bending compressive stress
							Variable	Level 1 earthquake ground motion	Self-weight, earth pressure, water pressure. surcharges	Cross-sectional failure of superstructure	Design cross-sectional resistance
								Berthing and traction of ships			

*1) Only for structures having superstructure supporting piles

(b) In addition to these provisions, the provisions regarding Article 22, Paragraph 3 (Scouring and Sand Washing Out) and Article 28 (Performance Criteria for Armor Stones and Blocks) of the Public Notice and their interpretations shall apply as needed.

2.9.1 General

(1) This section is applicable to the performance verification of quaywalls using a steel-sheet-pile cellular-bulkhead structure (hereinafter steel-sheet-pile cellular-bulkhead quaywalls) and quaywalls having a steel plate cellular-bulkhead structure with embedded sections (hereinafter embedded-type steel plate cellular-bulkhead quaywalls).

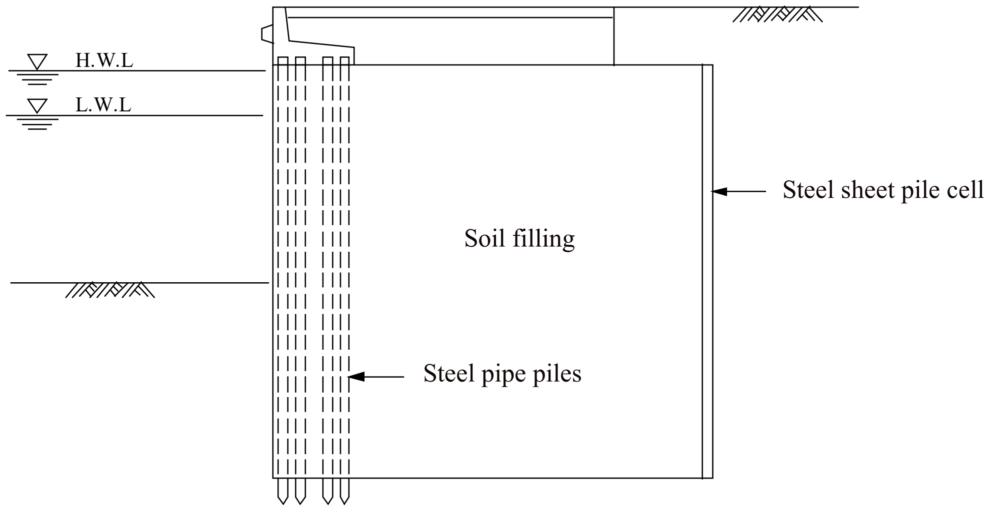
(2) The performance verification method described in this section is mainly based on the results of cellular-bulkhead model tests [71), 72), 73), 74)] conducted on a sandy soil ground with an embedded length ratio of 0 to 1.5 and a ratio of equivalent wall width to wall height of 1 to 2.5. For cases wherein the embedded length ratio is small (i.e., less than 1/8) and the equivalent wall width is small relative to the wall height or cases wherein the quaywall should be constructed on a clay soil ground or ground improved by the sand compaction piles, further examinations (e.g., a numerical analysis that takes into consideration the nonlinear characteristics of the ground) should be preferably

made, in addition to the examination using the performance verification method described in this section, because these cases involve factors that cannot be fully clarified with the method described here.

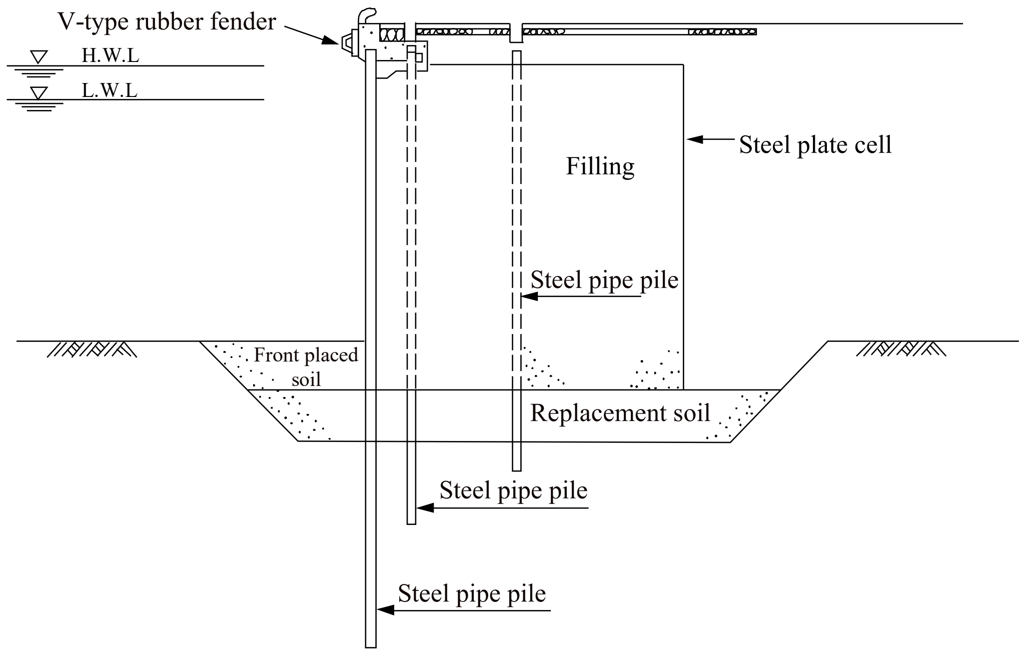
(3) **Fig. 2.9.1** shows examples of the cross section of a steel-sheet-pile cellular-bulkhead quaywall and an embedded-type steel-plate cellular-bulkhead quaywall.

(4) The approaches in **Part III, Chapter 5, 2.9.2 Action** and **Part III, Chapter 5**, **2.9.4 Performance Verification** may be used for simple verification, but it is necessary to be careful when adopting these approaches. Highly accurate methods (model tests and numerical analysis techniques that can simulate mechanisms) should be used for verification.

(5) **Fig. 2.9.2** shows an example of the sequence of performance verification of an embedded-type cellular-bulkhead quaywall. Note that the evaluation of the effects of liquefaction, settlement, and other phenomena caused by earthquake ground motions is not shown in **Fig. 2.9.2**. Therefore, for liquefaction, it is necessary to determine whether there is liquefaction and to consider measures against it appropriately by referring to **Part II, Chapter 7 Ground Liquefaction**. It is possible to verify the variable situation associated with Level 1 earthquake ground motions by using the seismic coefficient method. For high-earthquake-resistance facilities, however, it is desirable to analyze the amount of deformation (e.g., by using the nonlinear seismic response analysis in consideration of the dynamic interaction between the ground and a structure). For embedded-type cellular-bulkhead quaywalls that are not categorized as high-earthquake-resistance facilities, the verification of the accidental situation associated with Level 2 earthquake ground motions can be omitted.

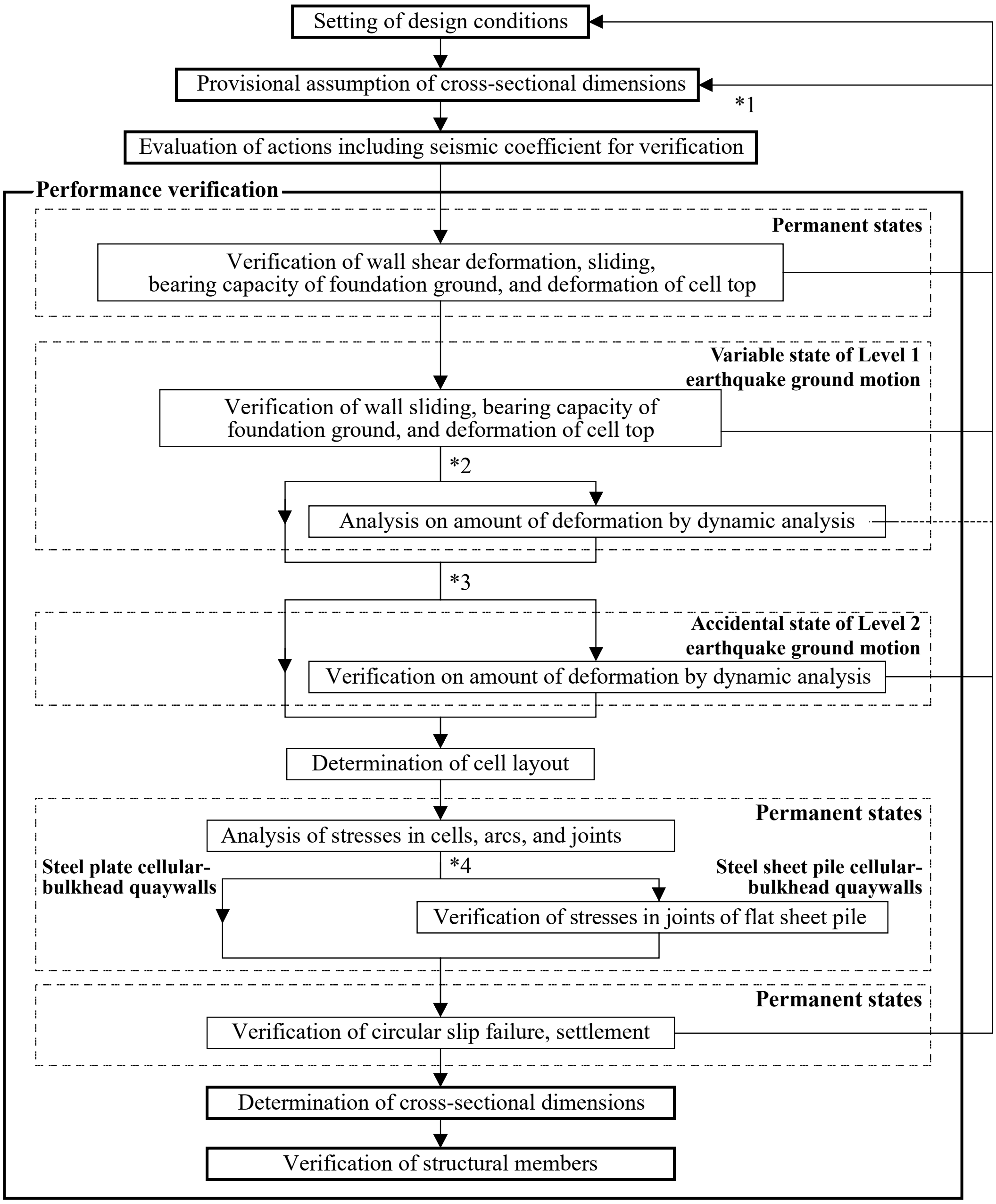


(a) Embedded-type steel sheet pile cellular-bulkhead quaywall



(b) Embedded-type steel plate cellular-bulkhead quaywall

Fig. 2.9.1 Examples of the Cross-section of Embedded-type Cellular-bulkhead Quaywalls



*1: The evaluation of the effect of liquefaction is not shown; therefore, this must be separately considered.
*2: The analysis of the amount of deformation due to Level 1 earthquake ground motion may be performed by dynamic analysis when necessary.
For high-earthquake-resistance facilities, the analysis of the amount of deformation due to Level 1 earthquake ground motion should also be performed by dynamic analysis.
*3: For high-earthquake-resistance facilities, verification is performed for Level 2 earthquake ground motion.
*4: For steel-sheet-pile cellular-bulkhead quaywalls, verification is performed for the joints of a flat sheet pile.

Fig. 2.9.2 Example of the Sequence of Performance Verification of Embedded-type Cellular-bulkhead Quaywalls

(6) It is recommended that the filling material in cells is a sufficient density sand or gravel of good quality. It is not desirable to use clayey soil as filling material. When clayey soil remained in the cells, it is necessary to make a separate examination because the deformation of the cells may become significantly large.

(7) When a foundation for a crane, shed, or warehouse is to be built within a cell, it is desirable to use foundation piles to transmit the load to the bearing stratum.

(8) When constructing steel-sheet-pile cells, the cells should be filled as soon as possible after driving the sheet piles to reduce the time during which the cells are unstable without any filling.

2.9.2 Actions

(1) To calculate the action to be considered in the performance verification of embedded-type cellular-bulkhead quaywalls, refer to **Part II, Chapter 6**, **2 Seismic Action**, **Part II, Chapter 4**, **2 Earth Pressure**, **Part II, Chapter 4**, **3 Water Pressure**, and **Part II, Chapter 10 Self-Weight and Surcharges**.

(2) During the examination of shear deformation of the cell wall body, the rear of the wall may be subjected to active earth pressure (see **Fig. 2.9.3**). The results of the model tests show that the embedded section of the cell is subjected to the action corresponding to the earth pressure at rest because the deformation of the embedded section of the cell is small. According to the results of the shaking table tests, the earth pressure acting on this part works as a resisting force against overturning of the wall. In examining the stability of the entire system, it can be generally considered that the active earth pressure acts on the rear of the wall above the seabed and that the seabed earth pressure acts on the rear of the wall at the seabed or below regardless of depth (see **Fig. 2.9.4**). The characteristic value of the earth pressure at the seabed can normally be calculated using **equation (2.9.1)**.

$$p_{ac} = 0.5\left(\sum wh + q\right), \qquad \textbf{(2.9.1)}$$

where

p_{ac} : earth pressure acting on the rear of wall below the seabed (kN/m^2);

w : unit weight of each layer of backfilling (kN/m^3);

h : thickness of each layer of backfilling (m);

q : surcharge (kN/m^2).

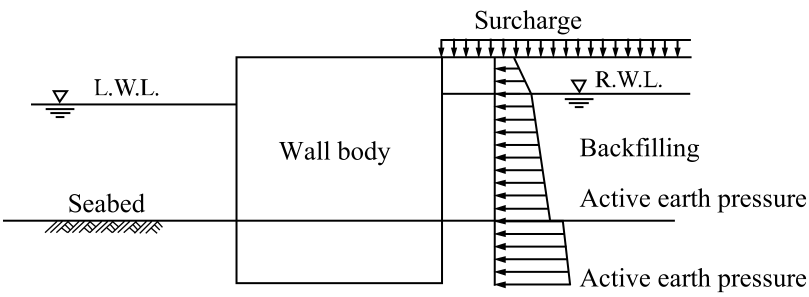


Fig. 2.9.3 Earth Pressure Acting on the Rear of Wall Body for Examination of Shear Deformation

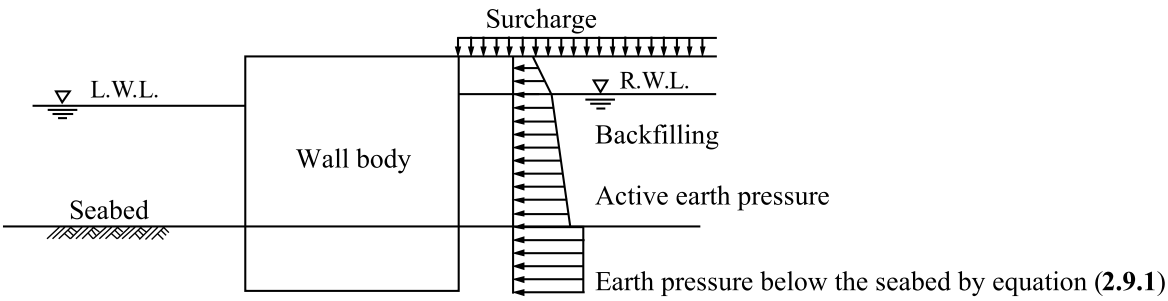


Fig. 2.9.4 Earth Pressure Acting on the Rear of Wall Body for Examination of the Stability as Gravity-type Wall

(3) In principle, the residual water level of the backfilling can be taken at the elevation with the height equivalent to two-thirds of the tidal range above the monthly mean lowest water level (L.W.L.). However, when using a backfilling with low permeability, the residual water level may become higher than this level. Therefore, it is desirable to determine the residual water level on the basis of the results of the investigations of existing structures and similar structures. The residual water level in the filling material in the cells may be set to the same level as that of the backfilling for the wall body. For steel-plate cells, joints are provided only at the connections between the cells and arcs. Therefore, the water blocking capability of steel-plate cells can be considered equivalent to or higher

than that of steel-sheet-pile cells, although there is no data on the actual measurements of the water blocking capability. It is necessary to note that the water level may significantly differ from that at the front of the cells when the ground water level at the rear of a quaywall rises due to rain or for any other reason.

(4) Seismic coefficient for verification used in the performance verification of embedded-type cellular-bulkhead quaywalls

① The characteristic value of the seismic coefficient for the performance verification of embedded-type cellular-bulkhead quaywalls under variable situations associated with Level 1 earthquake ground motion, as well as the allowable value of the amount of deformation set corresponding to the seismic coefficient for verification, shall be appropriately calculated by taking the structural characteristics into consideration.

② The characteristic value of the seismic coefficient for the verification of embedded-type cellular-bulkhead quaywalls can be calculated by using **equation (2.9.2)**. The basic approach described in **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls** shall apply to the calculation. For the filter used for considering frequency characteristics, the reduction coefficient, the equation for calculating the seismic coefficient for verification, and the allowable value of the amount of deformation, refer to **References (Part III)**, **Chapter 1, 1 Details about Seismic Coefficient for Verification**[75].

$$k_h = 1.62\left(\frac{D_a}{D_r}\right)^{-0.58} \cdot \frac{\alpha_c}{g} + 0.04, \tag{2.9.2}$$

where

k_h : seismic coefficient for verification;

D_a : allowable deformation (10 cm);

D_r : referenfedeformation (10 cm);

g : acceleration of gravity (980 cm/s^2).

(5) For the part above the seabed, the seismic coefficient to be used in the calculation of the seismic inertia force that acts on the filling material shall be the seismic coefficient for verification. For the seabed and area below the seabed, this value is reduced linearly in such a way that it becomes 0 at 10 m below the seabed. In principle, the seismic inertia force is not considered for parts deeper than that level (see **Fig. 2.9.5**).

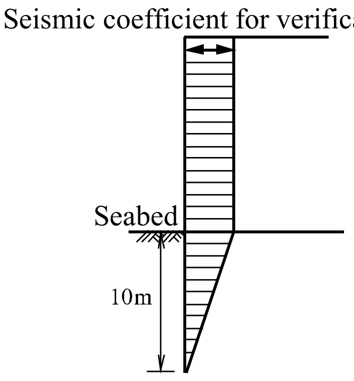


Fig. 2.9.5 Inertia Force Acting on Filling

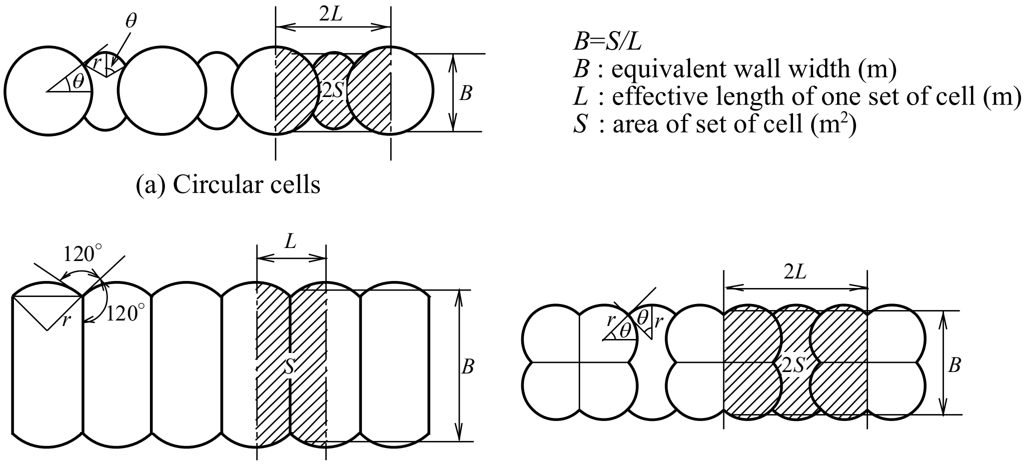
(6) With regard to setting the tide level, refer to **Part II, Chapter 2, 3.6 Design Tide Level Conditions**.

2.9.3 Setting of the Equivalent Wall Width

(1) The equivalent wall width may be used for performance verification. In this case, the equivalent wall width shall be the width of a rectangular virtual wall substituted for the combination of cells and arcs.

(2) The equivalent wall width is the width of a rectangular virtual wall width that is used in place of the combination of cells and arcs to simplify performance verifications (see **Fig. 2.9.6**). The virtual wall is defined in such a way that the area of the horizontal cross section of the virtual wall body becomes the same as that of the actual wall body.

(3) The equivalent wall width is normally determined to satisfy the analysis of the shear deformation of the wall body.



(a) Circular cells

(b) Diaphragm Type Cells (c) Clover Leaf Type Cells

Fig. 2.9.6 Plain View of Cellular-bulkhead Quaywall Structure and Equivalent Wall Width *B*

2.9.4 Performance Verification

(1) Analysis of the Shear Deformation of the Wall Body

① The cell shell and filling of the cellular-bulkhead quaywall usually act as an integrated structure because the filling is constrained in the cell shell. Therefore, the deformation of the cell wall body may be ignored relative to its displacement, and the overall behavior of the cell wall body may be considered to be the same as that of a rigid body. This has been verified by model tests in which the cell wall body did not show significant deformation under loads larger than the external forces that are expected to act on the cell wall body both under a permanent situation and variable situation associated with Level 1 earthquake ground motion. However, when the diameter of the cell is small or the strength of the filling material is extremely low, it may not be possible to satisfy the assumption that the cell wall body is a rigid body. Therefore, it is necessary to examine the strength of the filling against shear deformation due to loads under a permanent situation to remain the deformation of the cell wall body to a negligible level.

② It is normally possible to analyze the shear deformation of the steel-sheet-pile cellular-bulkhead quaywalls with **equations (2.9.3)** and **(2.9.4)** by using the resistant and deformation moments at the cell bottom and the resistant and deformation moments of the soil within the cells at the seabed. Furthermore, an analysis of the shear deformation of steel-plate cellular-bulkhead quaywalls can be performed using **equation (2.9.4)**. Subscripts k and d in the following equations indicate the characteristic value and design value, respectively. For the calculation of the characteristic values, refer to ③ **Calculation of deformation moment**, ④ **Calculation of the resistant moment at the cell bottom**, and ⑤ **Resistant moment of the filling with respect to the seabed**. An appropriate value of 1.20 or higher may be used as the adjustment factor m, and a value of 1.00 may be used as γ_R and γ_S to simplify the calculations.

$$\left.\begin{aligned} & m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\ & S_k = M_{d_k} \\ & R_k = M_{r_k} \end{aligned}\right\}, \qquad \textbf{(2.9.3)}$$

$$\left.\begin{aligned} & m\cdot\frac{S_d}{R_d}\leq 1.0 \quad R_d=\gamma_R R_k \quad S_d=\gamma_S S_k \\ & S_k = M'_{d_k} \\ & R_k = M'_{r_k} \end{aligned}\right\}, \qquad \textbf{(2.9.4)}$$

where

M_r : resistant moment at the cell bottom (kN·m/m);

M_d : deformation moment at the cell bottom (kN·m/m);

M'_r : resistant moment of filling soil at the seabed (kN·m/m);

M'_d : deformation moment at the seabed (kN·m/m);

R : resistance term (kN·m/m);

S : load term (kN·m/m);

γ_R : partial factor by which the resistance term is multiplied;

γ_S : partial factor by which the load term is multiplied;

m : adjustment factor.

③ **Calculation of deformation moment**

(a) The deformation moment to be used in the performance verification of steel-sheet-pile cellular-bulkhead quaywalls shall be the moment at the cell bottom or the seabed due to external forces such as active and passive earth pressures and the residual water pressure above the cell bottom or the seabed. The deformation moment for steel-plate cellular-bulkhead quaywalls shall be the moment at the seabed due to external forces such as active and passive earth pressures and the residual water pressure above the seabed.

(b) Earth pressure is considered only in terms of the horizontal component in the calculation of the deformation moment. The vertical component is not taken into consideration. The vertical force of the surcharge that acts on the cell top is not taken into consideration in the calculation of deformation moment. However, the surcharge is taken into consideration in the calculation of active earth pressure (**Fig. 2.9.7**).

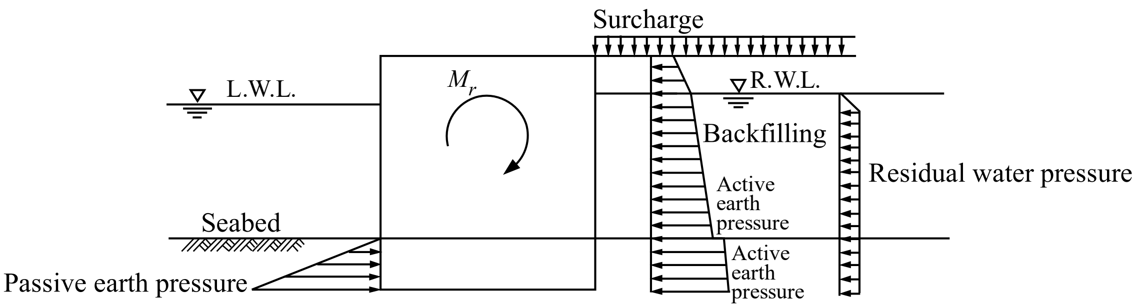


Fig. 2.9.7 Loads and Resisting Forces to be taken into consideration in the Examination of Shear Deformation

④ **Calculation of resistant moment at the cell bottom**

(a) The resistant moment at the cell bottom shall be calculated appropriately in consideration of the structural characteristics of the cell and deformation of the wall.

(b) The result of the model tests[71)] shows that the resistant moment with respect to the cell bottom may be increased by increasing the embedded length ratio D/H (**Fig. 2.9.8**). This can be calculated using **equation (2.9.5)**.

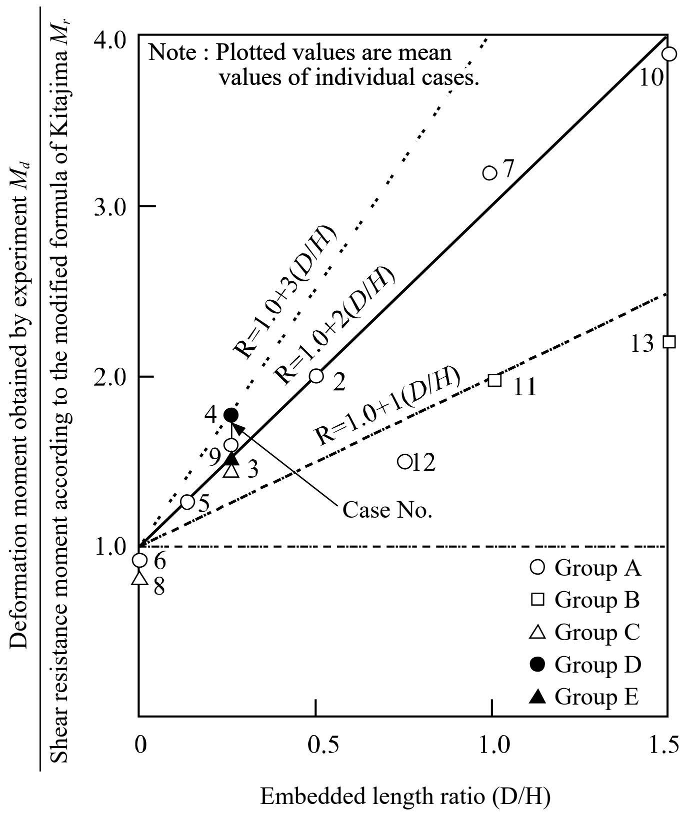


Fig. 2.9.8 Relationship between Resistant Moment and Embedded Length Ratio

$$M_{r_k} = \left(M_{r0_k} + M_{rs_k}\right)\left(1 + \alpha \frac{D}{H}\right), \tag{2.9.5}$$

where

M_r : resistant moment with respect to the cell bottom (kN·m/m);

M_{r0} : resistant moment of the filling with respect to the cell bottom (kN·m/m);

M_{rs} : resistant moment due to the friction force of sheet-pile joints with respect to the cell bottom (kN·m/m);

D : embedded length (m);

H : height from the seabed to the wall top (m) (see **Fig. 2.9.9**);

α : required additional rate against the embedded length ratio (D/H).

It is recommended to use 1.0, which is close to the lowest value found in the test results shown in **Fig. 2.9.8**, for the required additional rate α because the equation given above has been derived on the basis of tests and has not been fully clarified theoretically.

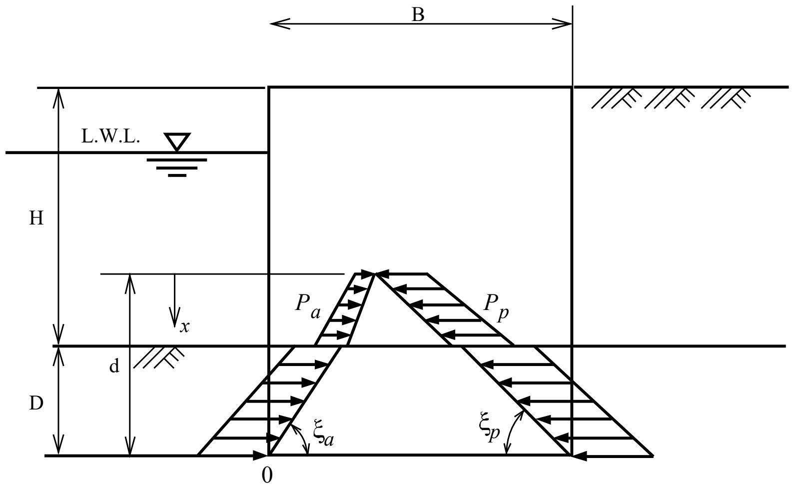


Fig. 2.9.9 Assumed shear surface of filling soil

(c) Equation for Calculating the Resistant Moment of the Filling

In the determination of the resistant moment of the filling at the cell bottom, it is assumed that an active failure surface is generated from the front of the cell bottom, a passive failure surface is generated from the rear, and active and passive earth pressures act on the respective failure surfaces (**Fig. 2.9.9**). The active and passive failure angles, as well as the active and passive earth pressures, may be calculated using Rankine's equations. Subscript k in the equations indicates the characteristic value.

$$\left.\begin{array}{ll} \text{active failure surface} & \zeta_{a_k} = \dfrac{\pi}{4} + \dfrac{\phi_k}{2} \\ \text{passive failure surface} & \zeta_{p_k} = \dfrac{\pi}{4} - \dfrac{\phi_k}{2} \\ \text{active earth pressure} & P_{a_k} = K_a w_k h \ , \quad K_a = \dfrac{1-\sin\phi_k}{1+\sin\phi_k} \\ \text{passive earth pressure} & P_{p_k} = K_p w_k h \ , \quad K_p = \dfrac{1+\sin\phi_k}{1-\sin\phi_k} \end{array}\right\} \qquad (2.9.6)$$

where

ϕ : angle of shear resistance of filling (°);

w : unit weight of the soil (kN/m^3);

h : thickness of the soil layer (m).

The moment caused by the earth pressure acting on the shear surface may be calculated by using **equation (2.9.7)** (see **Fig. 2.9.9**).

$$M_{r0_k} = \int_0^d (P_{p_k} - P_{a_k})(d-x)\frac{2}{3}\tan\phi_k dx \qquad (2.9.7)$$

When the geotechnical constants of the ground and those of the filling differ, **equation (2.9.7)** becomes complex as the failure angle, and the earth pressure level varies from one soil layer to another. However, when there is no significant difference in the angle of shear resistance between the ground and filling or when the embedded length ratio is large and the failure surfaces do not reach the filling portion, the following simplified equation may be used. In the equations below, subscript k indicates the characteristic value.

$$M_{r0_k} = \frac{1}{6} w_{0_k} R_0 H_0{}^3, \tag{2.9.8}$$

$$R_{0_k} = \frac{2}{3} \nu_{0_k}{}^2 \left(3 - \nu_{0_k} \cos\phi_k\right) \tan\phi_k \sin\phi_k, \tag{2.9.9}$$

where

w_0 : equivalent unit weight of the filling that assumes that the unit weight is uniform throughout the filling (normally, w_{0k} = 10 kN/m^3 is used);

H_0 : equivalent wall height measured from the cell bottom (the equivalent wall height is employed to calculate the resistant moment due to the filling by using the equivalent unit weight of the filling, and it is calculated by **equation (2.9.10))**;

ϕ : angle of the shear resistance of the filling (°).

$$H_{0_k} = \frac{1}{w_{0_k}} \sum_i w_{i_k} h_i, \tag{2.9.10}$$

w_i : unit weight of the i-th layer of the filling (kN/m^3);

h_i : thickness of the i-th layer from the cell bottom to the top of the quaywall (m).

$$v_{0_k} = \frac{B}{H_{0_k}}, \tag{2.9.11}$$

B : equivalent wall width (m).

(d) Equation for Calculating the Resistant Moment due to the Friction Force of the Joints of the Sheet Piles

The resistant moment due to the friction force of the joints may be calculated as follows. In the equations below, subscript k indicates for the characteristic value.

$$M_{rs_k} = \frac{1}{6} w_{0_k} R_{s_k} H_{s_k}{}^3, \tag{2.9.12}$$

$$R_{s_k} = \frac{3}{2} v_{sk} f \tan\phi_k, \tag{2.9.13}$$

where

H_{sk} : The equivalent wall height measured from the cell bottom and employed to calculate the resistant moment due to the friction force between the sheet-pile joints when the equivalent unit weight of the filling is used. It is evaluated using **equation (2.9.14)** so that the resultant force of the distributed earth pressure in diagram (a) becomes equal to that of (b) in **Fig. 2.9.10**. In this calculation, 0.5tanϕ can be used as the coefficient of the earth pressure of the filling.

$$H_{s_k} = 2\sqrt{\frac{\sum P_{i_k}}{w_{0_k} \tan\phi_k}}, \tag{2.9.14}$$

P_i : resultant earth pressure of the i–th layer of filling (kN/m) (in this case, the surcharge is ignored);

w_0 : equivalent unit weight of filling (kN/m^3);

ϕ : angle of shear resistance of filling (°).

$$v_{s_k} = \frac{B}{H_{s_k}}, \tag{2.9.15}$$

B : equivalent wall width (m);

f : friction coefficient of the sheet-pile interlock (usually, 0.3 is used).

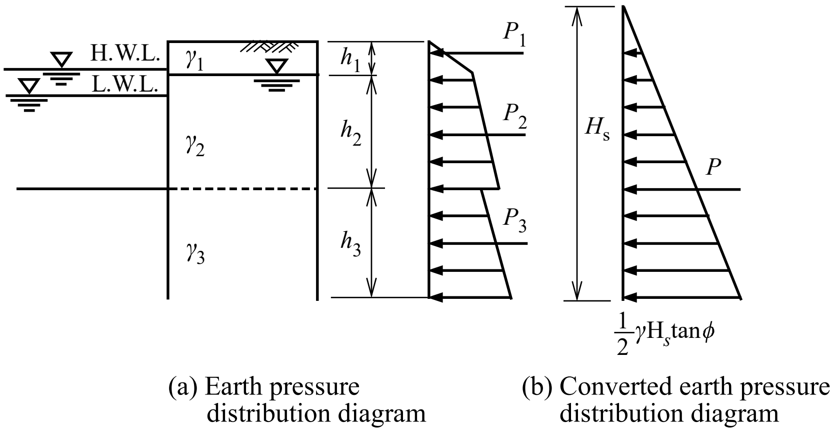


Fig. 2.9.10 Equivalent Wall Height

⑤ **Resistant moment of the filling with respect to the seabed**

(a) The resistant moment with respect to the seabed should be calculated appropriately by taking into consideration the structural characteristics of the cell and the deformation of the wall.

(b) In the calculation of the resistant moment of the filling with respect to the seabed, **equations (2.9.16)** and **(2.9.17)** may be used. Subscript k in the equations indicates the characteristics value:

$$M'_{r_k} = \frac{1}{6} w_{0_k} R_{0_k} H'^{\,3}_{0_k}, \tag{2.9.16}$$

$$R_{0_k}' = v_{0_k}'^{2}(3 - v_{0_k}'\cos\phi')\sin\phi', \tag{2.9.17}$$

where

M_r' : resistant moment of sheet-pile cell with respect to seabed (kN·m/m);

H_0' : equivalent wall height measured from the seabed (the equivalent wall height is employed to calculate the resistant moment due to the filling by using the equivalent unit weight of the filling, and it is evaluated by means of **equation (2.9.18)**);

$$H_{0_k}' = \frac{1}{w_{0_k}} \sum_i w'_{i_k} h'_i \tag{2.9.18}$$

w'_i : unit weight of the filling of the i-th layer above the seabed (kN/m^3);

h'_i : thickness of the i-th layer above seabed between seabed and top of quaywall (m).

$$v_{0_k}' = \frac{B}{H_{0_k}'} \tag{2.9.19}$$

ϕ' : angle of the shear resistance of the filling above seabed (°).

⑥ Increasing the strength of the filling enhances the rigidity of the cell wall. Therefore, the improvement work of filling is effective in increasing the stability of the cell wall.

⑦ Cells containing clayey soil as a filling material are not considered a desirable structure because there are many uncertainties in the behavior of cells and because clayey soil has higher plasticity than sandy soil. Therefore, the use of clayey soil as a filling material should be avoided whenever possible. The results of the analysis using the finite element method show that the stability of a structure made of cells embedded in clayey soil ground depends on the deformation of the ground at the front of the cellular structure and does not depend on the shear deformation of the filling. Therefore, the resistant moments of cells containing clayey soil as a filling may be calculated in the same way as the analysis of those of cells filled with sandy soil.

The resistant moment due to a filling material M_{r0} and the resistant moment due to the friction force between the sheet-pile joints M_{rs} may be calculated by using **equations (2.9.20)** and **(2.9.21)**. Considering that the resistant moments of cells containing clayey soil as a filling material are unknown, it is necessary to examine shear deformation not only at the seabed and at the cell bottom but also on surfaces that are considered dangerous, such as the bottom of a clayey soil layer (**Fig. 2.9.11**). In this manner, the embedded length ratios for the examined surfaces shall be used in the calculation of the embedding effect. In this case, the adjustment factor *m* to be used in **equations (2.9.3)** and **(2.9.4)** for the verification of shear deformation may be set as 1.20 or larger, and partial factors may be set as 1.00.

$$\left.\begin{aligned} M_{r0_k} &= \int_0^d (P_{p_k} - P_{a_k})(d-x)dx \\ P_{a_k} &= K_a w_k h - 2c_k\sqrt{K_a}, \quad K_a = \frac{1-\sin\phi_k}{1+\sin\phi_k} \\ P_{p_k} &= K_p w_k h + 2c_k\sqrt{K_p}, \quad K_p = \frac{1+\sin\phi_k}{1-\sin\phi_k} \end{aligned}\right\}, \qquad \textbf{(2.9.20)}$$

where

ϕ : angle of shear resistance of filling (°);

c : cohesion of filling (kN/m^2);

w : unit weight of soil (kN/m^3);

h : thickness of soil layer being considered (m).

$$M_{rs_k} = \frac{2}{3}(P_{1_k} + P_{2_k} + P_{3_k}) f_k B, \qquad \textbf{(2.9.21)}$$

where

P_1, P_2, P_3 : resultant force in each layer of the filling (kN/m), as shown in **Fig. 2.9.12**;

w_1, w_2, w_c : unit weight of filling material in each layer (kN/m), as shown in **Fig. 2.9.12**;

h_1, h_2, h_c : thickness of each layer of the filling (m), as shown in **Fig. 2.9.12**;

K_s : coefficient of earth pressure of sandy soil used for filling (normally, K_s = 0.6 may be used);

K_c : coefficient of earth pressure of clayey soil used for filling (normally, K_c = 0.5 may be used);

B : equivalent wall width (m);

f : friction coefficient of sheet-pile interlock (normally, f_k = 0.3 may be used).

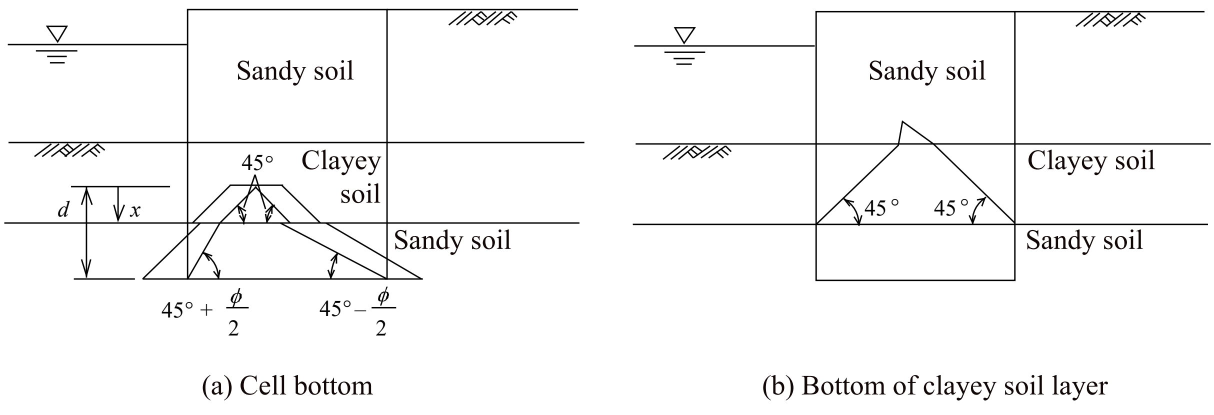


Fig. 2.9.11 Assumption of Shear Surface of Filling

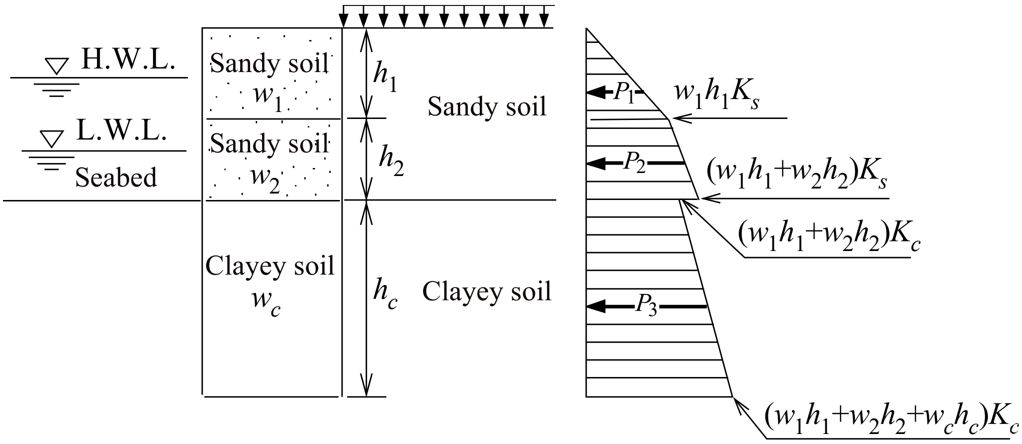


Fig. 2.9.12 Earth Pressure of Filling

(2) Calculation of the amount of deformation of wall body under permanent situations and variable situations associated with Level 1 earthquake ground motion may be carried out based on the following items.

① **General**

(a) In the examination of the stability of the wall as a whole, the subgrade reaction generated against the load and the displacement of the wall are calculated by considering the wall as a rigid body elastically supported by the ground.

(b) Within the elastic range of the ground, the subgrade reaction is calculated as the product of the modulus of subgrade reaction and the displacement. Here, it is considered that the stability of the wall as a gravity wall is obtained when the subgrade reaction and the displacement of the wall do not exceed the respective allowable limits.

② **Modulus of subgrade reaction**

(a) The modulus of the subgrade reaction to be used in the examination of the stability of the wall as a gravity wall shall be set on the basis of the results of soil investigation.

(b) The modulus of subgrade reaction includes the coefficient of lateral subgrade reaction, the coefficient of vertical subgrade reaction, and the horizontal shear modulus at the cell bottom.

(c) The modulus of subgrade reaction may be calculated on the basis of the results of the soil investigation:

1) Coefficient of lateral subgrade reaction

The coefficient of lateral subgrade reaction may be calculated by referring to Yokoyama's diagram[76)] shown in **Part III, Chapter 2**, **3.4.7 Calculation of Pile Deflection Using Chang's Method**.

$$k_{CH} = 2000N, \qquad \textbf{(2.9.22)}$$

where

k_{CH} : coefficient of lateral subgrade reaction (kN/m^3);

N : N value.

The coefficient of lateral subgrade reaction should be calculated for each stratum when the ground consists of the strata of different characteristics.

2) Coefficient of vertical subgrade reaction

For the coefficient of vertical subgrade reaction at the cell bottom, the same value as the coefficient of lateral subgrade reaction at the cell bottom can be used. When the ground consists of the strata of different characteristics, the coefficient of vertical subgrade reaction shall not correspond to the stratum at the cell bottom. However, when there is an extremely soft stratum below the cell bottom, it is necessary to carefully consider the possible effects.

3) Horizontal shear modulus

The horizontal shear modulus at the cell bottom may be calculated by **equation (2.9.23)** using the coefficient of vertical subgrade reaction.

$$k_s = \lambda k_v, \tag{2.9.23}$$

where

k_s : horizontal shear modulus (kN/m^3);

λ : ratio of the horizontal shear modulus to the coefficient of vertical subgrade reaction;

k_v : coefficient of vertical subgrade reaction (kN/m^3).

Past studies suggest the use of λ values in the range of 1/2 to 1/5[77), 78)]. In the case of steel-sheet-pile cellular bulkhead, the value of λ may be set as approximately 1/3.

③ **Calculation of subgrade reaction and wall displacement**

(a) The subgrade reaction acting on the embedded part of the steel-sheet-pile cellular bulkhead and the wall displacement can be calculated on the assumption that the wall subject to the external forces is supported by the horizontal subgrade reaction, vertical subgrade reaction, and horizontal shear reaction at the wall bottom and the vertical frictional resistance along the front and rear of the wall.

(b) Subgrade reaction

1) Horizontal subgrade reaction

Horizontal subgrade reaction may be calculated by **equation (2.9.24)**, but this should not exceed the passive earth pressure intensity calculated in accordance with **Part II, Chapter 4**, **2 Earth Pressure** to prevent the yielding of the ground. The angle of the wall friction used to calculate the passive earth pressure can basically be taken at −15°. **Fig. 2.9.13** illustrates the distribution of the subgrade reaction of a sample case in which the subgrade reaction reaches the passive earth pressure intensity up to a certain depth.

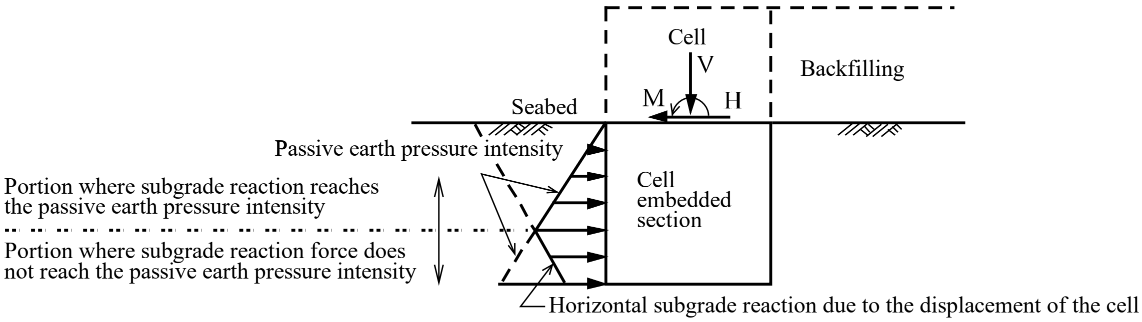


Fig. 2.9.13 Example of Distribution of Horizontal Subgrade Reaction

2) Vertical subgrade reaction

The vertical subgrade reaction at the cell bottom acts in a trapezoidal or triangular distribution. It should be assumed that no tensile stress is generated.

(c) Vertical frictional resistance

It should be assumed that vertical frictional force acts on the front and rear of the wall and the vertical frictional resistance is calculated as the product of the horizontal earth pressure or subgrade reaction and tanδ, where δ denotes the angle of wall friction.

(d) Distribution of external forces

Fig. 2.9.14 shows the standard distribution patterns of the external forces acting on a steel-sheet-pile cellular-bulkhead quaywall.

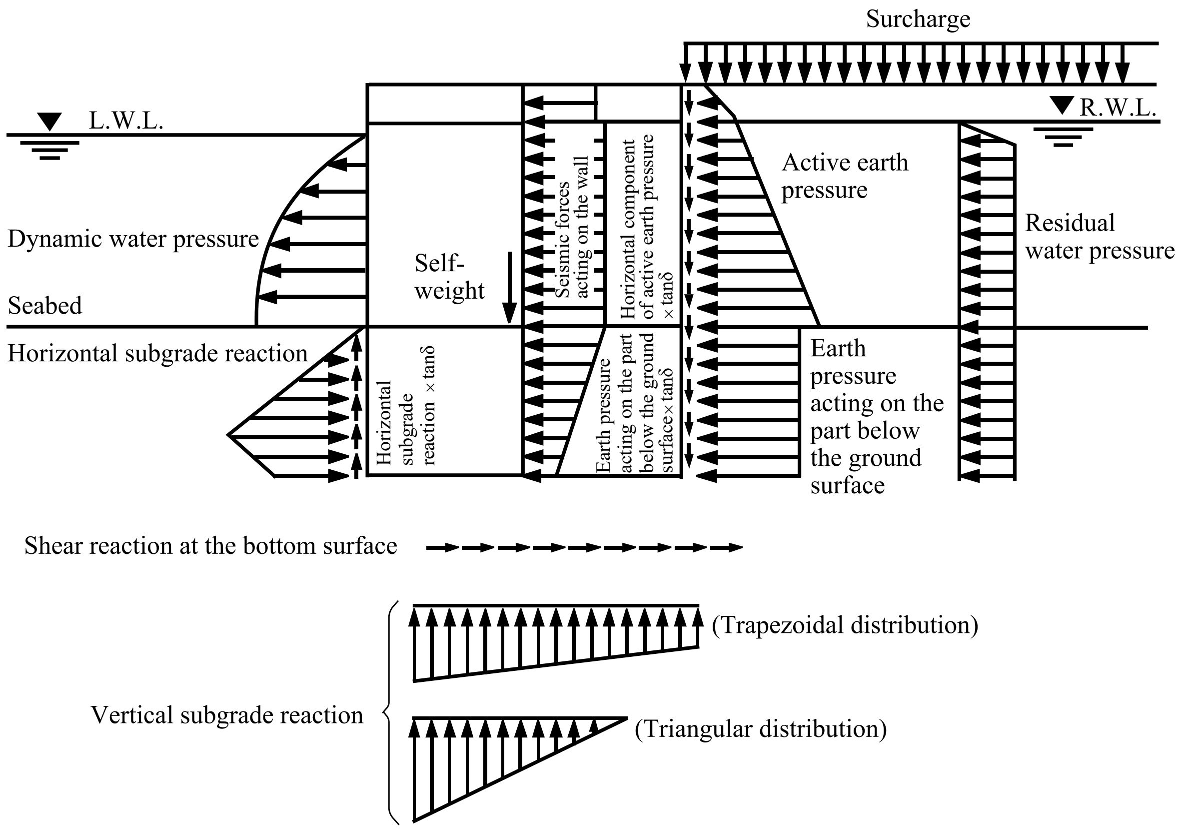


Fig. 2.9.14 Distribution Patterns of External Forces Acting on a Steel Sheet Pile Cellular-bulkhead Quaywall

(e) Displacement modes of cell

As shown in **Fig. 2.9.15**, it is assumed that the cell wall rotates around its center of rotation O, which is horizontally away from the center axis of the cell by distance e and vertically away from the seabed by depth h. When the center of the rotation is located inside the cell, the horizontal subgrade reaction is generated in the rear of the wall for the part below the center of rotation.

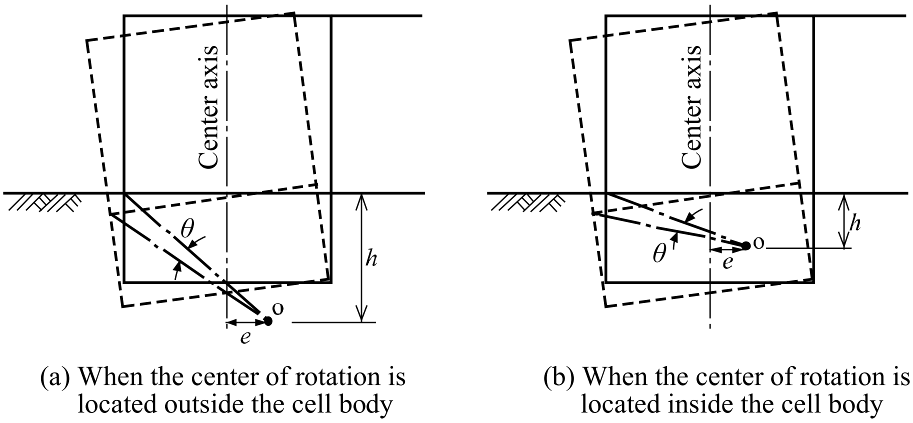


Fig. 2.9.15 Displacement Modes of Cell

(f) Equation for calculating subgrade reaction and wall displacement

Fig. 2.9.16 shows a calculation model for a case in which horizontal force, vertical force, and moment act at the intersection of the ground surface and the center axis of the cell wall and when the ground comprises n layers of soil. The equations for calculating the subgrade reaction and cell wall displacement of the model shown in **Fig. 2.9.16** are as follows. This method does not necessarily accurately calculate the displacement during an earthquake; therefore, caution is needed. In other words, if the embedment length is increased to improve the earthquake-resistant performance, the following methods can overevaluate the deformation in seismic response analysis. For the consistency with seismic response analysis, refer to the related literature[79) 80)].

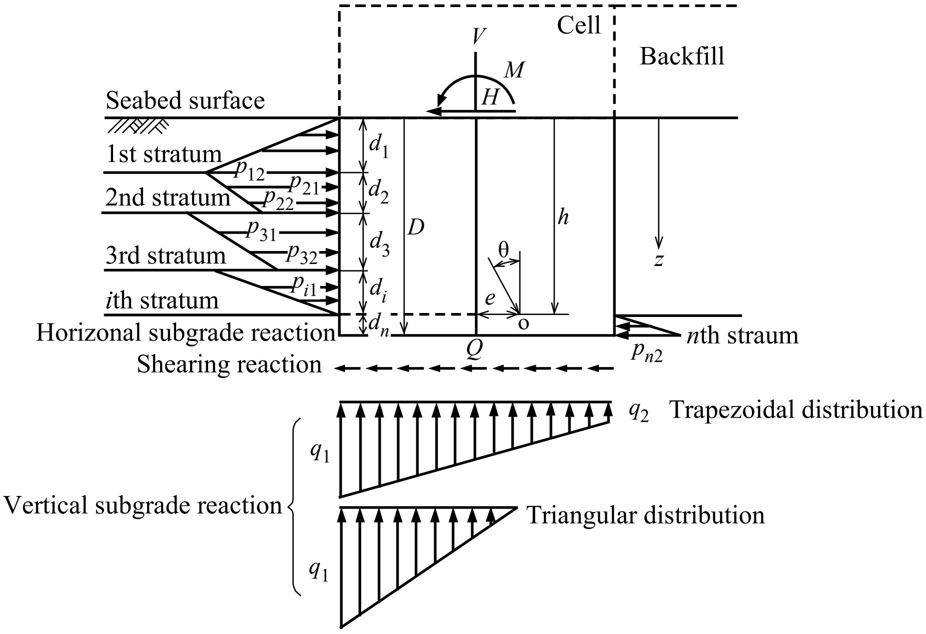


Fig. 2.9.16 Calculation Model

1) When the vertical subgrade reaction acts in a trapezoidal distribution

   i) Horizontal subgrade reaction (kN/m²)

$$\left.\begin{aligned}
p_{12} &= k_{CH_1}\left(h-d_1\right)\theta \\
p_{21} &= k_{CH_2}\left(h-d_1\right)\theta \\
p_{22} &= k_{CH_2}\left(h-d_1-d_2\right)\theta \\
&\vdots \\
p_{i1} &= k_{CH_i}\left(h-\sum_{j=1}^{i-1}d_j\right)\theta \\
p_{i2} &= k_{CH_i}\left(h-\sum_{j=1}^{i}d_j\right)\theta \\
&\vdots \\
p_{n1} &= k_{CH_n}\left(h-\sum_{j=1}^{n-1}d_j\right)\theta \\
p_{n2} &= k_{CH_n}\left(h-\sum_{j=1}^{n}d_j\right)\theta
\end{aligned}\right\} . \qquad \textbf{(2.9.24)}$$

ii) Vertical subgrade reaction (kN/m²)

$$\left.\begin{array}{l} q_1 = k_v(e + B/2)\theta \\ q_2 = k_v(e - B/2)\theta \end{array}\right\} . \tag{2.9.25}$$

iii) Shear reaction force that acts at the wall bottom (kN/m)

$$Q = k_s(h - D)\theta A \ . \tag{2.9.26}$$

iv) Horizontal displacement of the wall (m)

$$\delta = (h - z)\theta \ . \tag{2.9.27}$$

v) Angle of wall rotation (°)

$$\theta = \frac{MK_1 + HK_3}{K_1K_4 - K_2K_3} . \tag{2.9.28}$$

vi) Depth of the center of rotation of the wall (m)

$$h = \frac{MK_2 + HK_4}{MK_1 + HK_3} . \tag{2.9.29}$$

vii) Distance from the wall center axis to the center of rotation of the wall (m)

$$e = \frac{1}{k_v A}\left\{\frac{V}{\theta} - h\sum_{i=1}^{n} k_{CH_i} d_i \tan\left|\delta_i\right| + \sum_{i=1}^{n} k_{CH_i} d_i \left(\sum_{j-1}^{i-1} d + \frac{d_i}{2}\right)\tan\left|\delta_i\right|\right\} . \tag{2.9.30}$$

where

$$\begin{aligned} K_1 &= \sum_{i=1}^{n} k_{CH_i} d_i + k_s A \\ K_2 &= \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2}\right)\right\} + k_s AD \\ K_3 &= \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2} + \frac{B}{2}\tan\delta_i\right)\right\} + k_s AD \\ K_4 &= \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\frac{d_i^2}{3} + \sum_{j=1}^{i-1} d_j \sum_{j=1}^{i} d_j + \frac{B}{2}\left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2}\right)\tan\delta_i\right)\right\} + k_s AD^2 + \frac{1}{12}k_v A^3 \end{aligned} .$$

The angle of wall friction δ is negative for the strata whose horizontal subgrade reaction acts on the front of the wall and is positive for the strata whose horizontal subgrade reaction acts on the rear of the wall.

2) When the vertical subgrade reaction acts in a triangular distribution

The horizontal subgrade reaction, horizontal wall displacement, angle of rotation, and depth of the center of rotation are expressed in the same form as those in 1).

i) Vertical subgrade reaction (kN/m²)

$$q_{1k} = k_v\left(e + \frac{B}{2}\right)\theta . \tag{2.9.31}$$

ii) Shear reaction that acts at the wall bottom (kN/m)

$$Q_k = k_s(h - D)\theta A' , \tag{2.9.32}$$

where

$$A' = e + \frac{B}{2}.$$

iii) Distance between the wall center axis and the center of rotation of the wall (m)

$$e = \sqrt{\frac{2}{k_v}\left\{\frac{V}{\theta} - h\sum k_{CH_i} d_i \tan\left|\delta_i\right| + \sum k_{CH_i} d_i \left(\sum d_j + \frac{d_i}{2}\right)\tan\left|\delta_i\right|\right\}} - \frac{B}{2}\ , \qquad \textbf{(2.9.33)}$$

where

$$K_1 = \sum_{i=1}^{n} k_{CH_i} d_i + k_s A'$$

$$K_2 = \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2}\right)\right\} + k_s A' D$$

$$K_3 = \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2} + \frac{B}{2}\tan\delta_i\right)\right\} + k_s A' D$$

$$K_4 = \sum_{i=1}^{n}\left\{k_{CH_i} d_i \left(\frac{d_i^2}{3} + \sum_{j=1}^{i-1} d_j \sum_{j=1}^{i} d_j + \frac{B}{2}\left(\sum_{j=1}^{i-1} d_j + \frac{d_i}{2}\right)\tan\delta_i\right)\right\}$$
$$+ k_s A' D^2 + \frac{1}{6} k_v A'^2 \left(B - e\right) .$$

The angle of wall friction δ should be negative for the strata whose horizontal subgrade reaction acts on the front of the wall and is positive for the strata whose horizontal subgrade reaction acts on the rear of the wall.

The notations used in Equations in 1) and 2) are as follows:

V : vertical force acting on the wall (kN/m);

H : horizontal force acting on the wall (kN/m);

M : moment acting on the center of the wall at the level of ground surface (kN·m/m) (provided external forces that act on the wall are those for the unit length in the direction along the face line of wall);

D : embedded length (m);

d_i : thickness of each soil layer of the embedded ground (m);

B : equivalent wall width (m);

k_{CHi} : coefficient of the lateral subgrade reaction of each layer of the embedded ground (kN/m^3);

k_v : coefficient of vertical subgrade reaction at wall bottom (kN/m^3);

k_s : horizontal shear modulus at wall bottom (kN/m^3);

A : area of the wall bottom per unit length of the wall in the direction along the face line of wall (m^2/m);

A' : area of the wall bottom per unit length of the wall in the direction along the face line of wall when the value of the vertical subgrade reaction is positive (m^2/m);

④ **Verification of the tilt angle of the wall body**

The allowable value of the tilt angle of the wall body is set by reference to relationships between the amount of deformation of the tops and the amount of damage obtained from earthquake damage reports from the past.[81)] It can be verified that the tilt angle of the wall body calculated by the method described above is equal to or less than the allowable value.

(3) Analysis of Bearing Capacity of Grounds

① For the analysis of the vertical bearing capacity of the grounds at the position of the wall bottom, refer to **Part III, Chapter 2**, **3.2.5 Bearing Capacity for Eccentric and Inclined Actions**.

② When the bearing capacity of shallow foundation for eccentric and inclined load is analyzed by using Bishop's method, the soil above the wall bottom is normally treated as a surcharge.

③ The vertical components of the earth pressure acting on the front and rear of the wall that should be taken into consideration include the following: (a) vertical component of the active earth pressure, (b) friction force due to the earth pressure acting on the embedded section, (c) vertical component of the passive earth pressure, and (d) vertical component of subgrade reaction. The vertical component of earth pressure is considered a positive force when it acts in the same direction as that of the cell weight.

④ When the angle of shear resistance of the soil above the wall bottom is different from that below the wall bottom, it is recommended to use the smaller value as the angle of shear resistance at the wall bottom.

(4) Examination against the Sliding of the Wall

① For the examination of wall stability against sliding, refer to the examination on wall sliding in **Chapter 5**, **2.2 Gravity-Type Quaywalls**.

② The sliding of the wall can be examined using **equation (2.9.34)**. In this equation, γ represents the partial factor for its subscript, and subscripts *d* and *k* stand for the design value and characteristic value, respectively. The values shown in **Table 2.9.2** may be used for the partial factors in the following equation. The mark "—" shown in **Table 2.9.2** indicates that the numerical value in parentheses may be used to simplify the verification calculations.

$$\left.\begin{aligned} & m \cdot \frac{S_d}{R_d} \le 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\ & S_k = k_s \delta b \\ & R_k = \left(W_k + P_{v_k}\right) \tan \phi_k \end{aligned}\right\}, \qquad \textbf{(2.9.34)}$$

where

W : weight of the wall (kN/m);

P_v : vertical component of earth pressure acting on the front and rear of the wall (kN/m);

ϕ : angle of shear resistance of the soil at wall bottom (°);

k_s : horizontal shear modulus at cell bottom (kN/m^2);

δ : cell bottom displacement (m);

b : distribution span of vertical subgrade reaction (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor by which the resistance term is multiplied;

γ_S : partial factor by which the load term is multiplied;

m : adjustment factor.

Table 2.9.2 Partial Factors to be used for Performance Verification of Sliding of Wall

Object of verification	Partial factor by which the resistance term is multiplied γ_R	Partial factor by which the load term is multiplied γ_S	Adjustment factor m
Sliding of the wall (Permanent state)	– (1.00)	– (1.00)	1.20
Sliding of the wall (Variable state of Level 1 earthquake ground motion)	– (1.00)	– (1.00)	1.00

③ The wall weight can be considered a weight that does not include surcharges and can be calculated by subtracting buoyancy.

④ The vertical components of the earth pressure acting on the front and rear of the wall that should be taken into consideration include the following: (a) vertical component of the active earth pressure, (b) vertical component of the passive earth pressure, and (c) vertical component of subgrade reaction. The vertical component of earth pressure is considered a positive force when it acts in the same direction as that of the wall weight.

⑤ When the angle of shear resistance of the soil above the wall bottom is different from that below the wall bottom, it is recommended to use the smaller value as the angle of shear resistance at the wall bottom.

(5) Verification of the Amount of Displacement during the Action of Earthquake Ground Motion by Using the Finite Element Method

For the modeling of cellular-bulkhead quaywalls in the seismic response analysis, refer to related **References 82)** and **83)**.

(6) Verification of Stability against Circular Slip Failure

① When the ground is soft, the examination of stability against circular slip failure shall be made. In the case of cellular-bulkhead quaywalls, it may be assumed that the wall is a rigid body; therefore, the circular slip surface does not go through the inside of the wall.

② For the examination of stability against circular slip failure, refer to **Chapter 5**, **2.2 Gravity-Type Quaywalls**.

(7) Cell layout

① The cells shall be arranged to make the area equal to the area of the wall with the equivalent width obtained by the examination of shear deformation of the wall body and the calculation of subgrade and wall deformation described in (1) and (2) above.

② It is common to use circular cells when examining cells in plain view. The following points should be considered when arranging circular cells.

(a) The wall with the equivalent wall width may be substituted with circular cells by using **equation (2.9.35)** in such a way that the area of the cross section of the circular cells becomes the same as that of the actual wall (**Fig. 2.9.17**).

$$\left.\begin{aligned}
S_1 &= \bigtriangledown ABC2 = \pi R^2 \frac{\theta}{360} 2 = \frac{\pi}{180} R^2 \theta \\
S_2 &= \triangle ACD2 = \frac{1}{2} \overline{AD}\,\overline{CD2} = \frac{R^2}{2} = \sin 2\theta_1 \\
S_3 &= \square CC'D'D = \overline{CD}\,\overline{CC'}2 = 2Rr \cos\theta_1 \sin\frac{\theta_2}{2} \\
S_4 &= \frown CGC' = \bigtriangledown ECGC' - \bigtriangledown ECC' \\
&= \pi r^2 \frac{\theta}{360} - \frac{1}{2} \overline{CC'}\,\overline{EF} = \left(\frac{\pi\theta_2}{360} - \frac{1}{2} \sin\theta_2 \right) r^2
\end{aligned}\right\} , \qquad \mathbf{(2.9.35)}$$

where

$$\theta + \frac{\theta_2}{2} = 90°$$

$$S = (S_1 + S_2 + S_3 + S_4) \times 2$$

$$\therefore B = \frac{S}{L}$$

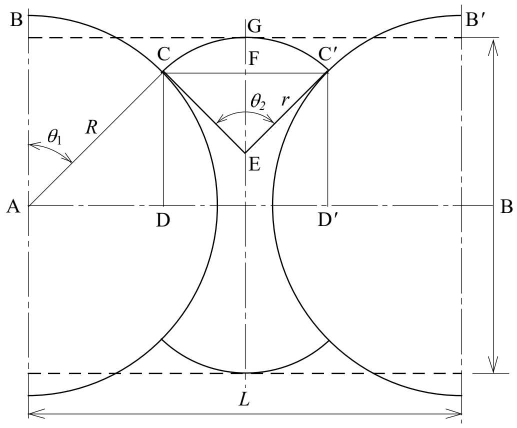


Fig. 2.9.17 Cell Area and Equivalent Wall Width

(b) Cells should be arranged evenly along the total length of the face line of the quaywall wherever possible. In general, it is advisable to set the cell center interval 10% to 15% longer than the cell diameter.

(c) Arcs should be arranged in such a way that they are connected perpendicularly to the wall of the cells. The radius of the arc should be made smaller than that of the cell.

(d) In general, the front tips of the arcs tend to shift forward during and/or after the filling work. Therefore, it is advisable to arrange arcs in such a way that their front surface is located approximately 100 to 150 cm inside the front face line of the cell walls. It is also advisable to arrange cells in such a way that their front face line is located approximately 30 cm inside the design face line of the quaywall.

(8) Analysis of Plate Thickness

① The analysis of the plate thickness of the cells and the arcs is normally performed using **equation (2.9.36)**. In the following equation, subscripts *k* and *d* indicate the characteristic value and design value respectively. The values shown in **Table 2.9.3** may be used for the partial factors in the equation. The mark "－" shown in **Table 2.9.3** indicates that the numerical value in parentheses may be used to simplify the verification calculations.

$$\left.\begin{aligned} & m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\ & S_k = \frac{T_k}{t} \\ & R_k = \sigma_{y_k} \end{aligned}\right\}, \qquad \textbf{(2.9.36)}$$

where

T : tension force acting on the cell (N/mm);

σ_y : yield stress of the cell shell and the arc (N/mm^2);

T : plate thickness of the cell shell and the arc (mm);

R : resistance term (N/mm^2);

S : load term (N/mm^2);

γ_R : partial factor by which the resistance term is multiplied;

γ_S : partial factor by which the load term is multiplied;

Table 2.9.3 Partial Factors to be used for Performance Verification of Plate Thickness of Cells and Arcs

Object of verification	Partial factor by which the resistance term is multiplied γ_R	Partial factor by which the load term is multiplied γ_S	Adjustment factor m
Material yield of cells and arcs (Permanent state)	– (1.00)	– (1.00)	1.67
Material yield of cells and arcs (Variable state of Level 1 earthquake ground motion)	– (1.00)	– (1.00)	1.12

Furthermore, the tensile force acting on the cell may be calculated using **equation (2.9.37).**

$$T_k = \left\{ \left(w_{0_k} H_0' + q_k\right) K_{i_x} + \rho_0 g h_{w_k} \right\} R, \tag{2.9.37}$$

where

T : tensile force acting on the cell (kN/m);

K_i : coefficient of earth pressure of filling;

w_0 : equivalent unit weight of filling (kN/m^3);

$\rho_0 g h_w$: buoyancy due to the difference in water level within the cell and on the front surface (kN/m);

H_0' : equivalent wall height (m);

R : radius of cell (m);

q : surcharge (kN/m^2);

② The equivalent wall height H_0' can be calculated using **equation (2.9.18)** for the calculation of the resistant moment in (1) above.

③ When materials with a large angle of shear resistance, such as gravel, are used for the filling or when no compaction is performed, the characteristic value of the coefficient of earth pressure of filling for the cells can be normally set as 0.6. When the filling is to be compacted, tanϕ can be used as the characteristic value of the coefficient of the earth pressure of filling because the internal pressure of the cell and the angle of shear resistance of the filling become larger. The characteristic value of the coefficient of earth pressure for filling for the arcs can be taken at 1/2tanϕ. This setting is based on the following knowledge obtained from the results of the model tests and field measurements of embedded-type steel-plate cellular blocks[84)]: when the ratio of the cell center interval to the cell diameter is 1.5 or less, the coefficient of earth pressure for filling for the arcs is 1/2 or less of that of the cells.

④ In determining the plate thickness of the cell shells and the arcs of the steel-plate cellular-bulkhead quaywalls, the fabrication, construction, and maintenance aspects must be considered sufficiently. If a corrosion allowance is considered for the cell shells and arcs, the corrosion allowance shall be added to the plate thickness obtained from **equation (2.9.36)** to obtain the plate thickness. **equation (2.9.38)** has been proposed as a method of obtaining the plate thickness of the cell shells necessary for the stresses during driving from tests on the buckling of cylindrical cells and from the construction experience of the past.[85)]

$$t \geq 0.032\left(R\overline{N}D'/E\right)^{0.5}, \tag{2.9.38}$$

where

t : plate thickness of the cell shell (mm);

E : young's modulus of the steel material (kN/mm^2);

R : radius of the cell shell (cm);

$\overline{N}$: average N value of the soils into which the cell is driven;

D' : depth of drive of the cell (cm).

Furthermore, the minimum plate thickness of the cell shell for which there is experience of driving in the past is 8 mm; therefore, it is desirable that the minimum plate thickness is approximately 8 mm.

(9) Verification of T-Shaped Sheet Piles of the Steel-Sheet-Pile Cellular-Bulkhead Quaywalls

① Normally, cells and arcs are connected by using T-shaped sheet piles. A T-shaped sheet pile is a deformed sheet pile to join the cell to arcs (see **Fig. 2.9.18**).

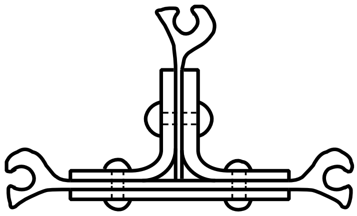


Fig. 2.9.18 T–Shaped Sheet Pile

② The structure of a T-shaped sheet pile shall have sufficient safety against the sheet-pile interlock tension acting on cells and arcs. The standard structures of T-shaped sheet pile are shown in **Figs. 2.9.19** and **2.9.20**.

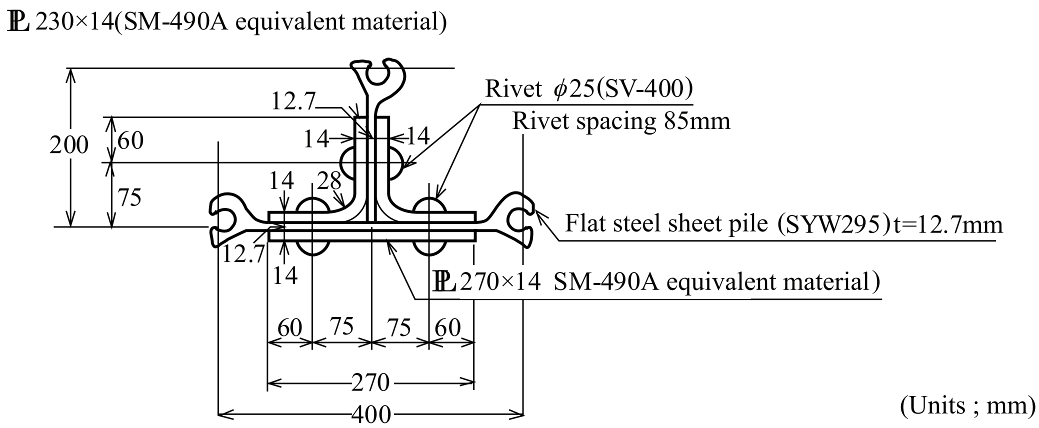


Fig. 2.9.19 Standard Cross Section of T-shaped Sheet Pile for Rivet Connection with Rivet Intervals of 85 mm

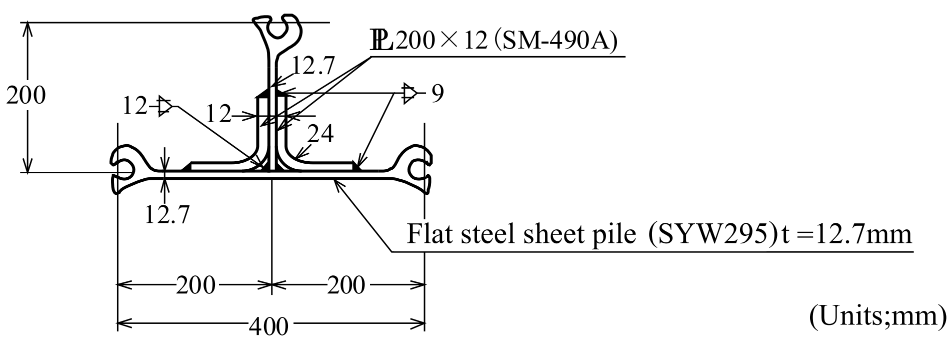


Fig. 2.9.20. Standard Cross Section of T-shaped Sheet Pile for Welding Connection

③ The strength of the cross sections shown in **Figs. 2.9.19** and **2.9.20** has been confirmed by a breaking test where the tensile strength of the joint of the sheet pile in a cell is 3,900 kN/m, and the arc diameter is 2/3 or less of the cell with a tensile strength of 2,600 kN/m. The rivet and welding joints for tests were made in a workshop.

④ **Figs. 2.9.19** and **2.9.20** show the standard cross sections for a flat steel sheet pile with a thickness t = 12.7 mm.

⑤ The strength of the cross sections shown in **Figs. 2.9.21** and **2.9.22** has been confirmed by a breaking test where the tensile strength of the joint of the sheet pile in a cell is 5,900 kN/m, and the arc diameter is 2/3 or less of the cell.

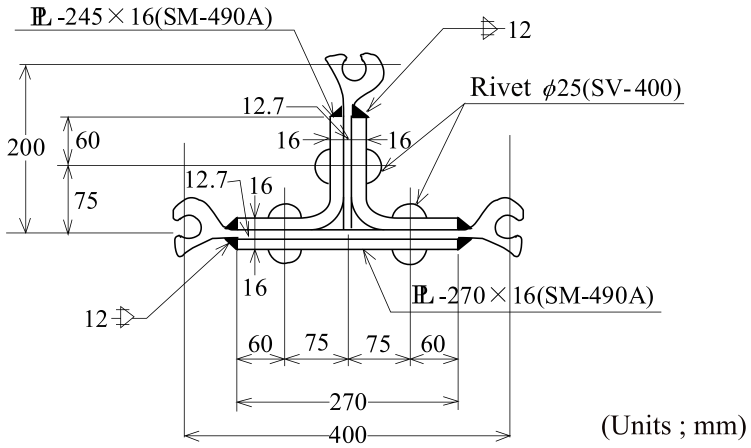


Fig. 2.9.21 Cross Section of T-shaped Sheet Pile for Rivet and Welding Connection with Rivet Intervals of 85 mm

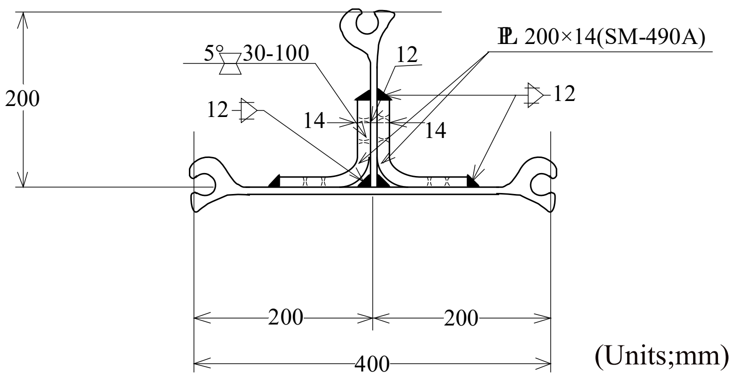


Fig. 2.9.22 Cross Section of T-shaped Sheet Pile for Welding Connection

(10) Joints and Stiffeners for Steel Plate Cellular-Bulkhead Quaywalls

① The joints of cells and arcs shall have a safe structure that resists the maximum horizontal tension acting on the arcs. Cell shells and arcs shall have safe structures that resist stresses that can occur during fabrication, transportation, and construction. The joints of cells and arcs must have a structure that is safe enough to resist tensions acting on the arcs, does not interfere with driving of arcs, and prevents filling and backfilling materials from leaking out of the arcs.

② **Fig. 2.9.23** shows an ordinary shape of a joint.

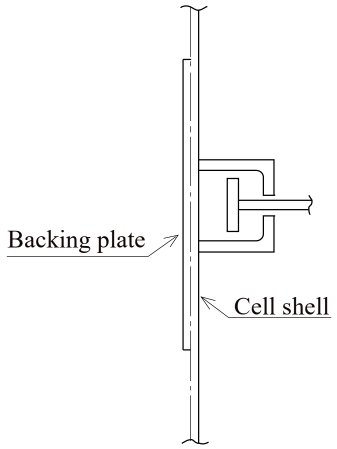


Fig. 2.9.23 Example of Joint Structure

③ To protect cell shells and arcs from stresses that can occur during fabrication, transportation, and construction, it is advisable to equip them with vertical stiffeners (longitudinal ribs), horizontal stiffeners (lateral ribs), and stiffeners that add strength to the top and bottom ends.

2.9.5 Performance Verification of Structural Members

(1) Performance Verification of Superstructure Bearing Piles

① Piles that support the superstructure may be verified for performance as piles subjected to a vertical force, a horizontal force, or a moment.

② Superstructures shall normally be supported by piles alone.

③ For actions on superstructures, refer to **Chapter 5**, **2.8 Quaywalls with Relieving Platforms**.

④ Normally, a horizontal force acting on a superstructure is not directly conveyed to the filling but is conveyed to piles first. It is then conveyed to the filling as a horizontal resistance of the piles. Therefore, piles that support the superstructure may be verified as piles subjected to a vertical force, a horizontal force, or a moment.

⑤ It is common to use vertical piles for supporting a superstructure. The pile-head moment may act on pile heads depending on the degree of constraint to the superstructure. In the performance verification of vertical piles, the superstructure bottom may be considered the ground surface compared with relieving platform piles for which the failure surface under active earth pressure is considered the ground surface.

⑥ For the performance verification of piles, refer to **Part III, Chapter 2**, **3.4 Pile Foundations**.

(2) Performance Verification of Superstructures

① Calculations for the arrangement of reinforcing bars must be made appropriately for the following parts of a superstructure:

(a) Upright walls

(b) Slabs

② The upright walls of a superstructure may be verified for performance as cantilevers supported by slabs and subjected to actions of earth pressure and residual water pressure.

③ Superstructure joints should be positioned at the center of a cell.

④ The rear end of a superstructure should be extended to approximately 1.0 m behind T-shaped sheet piles.

⑤ For the performance verification of slabs, refer to **Part III, Chapter 5**, **2.8 Quaywalls with Relieving Platforms**, excluding the descriptions about the horizontal force transmitted from sheet piles.

⑥ It is advisable to apply concrete jacketing to the upper part of the sheet piles at the front of the cells to prevent sand leakage and to protect corrosion.

2.10 Placement-Type Cellular-Bulkhead Quaywalls

[Public Notice] (Performance Criteria of Cellular-Bulkhead Quaywalls)

Article 52

2 In addition to the provisions of the preceding paragraph, for the performance criteria of placement-type cellular-bulkhead quaywalls, the risk of occurrence of overturning under the variable situation, in which the dominating action is Level 1 earthquake ground motion, shall be equal to or less than the threshold level.

[Interpretation]

11. Mooring Facilities

(6) Performance Criteria of Cellular-Bulkhead Quaywalls

② **Placement-type cellular-bulkhead quaywalls** (Article 26, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 52, Paragraph 2 of the Public Notice

(a) The performance criteria and interpretation of embedded-type cellular-bulkhead quaywalls shall apply correspondingly to placement-type cellular-bulkhead quaywalls, and the following provision shall apply to placement-type cellular-bulkhead quaywalls.

(b) Serviceability shall be the required performance for placement-type cellular-bulkhead quaywalls under a variable situation in which the dominating action is Level 1 earthquake ground motions. The performance verification items for those actions and the standard indexes for setting limit values shall be in accordance with **Attached Tables 11-17**.

Attached Table 11-17 Performance Verification Items and Standard Indexes for Setting Limit Values for Placement-type Cellular-bulkhead Quaywalls under Different Design Situations

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
26	1	2	52	2	—	Serviceability	Variable	Level 1 earthquake ground motion	Self-weight, earth pressure, water pressure. surcharges	Overturning of wall	Action-to-resistance ratio for overturning

(c) In addition to these provisions, provisions regarding Article 22, Paragraph 3 (Scouring and Sand Washing Out) and Article 28 (Performance Criteria for Armor Stones and Blocks) of the Public Notice and their interpretations shall apply as needed.

2.10.1 General

(1) This section is applicable to the performance verification of placement-type cellular-bulkhead quaywalls. The performance verification method described in this section may also be applied to the performance verification of revetments using this structure.

(2) Placement-type cellular-bulkhead quaywalls are cellular-bulkhead quaywalls without an embedded section. In many cases, these quaywalls are constructed on strong foundation ground whose bearing capacity is considered sufficiently large or on ground that has been improved to have sufficient bearing capacity.

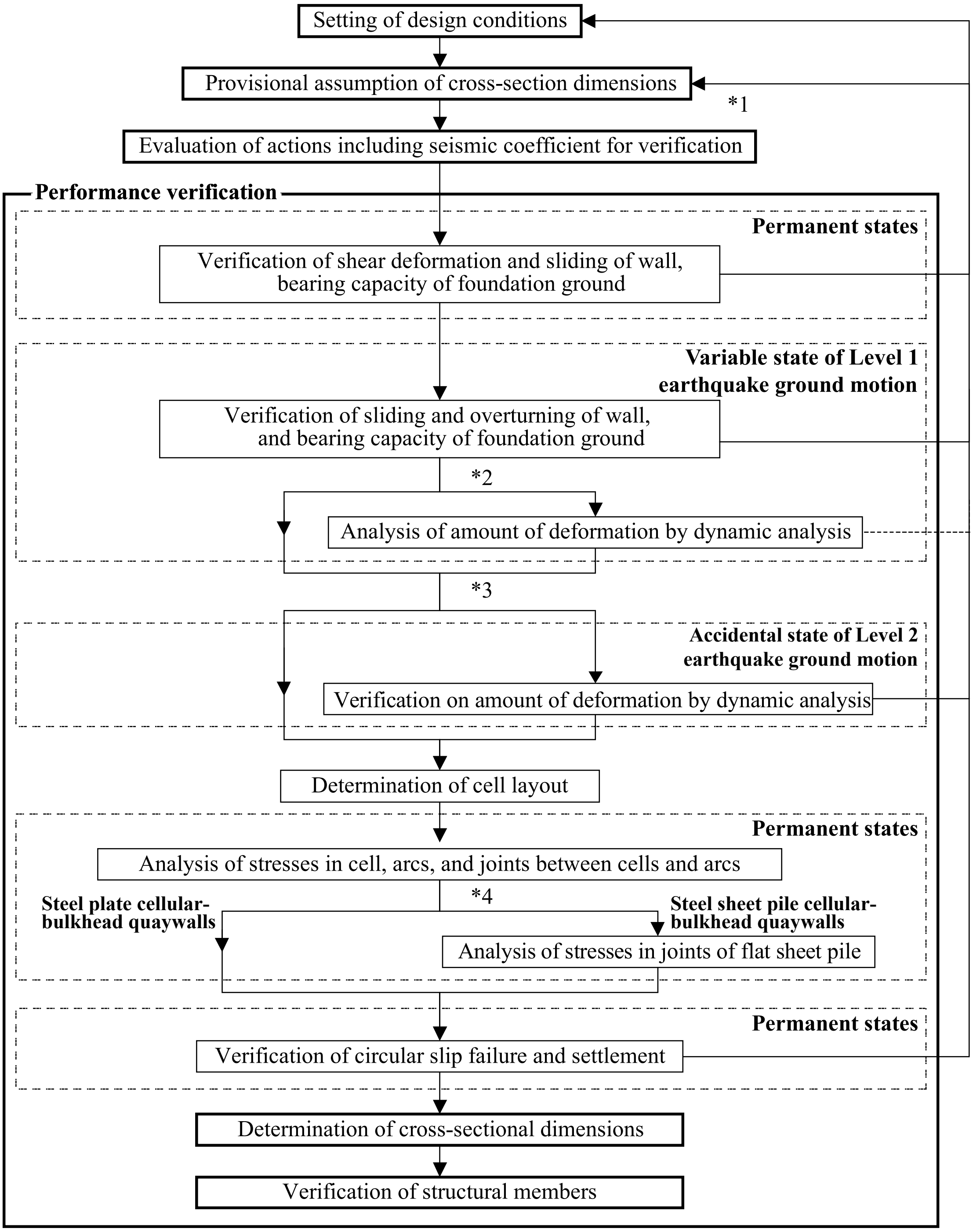
(3) The approaches given in **Part III, Chapter 5**, **2.10.2 Actions** and **Part III, Chapter 5**, **2.10.4 Performance Verification** are simplified approaches, and it is necessary to be careful when adopting these approaches. Highly precise methods (model tests and numerical analysis techniques that can simulate mechanisms) should be used for verification.

(4) **Fig. 2.10.1** shows an example of the sequence of performance verification of placement-type cellular-bulkhead quaywalls. Note that the evaluation of the effects of liquefaction, settlement, and other phenomena caused by earthquake ground motions is not shown in **Fig. 2.10.1**. Therefore, for liquefaction, for example, it is necessary to determine whether there is liquefaction and to consider measures against it appropriately by referring to **Part II, Chapter 7 Ground Liquefaction**. It is possible to verify the variable situation associated with Level 1 earthquake ground motions by using the seismic coefficient method. However, for high-earthquake-resistance facilities, it is desirable to analyze the amount of deformation by using the nonlinear seismic response analysis in consideration of dynamic interaction between the ground and a structure. For placement-type cellular-bulkhead quaywalls that are not categorized as high earthquake-resistance facilities, the verification of the accidental situation associated with Level 2 earthquake ground motions can be omitted.

(5) For other general matters, refer to **Part III, Chapter 5**, **2.9.1 General**.

2.10.2 Actions

For the action on placement-type cellular-bulkhead quaywalls, refer to **Part III, Chapter 5**, **2.9 Embedded-type Cellular-bulkhead Quaywalls**. The characteristic value of seismic coefficient for verification used in the performance verification of placement-type cellular-bulkhead quaywalls under variable situations associated with Level 1 earthquake ground motion shall be appropriately calculated by taking into consideration the structural characteristics. For the purpose of convenience, the characteristic value of seismic coefficient for the verification of placement-type cellular-bulkhead quaywalls may be calculated in accordance with **Part III, Chapter 5**, **2.2 Gravity-type Quaywalls**.



*1: The evaluation of the effect of liquefaction is not shown; therefore, this must be separately considered.
*2: Analysis of the amount of deformation due to Level 1 earthquake ground motion may be performed by dynamic analysis when necessary.
For high-earthquake-resistance facilities, the analysis of the amount of deformation due to Level 1 earthquake ground motion should also be performed by dynamic analysis.
*3: For high-earthquake-resistance facilities, verification is performed for Level 2 earthquake ground motion.
*4: For steel-sheet-pile cellular-bulkhead quaywalls, verification is performed for the joints of a flat sheet pile.

Fig. 2.10.1 Example of the Sequence of Performance Verification of Placement-type Cellular-bulkhead Quaywalls

2.10.3 Setting of Cross-Sectional Dimensions

The width of the wall structure used in performance verification may be the equivalent wall width, which is an imaginary wall width obtained by replacing cells and arcs with a rectangular wall structure. For the equivalent wall width, refer to **Part III, Chapter 5, 2.9 Embedded-type Cellular-bulkhead Quaywalls**.

2.10.4 Performance Verification

(1) Examination of the Shear Deformation of the Wall

① The examination of the shear deformation of the wall body shall be made in accordance with the performance verification methods described in **Part III, Chapter 5**: **2.9 Embedded-type Cellular-bulkhead Quaywalls**. The resistant moment shall be calculated appropriately in consideration of the structural characteristics of the cellular-bulkhead and the deformation of the wall. The deformation moment to be used in the verification shall be the moment at the seabed due to external forces acting on the wall body above the seabed, including active earth pressure and residual water pressure.

② When the deformation of the steel plate cells is not allowed, i.e., when the horizontal displacement of the cell top is approximately less than 0.5% of the cell height, the resistant moment against deformation can be normally calculated using **equations (2.10.1)** and **(2.10.2)**. Subscript k in the following equations means the characteristic value.

$$M_{rd_k} = \frac{1}{6} w_{0_k} H_d'^3 R \tag{2.10.1}$$

$$R = \nu^2 (3 - \nu \cos\phi_k) \sin\phi_k \tag{2.10.2}$$

where

M_{rd} : resistant moment of the cell (kN･m/m);

H'_d : equivalent wall height used in the examination of cell deformation (m);

w_0 : equivalent unit weight of filling (kN/m^3) (normally, w_0 = 10 kN/m^3);

ν : ratio of the equivalent wall width to the equivalent wall height used in the examination of cell deformation $\nu = B/H_d'$

B : equivalent wall width (m);

ϕ : angle of shear resistance of filling (°).

③ In the calculation of resistant moment, the equivalent wall height of the cell H'_d is calculated by **equation (2.10.3)**. The height H'_d is the height above the seabed.

$$H_d' = \left(w_k' / w_{0_k}\right) H_w + \left(w_{tk} / w_{0_k}\right)\left(H_d - H_w\right) \tag{2.10.3}$$

where

H'_d : height from the seabed to the top of the quaywall (m);

H_w : height from the seabed to the residual water level (m);

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

w' : saturated unit weight of saturated filling (kN/m^3);

w_0 : equivalent unit weight of the filling (kN/m^3) (normally, w_0= 10 kN/m^3).

In the calculation of the equivalent wall height H'_d, a surcharge may be ignored similar to the case of resistant moment calculation discussed for the performance verification in **Part III, Chapter 5**, **2.9 Embedded-type Cellular-bulkhead Quaywalls**.

④ When the filling material can be regarded as uniform, the height H_d of the quaywall top above the seabed can be used in place of the equivalent wall height H'_d of **equation (2.10.1)**.

(2) Examination of the Sliding of the Wall

For the examination of sliding, refer to Part III, Chapter 5, 2.9 Embedded-type Cellular-bulkhead Quaywalls.

(3) Examination of the Overturning of the Wall

① In the calculations to examine the stability of a cell wall body against overturning, the stability of the cell shall be examined against the external forces acting above the cell bottom, including earth pressure, residual water pressure, and earthquake ground motion.

② For the performance verification for overturning, **equation (2.10.4)** can normally be used. In the equation, subscripts k means the characteristic and d means design values, respectively. The values shown in **Table 2.10.1** may be used for the partial factors in the following equation. The mark "－" shown in **Table 2.10.1** means that the numerical value in parentheses may be used to simplify the verification calculations.

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

$$S_k = M_{dk} \tag{2.10.4}$$

$$R_k = M_{rdk}$$

where

M_{rd} : resistant moment against the overturning of the steel plate cell (kN·m/m);

M_d : deformation moment of the cell bottom (kN·m/m).

Table 2.10.1 Partial Factors to be Used for Performance Verification of Overturning of Wall

Object of verification	Partial factor by which the resistance term is multiplied γ_R	Partial factor by which the load term is multiplied γ_S	Adjustment factor m
Overturning of the wall (Variable state of Level 1 earthquake ground motion)	– (1.00)	– (1.00)	1.10

③ The resistant moment of the cell against overturning can be calculated using **equations (2.10.5)** and **(2.10.6)**.

$$M_{rd_k} = \frac{1}{6} w_{0_k} {H'_d}^3 R_t \tag{2.10.5}$$

$$\left.\begin{aligned}
R_t &= \nu'^2 (3 - \nu' \cos\phi_k) \sin\phi_k + 3(\alpha^2 + \beta^2) + 6\nu\beta \\
\alpha &= K_a \tan\delta_k \\
\beta &= K_a \tan\delta_k (\nu'/2)(4 - \nu' \cos\phi_k) \tan\phi_k \tan\delta_k \\
\nu' &= \nu - (\alpha + \beta)
\end{aligned}\right\} \tag{2.10.6}$$

where

M_{rd} : resistant moment of the steel cell plate against overturning (kN·m/m);

H' : equivalent wall height of the cell to obtain the resistant moment against overturning (m);

v : ratio of the equivalent wall width to the equivalent wall height of the cell $v = B/H'$

B : equivalent wall width of the cell (m);

δ : angle of wall friction of the filling material (°) (normally, δ = 15° is used);

K_a : coefficient of the active earth pressure of the filling material.

④ The equivalent wall height H' used to calculate the resistant moment against the overturning of the cell can be calculated using **equation (2.10.7).**

$$H' = \left(w'_k / w_{0_k}\right) H_w + \left(w_{tk} / w_{0_k}\right)\left(H_d - H_w\right), \qquad \textbf{(2.10.7)}$$

where

H' : equivalent wall height of the cell used to calculate the resistant moment against overturning (m);

H_d : distance from the cell bottom to the top of the quaywall (m);

H_w : distance from the cell bottom to the residual water level (m);

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

w' : saturated unit weight of saturated filling (kN/m^3);

w_0 : equivalent unit weight of filling (kN/m^3); normally, w_0= 10 kN/m^3.

⑤ In general, the filling of a cell used as a mooring facility is not uniform because the major portion of such filling is under the water and is subjected to buoyancy. Therefore, the equivalent wall height is used here in the calculation of the resistant moment of the cell against deformation. When the filling material can be considered uniform, the total wall height of the cell H may be used in the same calculation in place of the equivalent wall height H' of **equation (2.10.7)**.

Although the actions of the filling against overturning are not uniform,[85)] given that the main part of the filling's resistance is the hanging effect, the margin of error is minimal, and safety is ensured even when the ratio of the equivalent wall width to the equivalent wall height v is used as that in **equation (2.10.6)**. In this case, a surcharge can be ignored.

⑥ The overturning moment is the moment at the cell bottom due to the external forces acting above the bottom. The equivalent wall height of the cell H' used in the calculation of the resistant moment should be the height above the cell bottom.

(4) Examination of Bearing Capacity on Cell Front Toe

① The maximum front toe reaction force on the cell shell front toe shall be calculated appropriately in consideration of the effect of the filling material acting on the front wall of the cell.

② The maximum front toe reaction force on the cell shell front toe may be obtained from **equation (2.10.8)**. Subscript k means the characteristic value.

$$V_{t_k} = \frac{1}{2} w_k H^2 \tan^2 \phi_k \qquad \textbf{(2.10.8)}$$

where

V_t : maximum front toe reaction force on the cell shell front toe (kN/m);

w : unit weight of filling (kN/m^3);

H : total wall height of the cell (m);

ϕ : angle of the shearing resistance of the filling (°).

Equation (2.10.8) calculates the weight of the filling weighing down on the front wall, with the product of the coefficient of earth pressure of the filling and the wall surface friction coefficient given by $\tan^2 \phi$. Therefore, when the filling is not uniform, it is necessary to perform the calculation for the same domain as the earth pressure calculation.

③ The wall height H should normally be considered the height of the cell top above the cell bottom. However, when the superstructure of the cell is supported by foundation piles, it may be considered the height of the bottom of the superstructure above the cell bottom.

④ **Equation (2.10.8)** represents the cell shell front toe reaction force when the overturning moment is roughly equal to the overturning resistant moment of **equation (2.10.5)**. Without the occurrence of overturning, the reaction force is smaller than the value obtained from **equation (2.10.8)**. According to a model test, the maximum front toe reaction force V_t is nearly proportional to the overturning moment.[86] Therefore, a reaction force without the occurrence of overturning should be calculated using **equation (2.10.9)**.

$$V_k = Vt_k (M_k / M_{r0_k}), \qquad \textbf{(2.10.9)}$$

where

V : front toe reaction force of the cell shell corresponding to overturning moment M (kN/m);

M : overturning moment (kN·m/m);

M_{r0} : resistant moment against overturning (kN·m/m).

Hence, the use of a larger cell shell radius makes the cell safer against overturning by increasing the resistant moment M_{r0} while reducing the front toe reaction force V.

⑤ For the bearing capacity of the ground, refer to the bearing capacity in **Part III, Chapter 2**, **3.2 Shallow Spread Foundations**.

⑥ When providing a footing at the cell shell bottom to reduce the reaction force of the foundation, it is favorable to locate the footing outside the cell shell.[85]

(5) Examination of Plate Thickness

① Examination of the plate thickness of the cells and arcs may be performed in accordance with the examination of plate thickness given for the performance verification in **Part III, Chapter 5, 2.9 Embedded-type Performance Verification of Cellular-bulkhead Quaywalls**.

② Forces acting on the cell shell include the horizontal tension due to filling, the compressive stress due to subgrade reaction near the front toe bottom, and the shear stress due to deformation moment near the side wall. However, the compressive stress due to the front toe reaction force does not pose a problem for stability because it is much smaller than the tensile stress and because there is a difference in the point where the maximum stress occurs. Care should be taken when providing a footing outside the cell bottom because a significantly large stress will occur owing to the bending moment. In a model test, no buckling occurred at the front toe bottom before the occurrence of overturning failure[85]. According to the results of an experiment on a cell shell with a small thickness, no local buckling occurred near the bulkhead, but a significant shear stress occurred at the side wall at the time of overturning because the resistance against overturning was dominated by the hanging effect of filling. However, the hanging effect of filling was not large before the occurrence of deformation. Therefore, it is considered that the shear stress of the cell shell is relatively small. The effect of the shear stress may be ignored because it does not matter if the stress at the cell shell exceeds the limit value at the time of overturning.

③ From the point of view of cell shell stiffness and corrosion, a minimum cell shell thickness should be more than 6 mm is necessary.

2.10.5 Performance Verification of Structural Members

For the performance verification of the structural members of placement-type cellular-bulkhead quaywalls, refer to the performance verification of the structural members in **Part III, Chapter 5**, **2.9 Embedded-type Cellular-bulkhead Quaywalls**.

2.11 Upright Wave-Absorbing-Type Quaywalls

2.11.1 General

(1) This section is applicable to upright wave-absorbing-type quaywalls, but it may also be applied to the performance verification of revetments.

(2) The upright wave-absorbing-type quaywall shall be structured so as to have the required capability of wave energy dissipation and shall be located at strategic positions to enhance the calmness within the harbor.

(3) Waves within a harbor are the result of the superposition of the waves entering the harbor through the breakwater openings, transmitted waves over the breakwaters, wind generated waves within the harbor, and reflected waves inside the harbor. By using the quaywalls of the wave-absorbing type, the reflection coefficient can be reduced to 0.3 to 0.6 from 0.7 to 1.0 of the upright walls. To improve the harbor calmness, it is important to design the alignments of breakwaters in a careful manner. The suppression of reflected waves through the provision of wave energy absorbing structures within the harbor is also an effective means of improving calmness. It is effective to apply energy absorbing structures to reflecting surfaces, particularly those that directly receive waves entering the harbor through the breakwater openings and those that receive waves coming from many directions.

(4) When it is necessary to improve harbor calmness for small craft facilities, energy absorbing structures should be installed in mooring facilities for small crafts and facilities that direct reflected waves toward such areas.

(5) Determination of Structural Type

① Upright wave-absorbing type quaywalls include upright wave-absorbing block type and upright wave-absorbing caisson type. An appropriate type of structure shall be selected depending on the scale of the mooring facility and wave conditions at the location.

② Upright wave-absorbing block type quaywalls are constructed by stacking layers of various shapes of wave-absorbing blocks. This type is normally used to build relatively small quaywalls. The quaywall width is determined by stability calculation as a gravity-type quaywall.

③ Upright wave-absorbing caisson-type quaywalls include slit-wall caisson type and perforated-wall caisson type. This type is normally used to build large quaywalls. The wave-absorbing performance can be enhanced by optimizing the aperture rate of the front slit wall, the water chamber width, and the others parameters for the given wave conditions.

④ An upright wave-absorbing-type quaywall normally consists of a permeable front wall, a water chamber, and an impermeable rear wall. To improve harbor calmness, this type of quaywall is designed to reduce the reflection rate via the energy losses mainly caused by the horizontal jet flow of water passing through the front wall, the resistance due to roughness inside the structure, and the occurrence of phase difference. The wave conditions to be considered in performance verification may include extreme waves for the examination of the stability of a facility and regular or extreme waves for the examination of the wave-absorbing performance.

⑤ The reflection coefficient is preferably determined by means of a hydraulic model test whenever possible, but it may also be determined in accordance with **[Facilities]** in **Chapter 4: 3.4 Gravity-Type Breakwaters (Breakwater Covered with Wave-dissipating Blocks)** and **[Facilities]** in **Chapter 4**: **3.5 Gravity-Type Breakwaters (Upright Wave-absorbing Block Breakwaters)**. **Figs. 2.11.1** and **2.11.2** show an example of the results of the model tests on slit-wall caissons and perforated-wall caissons with round holes[87) 88) 89)].

⑥ It is recommended that the crown height of the wave-dissipating work of an upright wave-absorbing block-type quaywall is set as high as 0.5 times the significant wave height or more above the monthly mean highest water level, and the bottom height of the wave-dissipating work is set twice as deep as the significant wave height or more below the monthly mean lowest water level.

⑦ The area of wave-dissipating works for upright wave-absorbing caisson-type quaywalls may be determined in the same way as that for upright wave-absorbing block-type quaywalls. It is advisable to examine the effects of ceiling slabs and air holes on the reflection rate by conducting a hydraulic model test.

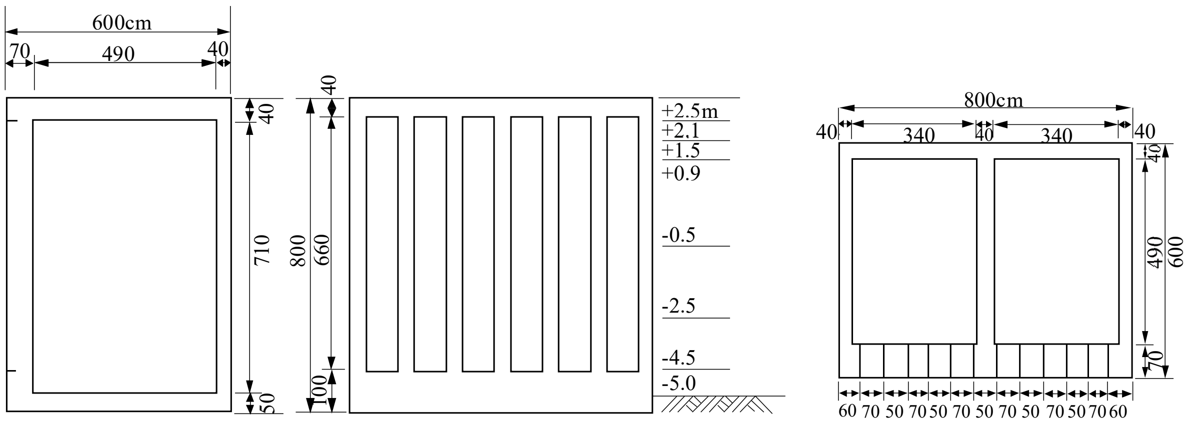


Fig. 2.11.1(a) Slit-wall Caisson Type Wave-absorbing Quaywall

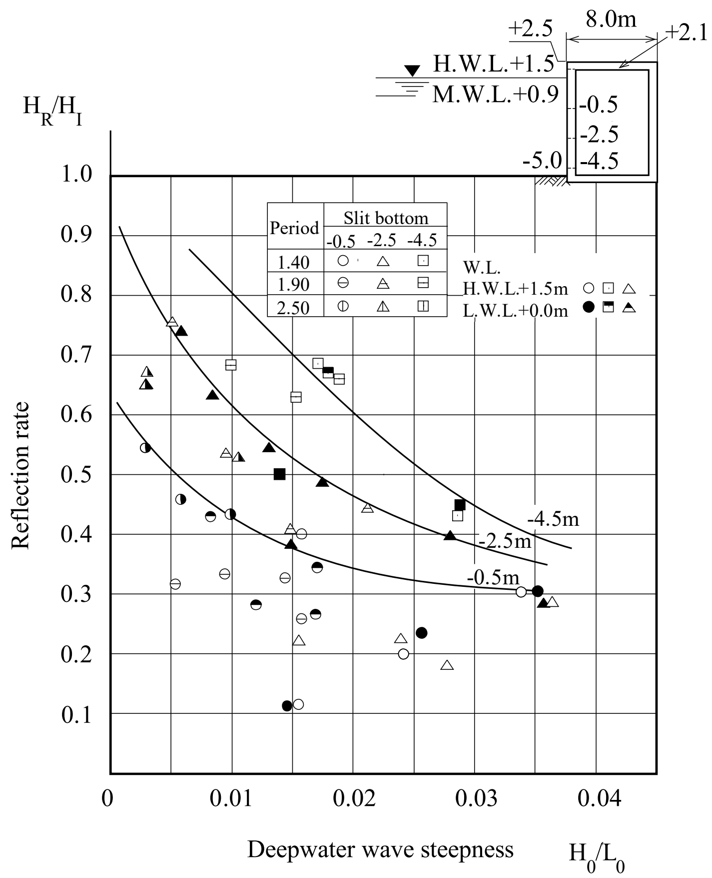


Fig. 2.11.1(b) Slit-wall Caisson Type Wave-absorbing Quaywall
(Relationship between Reflection Rate and Slit Length without Filling Blocks)

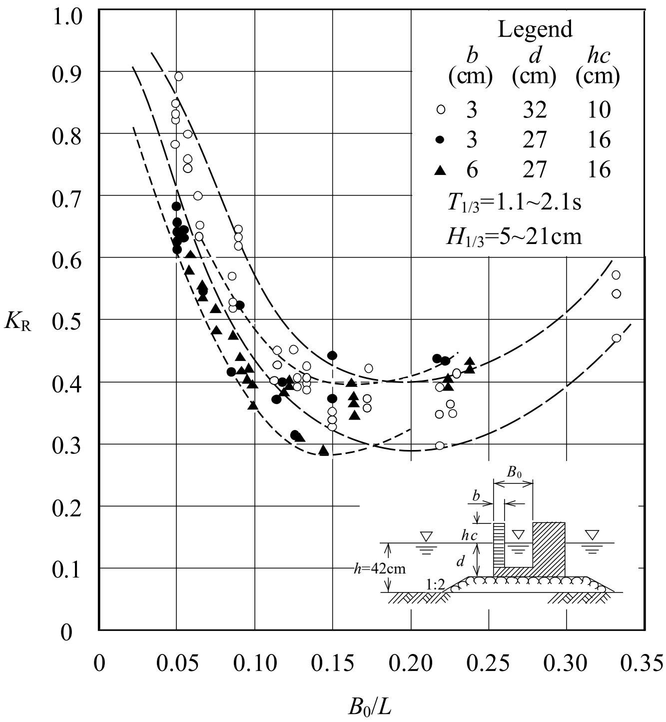


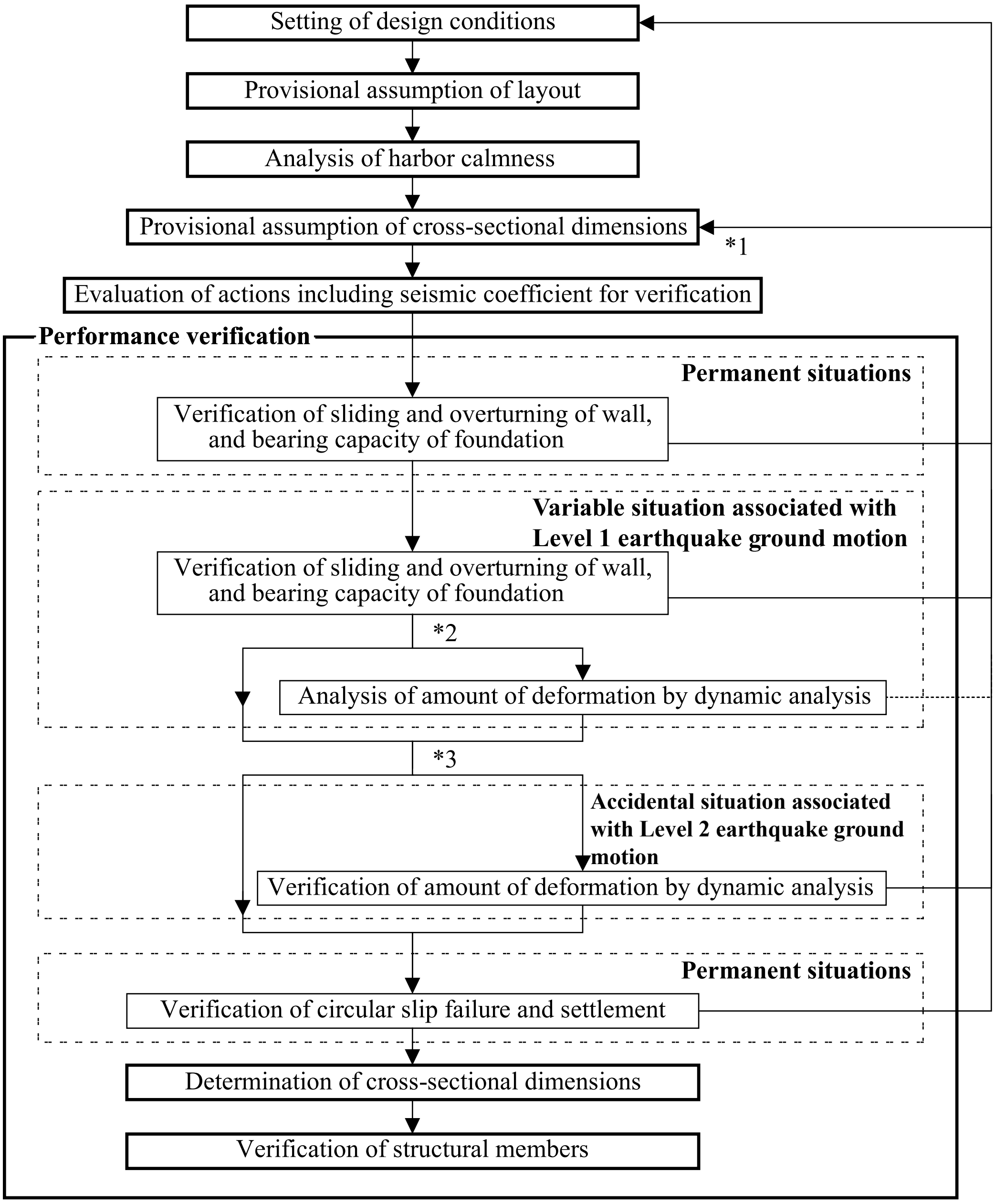
Fig. 2.11.2 Random Wave Reflection Rate of Perforated-wall Caisson with Round Holes[87)]

2.11.2 Performance Verification

(1) **Fig. 2.11.3** shows an example of the sequence of the performance verification of upright wave-absorbing-type quaywalls. The evaluation of the effects of liquefaction, settlement, and other phenomena caused by earthquake ground motions is not shown in **Fig. 2.11.3**. Therefore, for liquefaction, it is necessary to determine whether there is liquefaction and to consider measures against it appropriately by referring to **[Actions and Material Strength Requirements]** in **Chapter 7: Ground Liquefaction**. It is possible to verify the variable situation associated with Level 1 earthquake ground motions by using the seismic coefficient method. However, for high earthquake-resistance facilities, it is desirable to analyze the amount of deformation by using nonlinear seismic response analysis in consideration of the dynamic interaction between the ground and a structure. For upright wave-absorbing-type quaywalls that are not categorized as high earthquake-resistance facilities, the verification of the accidental situation associated with Level 2 earthquake ground motions can be omitted.

(2) The characteristic value of the seismic coefficient for the verification used in the performance verification of upright wave-absorbing-type quaywalls for the variable situations associated with Level 1 earthquake ground motion shall be appropriately calculated by taking the structural characteristics into consideration. For convenience, the characteristic value of the seismic coefficient for the verification of upright wave-absorbing-type quaywalls may be calculated in accordance with that for the gravity-type quaywalls shown in **Chapter 5**: **2.2 Gravity-type Quaywalls**.

(3) It is advisable to examine the wave-absorbing performance by conducting a hydraulic model test. When doing so, it should be noted that the wave-absorbing performance varies depending on the tide level, in addition to the characteristics of incident waves.



*1: The evaluation of the effect of liquefaction is not shown; therefore, this must be separately considered.

*2: The analysis of the amount of deformation due to Level 1 earthquake ground motion may be performed by dynamic analysis when necessary.
For high-earthquake-resistance facilities, the analysis of the amount of deformation due to Level 1 earthquake ground motion should also be performed by dynamic analysis.

*3: For high-earthquake-resistance facilities, verification is performed for Level 2 earthquake ground motion.

Fig.2.11.3 Example of the Sequence of Performance Verification of Upright Wave-absorbing Type Quaywalls

[References]

1) About the Implementation of Hydrographic Survey of Channels Associated with Port Development, Ports and Harbours Bureau Notice No.61, 1972 (in Japanese)

2) Ports and Harbours Association of Japan: Port Construction Design Manual, p.215, 1959 (in Japanese)

3) Miyazaki, M.: Construction Limits of Quay Wall, Journal of JSCE Vol.36 No.8, pp.26-27, 1951 (in Japanese)

4) C. Zimmerman, H. Schwarze, N. Schulz, and S. Henkel: International Conference on Coastal and Port Engineering in Developing Countries, 25/29 September, Rj, Brazil, pp.2437-2451, 1995.

5) PIANC:Guidelines for the Design and Constructions of Flexible Revetments Incorporating Geotextiles for Inland Waterways, Supplement to Bulletin No.57, 1987.

6) Takano, H., S. Iyama, K. Sakata, A. Fujii, M. Miyata and S. Nishioka: A Study on the Considerations in the Maintenance and Repair based on the Assessment Results of Port Facilities, Technical Note of National Institute for Land and Infrastructure Management, No.921, 2016 (in Japanese)

7) Ports and Harbours Bureau, Ministry of Land, Infrastructure and Transport: Guidelines for Maintenance and Management of Green Port Structures. http://www.mlit.go.jp/common/001048849.pdf, 2014. (2020. November. 26)

8) Kishitani, K., Y. Kunishige, S. Hirano and M. Yamashita: Design Method and Characteristics of Wedged Caisson Quay Walls, Proceedings of the 53th Annual Meeting of JSCE, 1998 (in Japanese)

9) Morita, T., G. Kimura, K. Shiramizu and H. Tanaka: Consideration on earthquake behavior of oblique bottom caisson type wharf, Proceedings of the 53th Annual Meeting of JSCE, 1998 (in Japanese)

10) Nagao, T., N. Iwata, M. Fujimura, N. Morishita, H. Sato and R. Ozaki: Seismic Coefficients of Caisson Type and Sheet Pile Type Quay Walls against Level 1 Earthquake Ground Motion, Research Report of National Institute of Land and Infrastructure Management No.310, 2006 (in Japanese)

11) Nagao, T. and N. Iwata: Seismic Coefficients of Caisson Type and Sheet Pile Type Quay Walls against Level 1 Earthquake Ground Motion, Journal of Structural Engineering, 2007 (in Japanese)

12) Fisheries Agency: Reference for the Design of Fisheries Facilities, 2015 (in Japanese)

13) Furutoi, M and T. Katayama: Study on Measurement of Residual Waver Levels, Technical Note of the Port and Harbour Research Institute, No.1115, 1971 (in Japanese)

14) Coastal Development Institute of Technology: Technical Manual for L-block Quay Walls, 2006 (in Japanese)

15) Kohama, E., Miura, K., Yoshida, N., Ohtsuka, N. and Kurita, S. : Instability of Gravity Type Quay Wall Induced by Liquefaction of Back.ll during Earthquake, Soils and Foundations, Vol.38, No.4, pp.71 -84, 1998.

16) Tsuchida, T., Y. Kikuchi, T. Fukuhara, T. Wako and K. Yamamura: Slice Method for Earth Pressure Analysis and its Application to Light-weight Fill, Technical Note of the Port and Harbor Research Institute, No.924, 1999 (in Japanese)

17) Kitajima, S., H. Sakamoto, S. Kishi, T. Nakano and S. Kakizaki: On Some Problems being Concerned with Preparation for the Design Standards on Port and Harbour Structures, Technical Note of the Port and Harbour Research Institute, No.30, pp.32-43, 1967 (in Japanese)

18) Kawamata, H., M. Takenobu and M. Miyata: A Basic Study of Level 1 Reliability-based Design Method of Circular Slip Failure Verification by Modified Fellenius' Method, Technical Note of National Institute of Land and Infrastructure Management, No.955, p.85, 2017 (in Japanese)

19) Takenobu, M. S. Nishioka, T. Sato and M. Miyata: A Basic Study of the Level 1 Reliability Design Method based on Load and Resistance Factor Approach- Performance verification of sliding failure and overturning failure for caisson type quay walls in permanent situation, Technical Note of National Institute of Land and Infrastructure Management, No.880, 2015 (in Japanese)

20) Matsunaga. Y., K. Oikawa and T. Wako: Deformation of Foundation Ground of Gravity-type Port Structures due to the Great Hanshin-Awaji Earthquake, Proceedings of the Conference on the Great Hanshin-Awaji Earthquake, pp.383-390, 1996 (in Japanese)

21) Nakahara, T., Kohama, E. and Sugano, T.: Model shake table test on the seismic performance of gravity type quay wall with different foundation ground properties, 13WCEE, 2004.

22) Iai, S., Y. Matsunaga and T. Kameoka: Strain Space Plasticity Model for Cyclic Mobility, Report of the Port and Harbour Research Institute, Vol.29 No.4, pp.27-56, 1990 (in Japanese)

23) Lysmar, J., Udaka,T., Tsai,C.F. and Seed, H.B.:FLUSH-A Computer program for earthquake response analysis of horizontally layered site, Report No.EERC72 -12, College of Engineering, University of California, Berkeley, 1972.

24) Susumu IAI, Koji ICHII, Hanglong LIU and Toshikazu MORITA: Effective stress analyses of port structures, Special Issue of Soils and Foundations, Japanese Geotechnical Society, pp.97-114, 1998.

25) ITASCA: FLAC - Fast Lagrangian Analysis of Contina, User.s Manual, Itasca Consulting Group, Inc., Minneapolis, Minnesota, 1995 .

26) Cundall, P.A.: A computer model for simulating progressive, large scale movement in blocky rock system, Symp. ISRM, Nancy, France, Proc., Vol.2, pp.129-136, 1971.

27) Kanatani, M., Kawai, T. and Tochigi, H.: Prediction method on deformation behavior of caisson-type seawalls covered with armored embankment on man-made islands during earthquakes, Soils and Foundations, Japanese Geotechnical Society, Vol.41 -6, 2001.

28) Iai, S.: Similitude for Shaking Table Tests on Soil-structure-fluid Model in 1 g Gravitational Field, Report of the Port and Harbour Research Institute, Vol.27 No.3, pp.3-24, 1988 (in Japanese)

29) Sugano, T.: Japan-US Symposium on the Earthquake Resistance of Port and Urban Functions- Full Scale Experiment at Tokachi Port, Earthquake Disaster Prevention, No.190, pp.3-5, 2003 (in Japanese)

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

31) Suzuki, M.: Port Engineering, Kazamashobo, p.474, 1955 (in Japanese)

32) Ozaki, R. and T. Nagao: Coseismic Behavior Analysis of Sheet Pile Type Quay Walls with the Positions of Anchor Piles as Parameters, Proceedings of the 60th Annual Meeting of JSCE, 2005 (in Japanese)

33) Kubo, K., F. Saigusa and A. Suzuki: Lateral Resistance of Vertical Anchor Piles, Report of the Port and Harbour Research Institute, Vol.4 No.2, 1965 (in Japanese)

34) Ishii, Y. trans.: Tschebotarioff's Soil Engineering (Fist Volume), Gihodo Shuppan, p.308, 1964 (in Japanese)

35) P.W. Rowe: Anchored sheet pile walls, Proc. of I.C.E., Vol.1 Pt.1., 1955.

36) Arai, H., T. Yokoi and T. Furube: On the Earthquake Resistance of Anchored Sheet Pile Walls (Second Report), Proceedings of the Second Conference of the Port and Harbour Research Institute, p.73, 1964 (in Japanese)

37) Arai, H. and T. Yokoi: On the Earthquake Resistance of Anchored Sheet Pile Walls (Third Report), Proceedings of the Third Conference of the Port and Harbour Research Institute, p.100, 1965 (in Japanese)

38) Sawada, G.: Calculation Method of Passive Earth Pressure on the Embedded Sections of Sheet Piles in Sloped Ground, Technical Note of the Port and Harbour Research Institute, No.9, 1964 (in Japanese)

39) Ishii, Y. trans.: Tschebotarioff's Soil Engineering (Second Volume), Gihodo Shuppan, p.192, 1964 (in Japanese)

40) P. W. Rowe: A theoretical and experimental analysis of sheet-pile walls, Proc. of I.C.E., Vol.4 Pt.1., 1955.

41) Ishiguro, K., M. Shiraishi and H. Kaiwa: Steel Sheet Pile Method (First Volume), Sankaido Publishing, p.95, 1982 (in Japanese)

42) Takahashi, K., Y. Kikuchi and K. Ishiguro: Analysis of Flexural Behavior of Anchored Sheet Pile Walls, Journal of Structural Engineering, Vol.42A, p.1195, 1996 (in Japanese)

43) Takahashi, K., Y. Kikuchi and Y. Asaki: Analysis of Flexural Behavior of Anchored Sheet Pile Walls, Technical Note of the Port and Harbour Research Institute, No.756, 1993

44) Takahashi, K. and K. Ishiguro: Vertical Beam Analysis Method for Piles and Sheet Piles Subject to Lateral Loads, Sankaido Publishing, pp.177-183, 1992 (in Japanese)

45) Terzaghi: Evaluation of coef.cients of subgrade Reaction, Geotechnique, Vol.5, pp.297-326, 1955.

46) Tschbotarioff: Large scale earth pressure tests with model .exible bulkheads, Princeton Univ., 1949.

47) Karl Terzaghi, Ralph B.Peck (K. Hoshino, et al. trans.): Soil Mechanics in Engineering Practic, Maruzen, p192, 1970

48) Morikawa, Y., Y. Kikuchi and T. Mizutani: Development of Design Method for Anchored Sheet Pile Wall Reinforced by Additional Anchorage Work, Report of the Port and Airport Research Institute, Vol.50 No.4, 2011 (in Japanese)

49) Mizuno, K. and T. Tsuchida: The Stability Analysis for Sliding Resistance of Sheet Pile, Journal of Structural Engineering, Vol.48A, pp.1441-1452, 2002 (in Japanese)

50) Matsubara, H., M. Takenobu, M. Miyata and Y. Watanabe: A Basic Study on Level 1 Reliability Design Method for Anchored Sheet Pile Quay Wall in a Permanent Design Situation, Technical Note of National Institute of Land and Infrastructure Management, Vol7 No.8, pp.135-167, 1968 (in Japanese)

51) Akatsuka, Y. and K. Asaoka: Experimental Studies on High Strength Tie Rod, Report of the Port and Airport Research Institute, Vol.7 No.8, pp.135-167, 1968 (in Japanese)

52) Mitsuhashi, I.: Inferring the Value of the 2-dimensional k-value, Technical Note of the Port and Harbour Research Institute, No.219, 1975 (in Japanese)

53) Katayama, T., T. Nakano, T. Hasumi and K. Yamaguchi: Examination of Current Design Method in View of Actual Disasters due to the 1968 Tokachi-oki Earthquake, Technical Note of the Port and Harbour Research Institute, No.93, pp.89-98 p.136, 1969 (in Japanese)

54) Ministry of Transport, Ports and Harbours Bureau, First Port Development Bureau, Port and Harbour Research Institute: Report on the Disasters due to the 1964 Niigata Earthquake Part 1, p.101, 1964 (in Japanese)

55) Ports and Harbours Association of Japan: Guidelines for the Construction of Steel Sheet Piles, 1969 (in Japanese)

56) Ohya, Y., Y. Shiozaki, E. Kohama and Y. Kawabata: Proposal of Modeling of Circular Steel Tube for Seismic Performance Evaluation, Report of the Port and Airport Research Institute, Vol.56 No.2, pp.3-33, 2017 (in Japanese)

57) Shiozaki, Y., Y. Ohya and E. Kohama: M-ϕ Characteristics of Steel Pipe Piles Considering Local Buckling, Proceedings of the 37th Conference of Earthquake Engineering, No.A12-1242, 2017 (in Japanese)

58) Tsuiji, K., S. Tagawa and T. Nagao: Seismic Coefficients of Cantilever Sheet Pile Type and Double Sheet Pile Type Quay Walls against the Level-one Earthquake Ground Motion, Technical Note of National Institute for Land and Infrastructure Management, No.454, 2008 (in Japanese)

59) Shiozaki, Y., R. Tanaka, A. Sowa and M. Ohnuki: Design Method of Secondary Stress on Steel Sheet Piles for Cantilever Type Sheet Quay Walls, Proceedings of the 70th Conference of JSCE, VI-202, pp.403-404, 2015 (in Japanese)

60) Ishiguro, K., M. Shiraishi and H. Kaiwa: Steel Sheet Pile Method (First Volume), Sankaido Publishing, p.95, 1982 (in Japanese)

61) Ohshima, M. and M. Sugiyama: Design Method of Sheet Pile Walls with Raked Anchor Piles, Soil Mechanics and Foundation Engineering, Vol.13 No.3, pp.11-18, 1965 (in Japanese)

62) Ishiwata, T., K. Ishiguro and Y. Higuchi: Measurement of Behavior of Sheet Pile Walls with Raked Anchor Piles, Technical Note of Fuji Iron and Steel, Vol.13 No.4, pp.73-87, 1964 (in Japanese)

63) Shiozaki, Y., K. Otsushi and A. Sowa: Seismic Deisgn of Open-type Quay Wall with Sheet Pile Wall Anchored by Forward Piles for Level-1 Earthquake Ground Motion, Journal of JSCE Division A1 (Structural Engineering & Earthquake Engineering), Vol.70 No.4, pp.I_407-418, 2014 (in Japanese)

64) Sugano, T., Y. Shiozaki and M. Ikegami: Shake Table Tests on the Seismic Behavior of Sheet Pile Quaywall with Batter Piles in Front and Container Crane with Isolation System, Soil Mechanics and Foundation Engineering, Vol.51 No.3, pp.22-24, 2003 (in Japanese)

65) Sawaguchi, M.: Lateral Behavior of a Double Sheet Pile Wall Structure, Soils and Foundations, Vol.14 No.1, pp.45-59, 1974.

66) Ohori, K., Y. Shoji, K. Takahashi, H. Ueda, M. Hara, Y. Kawai and K. Shioda: Static Behavior of Double Sheet Pile Structure, Report of the Port and Harbour Research Institute, Vol.23 No.1, pp.103-151, 1984 (in Japanese)

67) Technical Committee of Shore Protection Facilities: Technical Standards and Commentary for shore Protection Facilities, 2004 (in Japanese)

68) Japan Road Association: Guidelines for Design and Construction of Temporary Structures for Road Earth Work, pp.76-87, 1999 (in Japanese)

69) G. P. Tschebotarioff, F. R. Ward: Measurements with Wicgmann Inclinometer on Five Sheet Pile Bulkheads, 4th Intern. Conf. Soil Mech. and Foundation Eng., Vol.2, 1957.

70) Edited by G. A. Leonards: Foundation Engineering, Mc Graw Hill Book Co., pp.514, 1962.

71) Takahashi, K., S. Noda, K. Kanda, S. Miura, T. Mizutani and S. Terasaki: Horizontal Loading Tests on Models of Steel Sheet Pile Cellular Bulkhead-Part 1 Static Behavior, Technical Note of the Port and Harbour Research Institute, No.638, 1989 (in Japanese)

72) Noda, S., K. Takahashi, K. Kanda, S. Terasaki, S. Miura and T. Mizutani: Horizontal Loading Tests on Models of Steel Sheet Pile Cellular Bulkhead- Part 2 Dynamic Behavior, Technical Note of the Port and Harbour Research Institute, No.639, 1989 (in Japanese)

73) Kitajima, S., S. Noda and T. Nakayama: An Experimental Study on the Static Stability of Steel Plate Cellular Bulkhead with Embedment, Technical Note of the Port and Harbour Research Institute, VNo.375, 1981 (in Japanese)

74) Noda, S., S. Kitazawa, T. Iida, N. Mori and H. Tabuchi: An Experimental Study on the Earthquake Resistance of Steel Plate Cellular Bulkheads with Embedment, Report of the Port and Harbour Research Institute, Vol.21 No.2, 1982 (in Japanese)

75) Shibata, D. and T. Nagao: Seismic Coefficients of Embedded-type Cellular Bulkhead Quay Walls against the Level-one Earthquake Ground Motion, Technical Note of National Institute of Land and Infrastructure Management, No.562, 2010 (in Japanese)

76) Yokoyama, Y.: Design and Construction of Steel Piles, Sankaido Publishing, pp.95-96, 1963 (in Japanese)

77) Yoshida, I. and R. Yoshinaka: Engineering Properties of Akashi and Kobe Layers, Report of Civil Engineering Research Institute, Vol.129, 1966 (in Japanese)

78) Yoshida, I. and Y. Adachi: Experimental Study on Static Horizontal Resistance of Caisson Foundation, Report of Civil Engineering Research Institute, Vol.139, pp.24-25, 1970 (in Japanese)

79) Nagao, T. and T. Kitamura: Optimal Dimension Decision Method of Cellular-bulkhead Quay Wall, Journals of JSCE Division B3 (Ocean Engineering), Vol.20, pp.203-208, 2004 (in Japanese)

80) Sumiya, K and T. Nagao: A Study on the Effect of the Embedment of Cellular Bulkheads on the Seismic Stability, Technical Note of National Institute of Land and Infrastructure Management, No.352, 2006 (in Japanese)

81) Noda, S., S. Kitazawa, T. Iida, N. Mori and H. Tabuchi: An Experimental Study on the Earthquake Resistance of Steel Plate Cellular Bulkheads with Embedment, Report of the Port and Harbour Research Institute, Vol.21 No.2, p.147 1982 (in Japanese)

82) Sugano, T., T. Kitamura, T. Morita and Y. Yui: Study on the Behavior of Steel Plate Cellular Bulkheads during Earthquake, Proceedings of the 10th Symposium of Japan Association for Earthquake Engineering, pp.1867-1872, 1998 (in Japanese)

83) Sato, S., M. Takenobu, E. Kohama and O. Kiyomiya: A Study on the Behavior of Cellular-bulkhead Quay Wall during Earthquake, Journal of JSCE Division A1 (Structural Engineering & Earthquake Engineering), pp.190-209, 2014 (in Japanese)

84) Saimura, Y., A. Morimoto and Y. Takase: Field Measurement Results the Earth Pressure of Fillings in Embedded Steel Plate Cellular Bulkheads, Proceedings of the 36th Conference of JSCE, Part 3, pp.562-563, 1981 (in Japanese)

85) Ito, Y, O. Iimura, M. Goto, T. Shiroe and T. Iida: On the Construction of Embedded Steel Plate Cellular Bulkheads, Sumitomo Metal, Vol.34 No. 2, pp.93-105, 1982 (in Japanese)

86) Ministry of Transport, the Third Port Development Bureau, Kawasaki Steel: Report on Steel Plate Cellular Bulkhead Tests, 1966 (in Japanese)

87) Tokikawa, K.: Experimental Study on Reflection Rates of Vertical Wave Dissipation Quay Walls (First Report), Proceedings of Coastal Eng. JSCE Vol. 21, pp.409-415, 1974 (in Japanese)

88) Tanimoto, K., Y. Haranaka, S. Takahashi, K. Komatsu, M. Todoroki and M. Ohsato: An Experimental Investigation of Wave Reflection, Overtopping and Wave Forces for Several Types of Breakwaters and Sea Walls, Technical Note of the Port and Harbour Research Institute, No.246, p.38, 1976 (in Japanese)

89) Goda, Y. and Y. Kishira: Experiments on Irregular Wave Overtopping Characteristics of Low Crest Types, Technical Note of the Port and Harbour Research Institute, No.242, p.28, 1976 (in Japanese)

3 Mooring Buoys

[Ministerial Ordinance] (Performance Requirements for Mooring Buoys)

Article 27

1 The performance requirements for mooring buoys shall be as prescribed respectively in the following items:

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

(2) Damage, etc. due to the actions of variable waves, water flows, traction by ships, etc. shall not impair the function of the mooring buoys, and shall not adversely affect the continuous use of the mooring buoys.

2 In addition to the provisions of the preceding paragraph, the performance requirements for mooring buoys in the place where there is a risk of having serious impact on human lives, property, or socioeconomic activity by damage to the mooring buoys shall be such that the structural stability of the mooring buoys is not seriously affected even in cases where the function of the mooring buoys is impaired by design tsunamis, accidental waves, etc.

[Public Notice] (Performance Criteria of Mooring Buoys)

Article 53

1 The performance criteria for mooring buoys shall be as prescribed respectively in the following items:

(1) The buoy shall have the necessary freeboard in consideration of the usage conditions.

(2) The mooring buoy shall have the dimensions necessary for the containment of the swinging area of moored ships within the allowable dimensions.

(3) The following criteria shall be satisfied under the variable situation, in which the dominating actions are variable waves, water flows, and traction by ships.

(a) The risk of impairing the integrity of the anchoring chains of floating bodies, ground chains, and sinker chains shall be equal to or less than the threshold level.

(b) The risk of losing the stability of the buoy due to tractive forces acting on mooring anchors, etc. shall be equal to or less than the threshold level.

2 In addition to the provisions of the preceding paragraph, the performance criteria of the mooring buoys for which there is a risk of serious impact on human lives, property, or socioeconomic activity by damage to the facilities shall be such that the degree of damage under the accidental situation in which the dominating actions are design tsunamis or accidental waves is equal to or less than the threshold level.

[Interpretation]

11. Mooring facilities

(7) Performance criteria of mooring buoys (Article 27 of the Ministerial Ordinance and the interpretation related to Article 53 of the Public Notice)

① The performance requirement for mooring buoys shall be serviceability. The serviceability mentioned here shall mean that the mooring buoy concerned has the necessary freeboard in consideration of the usage conditions as well as the dimensions required for containment of the swinging area of moored ships within the allowable range.

② In setting the freeboard, the expected usage conditions of the facilities concerned shall be appropriately taken into account. Further, in setting the dimensions of mooring buoys, the structure and sectional dimensions of the facilities shall be set so that the containment of the swinging area of floating bodies is appropriately considered in compliance with their expected usage conditions.

③ In addition to the above, the performance requirement for mooring buoys under the variable situation in which the dominating actions are variable waves, water flows and/or tractive forces by ships shall be serviceability. The performance verification items and the standard indexes for the determination of the limit values to such actions shall be as shown in **Attached Table 11-18**.

Attached Table 11-18 Performance Verification Items and Standard Indexes for Determination of Limit Values of Mooring Buoys in Each Design State (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
27	1	2	53	1	3a	Serviceability	Variable	Variable waves [water flow] [traction by ships]	Self-weight, water pressure, water flow	Yield of anchoring chains of floating body, ground chains, or sinker chains	Design yield stress
					3b					Stability of mooring anchors, etc.	Resistance force of mooring anchors, etc. (horizontal, vertical)

* Items in [　　] denote replacing the dominating action according to the design state.
* The stability verification of mooring anchors, etc. refers to verifying that the tensile force acting on such facilities does not exceed their resistance force.

④ The term "mooring anchors, etc." shown in **Attached Table 11-18** is used as a general term for equipment placed on the seabed to retain floating bodies and includes sinkers and the like in addition to mooring anchors.

⑤ The performance requirement for mooring buoys under the accidental situation in which the dominating actions are design tsunamis and/or accidental waves shall be safety. Further, the performance verification item and the standard index for the determination of the limit value to the actions shall be as shown in **Attached Table 11-19**. In addition, to proceed with the performance verification of mooring buoys by referring to **Attached Table 11-19**, the standard indexes for the determination of the limit values shall be appropriately designated based on the structure types.

Attached Table 11-19 Performance Verification Item and Standard Index for Determination of Limit Value of Mooring Buoys in Facilities Prepared for Accidental Incidents under Accidental Situations

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non- dominating action		
27	2	–	53	2	–	Safety	Accidental	Design tsunami [Accidental waves]	Self-weight, water pressure, water flow	Stability of mooring system	–

* Items in [　　] denote replacing the dominating action according to the design state.

3.1 Fundamentals of Performance Verification

(1) The mooring buoy shall secure appropriate stability under the mooring method, the natural conditions at the site, the principal dimensions of the design ships etc.. The mooring buoy here shall be the facilities for mooring ships other than cargo ships and the like that do not involve cargo handling work when moored. In addition, for mooring buoys at which moored ships involve handling work of hazardous cargo such as oil pipelining, **Reference (Part III), Chapter 2, 5.8 Design of Floating Mooring Facilities**, etc. shall be referred to.

(2) Mooring buoys are structurally categorized into three types: sinker type, anchor chain type, and anchored sinker type. The sinker type mooring buoy comprises a floating body, an anchoring chain of a floating body, and a sinker; a mooring anchor is not used, as shown in **Fig. 3.1.1(a)**. The anchor chain type mooring buoy comprises a floating body, an anchor chain, and a mooring anchor; it does not have a sinker, as shown in **Fig. 3.1.1(b)**. Although the construction cost of this type is lower than that of the other types, it is not generally suitable for cases where the area of the mooring basin is limited, because the radius of a moored ship's swinging motion becomes large. The anchored sinker type mooring buoy comprises a floating body, an anchoring chain of a floating body, a ground chain, a sinker chain, a mooring anchor, and a sinker as shown in **Fig. 3.1.1(c)**. Mooring buoys of this type are being used widely in ports and harbors. This type of buoy can be used even when the area of the mooring basin is limited because the radius of a moored ship's swinging motion can be reduced by increasing the weight of the sinker.

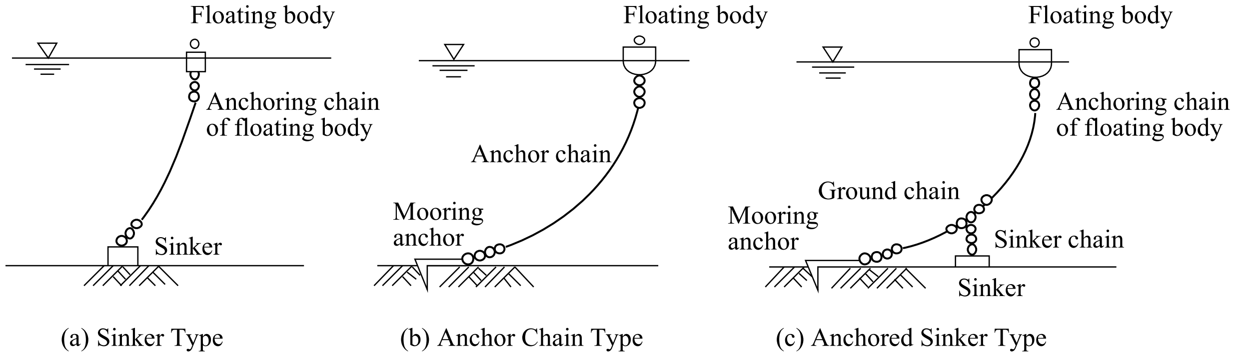


Fig. 3.1.1 Types of Mooring Buoys

(3) The procedure for performance verification of mooring buoys is shown in **Fig. 3.1.2** as an example. Here, the mooring system shall comprise every part of a mooring buoy, and for example, that of the anchored sinker type mooring buoy shall mean each part of a floating body, an anchoring chain of a floating body, a ground chain, a sinker chain, a mooring anchor and a sinker.

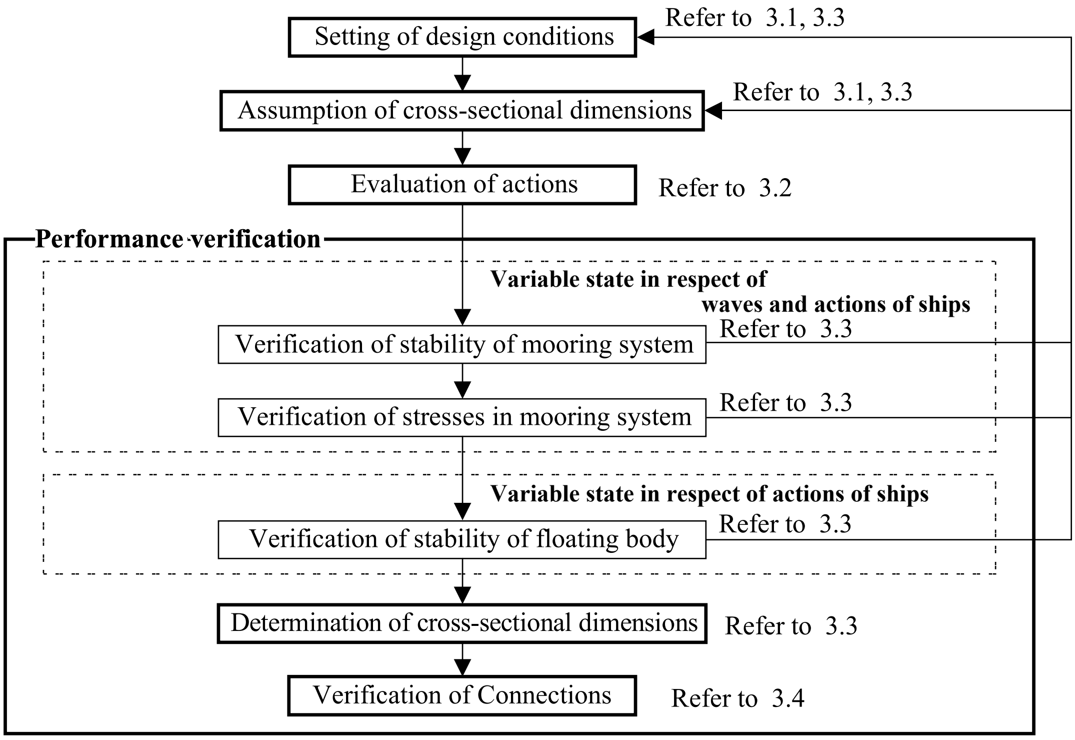
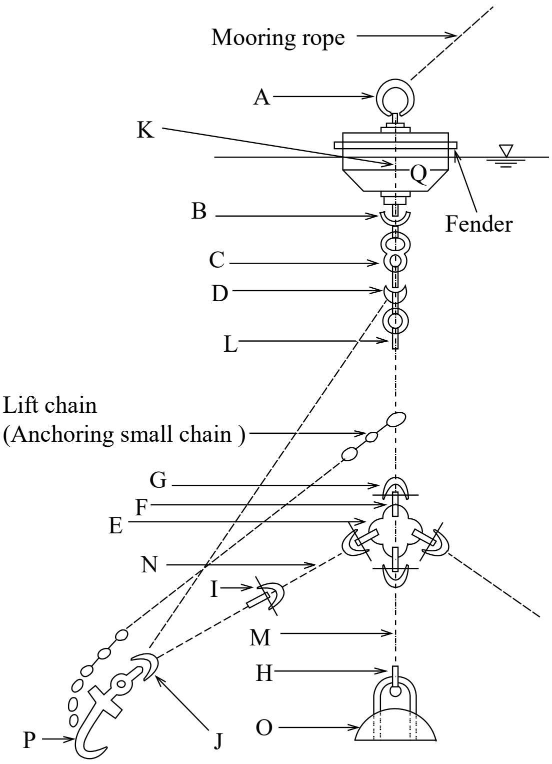


Fig. 3.1.2 Example of Performance Verification Procedure for Mooring Buoys

(4) **Fig. 3.1.3** shows an example of the structure of members of a mooring buoy.



A: Harp shackle (mooring ring) or quick release hook	1
B: Anchor shackle	1
C: Swivel piece	1
D: Joining shackle	1
E: Mooring piece	1
F: Long ring	2
G: Joining shackle	2
H: Anchor shackle	1
I: Joining shackle	2
J: Anchor shackle	2
K: Chain	1
L: Main chain (Anchoring chain of floating body)	1
M: Sinker chain	1
N: Chain or ground chain	4
O: Sinker or pan	1
P: Anchor or screw anchor	1
Q: Buoy	1

Fig. 3.1.3 Example of Structure of Members of Mooring Buoy

(5) The performance verification method for mooring buoys shown in this section shall apply to those categorized as the anchored sinker type from among the types mentioned above. Moreover, since the sinker type and the anchor chain type can be regarded as a simplified anchored sinker type, the same performance verification provisions can be applied to their performance verifications as well.

(6) The tensile force or other force acting on the structural members of a mooring buoy, i.e., an anchoring chain of a floating body and a ground chain, is to be determined in accordance with the shape and/or the weight of each structural member, so that changing even the shape of one structural member can result in a change in all the values. Therefore, to examine the performance verification of a mooring buoy in an economical way, it is necessary to decide the most optimal structure in such a way that the shape of each member is assumed beforehand, the tensile force or other force at this stage is calculated, and then the shape of each member is modified one after another in trial calculations.

(7) The mooring of ships with the mooring buoy can be categorized into single buoy mooring and dual buoy mooring. In the case of the single buoy mooring system, the tractive force acting on the mooring buoy is small, but the area of the mooring basin required is large. In contrast, since the dual buoy mooring system applies two or more mooring buoys to moor a ship, the area of one ship's mooring basin can be made small as it is almost stabilized in the bow to stern direction; however, a large tractive force will act on the buoys. Further, in the case of the dual buoy mooring system, it is desirable that the mooring buoys be arranged so as to be in parallel with the direction of the wind and/or water flow to reduce the tractive force because the ship's swinging motion is small.

(8) The performance verification of the stability of mooring anchors, etc. for the case in which the dominating actions are tsunamis and/or accidental waves must be examined with the drifting of a mooring buoy or a moored ship taken into account, which may be caused by such a tsunami or accidental waves, in order to ensure that it will not make a serious impact on the surroundings.

3.2 Actions

(1) In principle, the tractive force acting on a mooring buoy shall be set considering structural characteristics of the mooring buoy in accordance with the provisions in **Part II, Chapter 8, 2.4 Actions Caused by Traction of Ships**. When setting the tractive force, consideration should be given to the effects of wind, water flows, and waves. However, it should be noted that these are dynamic actions, and thus there are many uncertainties in their relationship with the tractive forces. Therefore, it is preferable that the tractive force acting on a mooring buoy be determined considering the actions that are exerted upon moored ships, such as wind, water flows, and waves, and by referring to the existing tractive force data on buoys of a similar type.

(2) When the motions of a buoy due to wave action are not negligible in terms of the stability of a mooring buoy, it is desirable that the effects of such motions be considered in the calculation of the wave force and/or the resistance force.

(3) When a dynamic analysis of a mooring buoy is performed, the response characteristics vary widely depending on how the waves are applied; in an analysis to which regular waves are applied, the motions of the mooring buoy would be generally either overestimated or underestimated. Therefore, random waves with spectral characteristics shall be employed in the analysis.

(4) In the case of single buoy mooring, the moored ship will enter into a swinging motion. The results of past hydraulic model tests show that the swing angle, the intersection angle between the wind direction and the ship's longitudinal axis, is about 30° at maximum, varying widely with the anchor chain length, the wind velocity, etc.[1] **Reference 2)** can be used as a reference for single buoy mooring.

(5) When the wave height is large under the variable situation related to the waves, a tensile force would act, making an impact upon a mooring buoy. To reduce the impact of the tensile force, it is desirable that an elastic chain be used for part of the mooring system.[3]

(6) **Table 3.2.1** shows examples of design conditions and corresponding tractive forces on mooring buoys.

Table 3.2.1 Examples of Design Conditions for Mooring Buoys

Design ship DWT (ton)	Mooring method	Wind velocity (m/s)	Tidal current (m/s)	Wave height (m)	Tractive force (kN)
1,000	Single buoy	50	0.5	2.0	185
3,000	Ditto	50	0.5	4.0	409
15,000	Ditto	15	0.51	0.7	245
20,000	Ditto	20	1.0	–	589
130,000	Ditto	60	0.67	10.0	1,370
260,000	Ditto	25	0.51	3.0	1,840
30,000	Dual buoy	15	–	–	1,490
100,000	6-points	20	–	1.5	1,470

3.3 Performance Verification of Each Part of a Mooring Buoy

(1) General

For the sizes and materials strength, etc. of each part of a mooring buoy, including the mooring anchor, sinker, sinker chain, ground chain, anchoring chain of the floating body, and the floating body itself, **Part III, Chapter 5, 6 Floating Piers** can be referred to; further, these factors shall be set appropriately in accordance with the tractive force of ships, the structure of the mooring buoy, the mooring method, and the like.

(2) Mooring Anchor

① Normally, three mooring anchors are attached to a mooring buoy. In the performance verification of a mooring buoy, however, it shall be assumed that only one of the three anchors is set to resist the horizontal force. The arrangement of the mooring anchors must be designed in such a way that the buoy would not capsize even if one of the anchor chains were to be broken.

② The horizontal force acting on a mooring buoy shall be resisted only by the mooring anchors. For the anchor holding power of mooring anchors, **Part III, Chapter 5, 6 Floating Piers** shall be referred to. There are cases in which the anchor holding power of a portion of the ground chains that is in contact with the seabed is taken into account; it is however desirable to consider in the performance verification that only the mooring anchors will resist the horizontal force because the interaction mechanism of the mooring anchor with the ground chain has some unclear points and the resistance force of such chains is quite small compared to that of the mooring anchor. Single fluke stock anchors are often used for mooring anchors in port facilities. Further, buried anchors[4)] are also used in place of single fluke stock anchors.

③ The anchor holding power of mooring anchors varies widely depending on the ground conditions of the seabed, the topography, the shape of the mooring anchors, etc., so that these conditions should be properly taken into consideration.

(3) Sinker and Sinker Chain

① Normally the length of a sinker chain ranges from 3 to 4 m. It is preferable not to use an excessively long sinker chain because it allows a large range for the upward movement of the sinker and increases the risk of tangling of the chain and thus the risk of abrasion and accidental breaking of the chain. Generally, the sinker chain should be of the same diameter as that of the anchoring chain of the floating body.

② The vertical and horizontal forces acting on the sinker can be calculated generally based on the tension of the anchoring chain of the floating body and the distance of the horizontal movement of the floating body, using the following **equation (3.3.1)**. [5)]

$$\left.\begin{aligned} P_V &= T_A \sin\theta_1 = (T_C - wl)\sin\theta_1 \\ P_H &= T_A \cos\theta_1 = (T_C - wl)\cos\theta_1 \end{aligned}\right\} \qquad \textbf{(3.3.1)}$$

where

P_V, P_H : vertical and horizontal forces acting on the sinker, etc. respectively (kN)

θ_1 : angle that the anchoring chain of the floating body makes with the horizontal plane at the sinker attachment point (º)

T_A : tension of the anchoring chain of the floating body at the sinker attachment point (kN)

T_C : tension of the anchoring chain of the floating body at the floating body attachment point (kN)

w : weight of the anchoring chain of the floating body per unit length in water (kN/m)

l : length of the anchoring chain of the floating body (m)

Further, θ_1 can be obtained by solving the following equations.

$$\left.\begin{aligned} l &= \frac{T_A \cos\theta_1}{w}(\tan\theta_2 - \tan\theta_1) \\ \Delta K &= \frac{T_A \cos\theta_1}{w}\left\{\sinh^{-1}(\tan\theta_2) - \sinh^{-1}(\tan\theta_1)\right\} \end{aligned}\right\} \qquad \textbf{(3.3.2)}$$

where

ΔK : distance of horizontal movement of the floating body (m)

θ_2 : angle that the anchoring chain of the floating body makes with the horizontal plane at the floating body attachment point (º)

In variable situations in respect of action by ships, the anchoring chain of the floating body usually becomes approximately straight when the tractive force acts on it, so that the following approximation can be used:

$$\theta_2 \approx \theta_1 = \cos^{-1}\frac{\Delta K}{l} \qquad \textbf{(3.3.3)}$$

③ The weight of the sinker most commonly used for 5,000 GT ships and 10,000 GT ships is about 50 kN and 80 kN respectively, so that it can be determined using these values as references. The values mentioned above indicate the weight in water. Sinkers may be of any shape and material as long as they satisfy the weight requirement, but in Japan disk-shaped cast iron sinkers are used commonly while concrete is seldom used.

When the bottom surface of the sinker is made slightly concave, if the seabed is soft, a considerable adhesion effect with the ground is expected.

④ The role of the sinker is to absorb the impact force acting on the chain and to make the anchoring chain of the floating body shorter. When the anchoring chain of the floating body is to be shortened to reduce the distance of movement of the ship, the weight of the sinker must be increased accordingly.

⑤ In certain cases, buried anchors may be used instead of sinkers.

(4) Ground Chain

① The angle that the ground chain makes with the seabed at the mooring anchor attachment point is desirably less than 3º because the holding power of the mooring anchor decreases sharply as the angle increases to 3º or more.[5)] In many cases, the weight of the ground chain is determined in such a way that the ground chain satisfies the above mentioned condition when the tractive force acts on the mooring buoy. When the tractive force is large, the attachment angle between the mooring anchor and the ground chain may be made smaller by lengthening the ground chain. The inclination angle θ_1 of the ground chain at the mooring anchor attachment point can be generally calculated by **equation (6.4.8)** described in **Part III, Chapter 5, 6.4. Performance Verification.** The equation is redefined and expressed as **equation (3.3.4)**. In addition, **Fig.3.3.1** shows a situation in which a ship is moored at a mooring buoy of the anchored sinker type, and indicates the notation of the lengths, angles, and so forth to be used in equations for the calculation of tensions on anchor chains and the like.

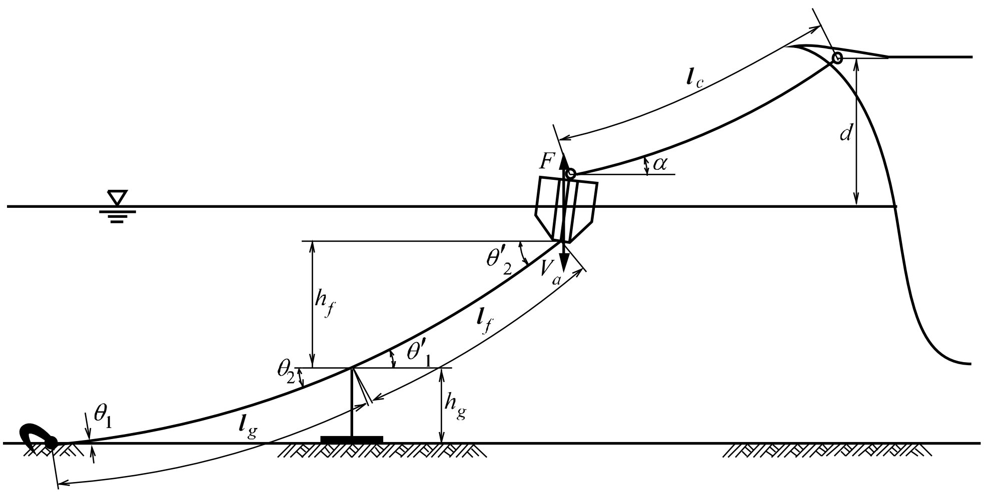


Fig. 3.3.1 Performance Verification of Anchored Sinker Type Mooring Buoy

$$\left.\begin{aligned} l_g &= \frac{P_H}{w}\left(\tan\theta_2 - \tan\theta_1\right) \\ h_g &= \frac{P_H}{w}\left(\sec\theta_2 - \sec\theta_1\right) \end{aligned}\right\} \qquad \textbf{(3.3.4)}$$

where

l_g : length of the ground chain (m)

h_g : vertical distance between the upper end of the ground chain and the seabed (i.e. the sum of the length of the sinker chain, the height of the sinker, and the allowance) (m)

P_H : horizontal component of the tractive force acting on the floating body (kN)

w : weight of the ground chain per unit length in water (kN/m)

θ_1 : inclination angle of the ground chain at the attachment point to the mooring anchor (º)

θ_2 : inclination angle of the ground chain at the upper end of the chain (º)

It should be noted that the calculation should be made by assuming l_g, w, and h_g such as to obtain θ_1 that becomes less than 3°.

② The maximum tension T_g of the ground chain can be calculated using **equation (6.4.5)** described in **Part III, Chapter 5, 6.4 Performance Verification**. The equation is redefined and expressed as **equation (3.3.5)**.

$$T_g = P_H \sec\theta_2 \tag{3.3.5}$$

where

P_H : horizontal component of the tractive force acting on the floating body (kN)

θ_2 : inclination angle of the ground chain at the upper end of the chain (º)

③ The tensile yield strength of the chain shall be set based on **Part III, Chapter 5, 6 Floating Piers**. In the case of mooring buoys, however, the diameter of the chain is usually determined not only on the basis of strength, but on the basis of the theory that the use of a heavier chain helps absorb the energy of impact forces, or, as is known from **equation (3.3.4)**, the use of a shorter chain reduces the radius of the ship's swinging motion; in general, the chain diameter is designed so that it is equal to that of a chain on which a maximum tension equivalent to around 1/5 to 1/8 of the breaking test load can act.

(5) Anchoring Chain of Floating Body

① The length l_f of the anchoring chain of the floating body shall be determined in such a way as to reduce the tension acting on both the anchoring chain of the floating body and the mooring rope as well as to lessen the radius of the ship's swinging motion. It should be noted that the relation between the ratio of the anchoring chain length to the water depth and the degree of abrasion of the anchoring chain of the floating body has not been clarified yet.

② It is desirable that the tension acting on the anchoring chain of a floating body and the displacement of the floating body be obtained by means of a numerical simulation of motions, but the results obtained under similar conditions in the past as well as the method shown in the following ③ to ⑤ can be applied.

③ The weight of the anchoring chain of a floating body per unit length in water w_f (kN/m) can be calculated generally by replacing w with w_f in **equation (3.3.4)**. Here, l_g and h_g in the equation should be replaced by the length of the anchoring chain of floating body l_f (m) and the vertical distance between the upper and the lower ends of the anchoring chain of floating body h_f (m), respectively. The vertical distance between the upper and the lower ends of the anchoring chain of floating body h_f denotes the vertical distance between the attachment point to the floating body and the upper end of the sinker chain when the sinker is lifted up to the point where its bottom is completely separated from the seabed surface. The force P_H (kN) represents the horizontal component of the tractive force acting on the mooring buoy, and θ_2 and θ_1 should be replaced by the inclination angles of the anchoring chain of the floating body at the upper and lower ends, θ_2' (º) and θ_1' (º), respectively. Further, the inclination angle θ_1' can be calculated as shown in **Fig. 3.3.2** from the conditions of balance among the lower end tension of the anchoring chain of floating body T_{fv}, the upper end tension of the ground chain T_g, and the upper end tension of the sinker chain T_{sv}, where T_{sv} is exactly the summation of the weights of the sinker and sinker chain in water, and T_g and its direction can be calculated using **equation (3.3.5)**.

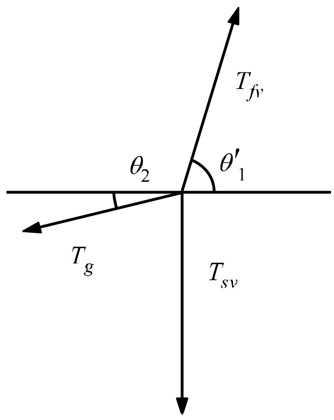


Fig. 3.3.2 Schematic Drawing for Tension of Ground Chain

④ The tension of the anchoring chain of a floating body at the upper end can be calculated using **equation (3.3.5)**. Here, the horizontal component of the tractive force can be adopted as the horizontal force. The angle θ_2' that the anchoring chain of the floating body makes with the horizontal plane at the floating body attachment point can be calculated by **equation (3.3.4)** using the previously obtained weight of the anchoring chain of the floating body per unit length in water. In general, this tension is used for the performance verification of the stress on the anchoring chain of the floating body.

⑤ The horizontal displacement ΔK of the floating body can be generally calculated by means of **equation (6.4.9)** described in **Part III, Chapter 5, 6.4 Performance Verification.** The equation is redefined and expressed as **equation (3.3.6)**.

$$\Delta K = \frac{P_H}{w}\left\{\sinh^{-1}(\tan\theta_2') - \sinh^{-1}(\tan\theta_1')\right\} \tag{3.3.6}$$

The appropriateness of the resultant value of the displacement of the floating body obtained from this equation should be examined in comparison with the area of the mooring basin; if the value is found to be excessively large, it is necessary to either shorten the anchoring chain of the floating body and increase the weight of the sinker or increase the unit length weight of the anchoring chain of the floating body.

(6) Floating Body

In variable situations in respect to the action of moored ships, the floating body of a mooring buoy shall be designed in such a way that it does not become submerged. Even when no ship is moored, the floating body must be kept afloat with a freeboard equal to around 1/2 to 1/3 of its height maintained with the anchoring chain of the floating body, and also, if applicable, part of the sinker chain and the ground chain suspended from it. The buoyancy necessary for the floating body shall be determined to meet these two requirements. A floating body buoyancy satisfying the former requirement can be generally calculated by **equation (3.3.7)**

$$F = V_a - \frac{P}{\sqrt{\left(\frac{l_c}{d}\right)^2 - 1}} \tag{3.3.7}$$

where

F : required buoyancy of the floating body (kN)

V_a : vertical force acting on the floating body (kN)

P : tractive force (kN)

l_c : length of the mooring rope (m)

d : vertical distance between the ship's hose pipe and the water surface (m)

Here, the vertical force V_a acting on the floating body can be obtained by **equation (6.4.6)** shown in **Part III, Chapter 5, 6.4 Performance Verification**. However, it should be noted that the total buoyancy actually required is the sum of the buoyancy needed to resist the tractive force and the self-weight of the floating body.

3.4 Performance Verification of Structural Members

(1) For floating bodies, there is a spinning top type, discus type, pear type, barrel type, sphere type, cone type, and so forth. The most commonly used types are the spinning top type and the discus type. Each part of a floating body shall have the strength to resist design water pressures under conditions in which the floating body is totally submerged and capsized in any direction. Partition walls may be built inside the floating body. There are also cases in which a movable lever is installed inside the floating body to keep its top surface constantly horizontal, as shown in **Fig. 3.4.1**. Wooden or rubber fenders need to be attached to the floating body to protect it against damage due to ship impact.

(2) The metal fittings to be used, including shackles, swivels, links, and mooring pieces, must have a strength corresponding to that of the chain.

(3) Various types of shackles, swivels, etc. used at the connection points of the floating body to the anchoring chain of a floating body and the sinker chain to the anchoring chain of the floating body are subject to extensive abrasion due to the swinging motion of the floating body, and this therefore should be noted for the implementation of performance verification and maintenance.

(4) It is desirable that anchoring chains of a floating body, ground chains, and sinker chains be inspected once or twice every year, and be renewed if abrasion and/or corrosion corresponding to 10% or more of the original diameter is found. Under average usage, they are generally renewed every 4 years.

(5) The standard sizes of harp shackles are as shown in **Table 3.4.1**. Simple metal fittings for line handling may sometimes be used with harp shackles.

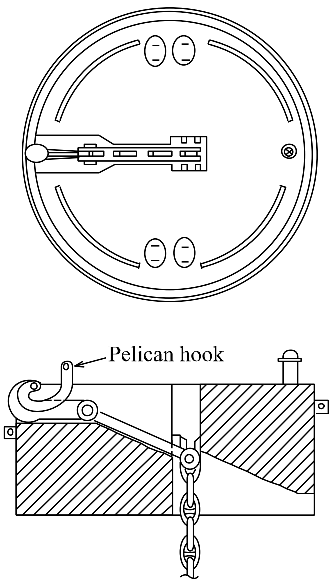


Fig. 3.4.1 Floating Body for Keeping Top Surface Horizontal

Table 3.4.1 Standard Sizes of Harp Shackles

Moored Ship GT (ton)	Inside Diameter (mm)	Thickness of Ring
500	200	80
1,000	240	80
2,000	280	100
3,000	320	100
4,000	360	110
5,000	400	110
6,000	440	110
8,000	480	120
10,000	520	130
15,000	520	130
20,000	520	130
25,000	560	140
30,000	600	150

[References]

1) Yoneda, K.: Wind tunnel experiment on drifting motion of buoy moored ship, Proceedings of 28th Conference of Japan Institute of Navigation, (Mooring buoy -process for standardization- reference), 1962. (in Japanese)

2) Suzuki, Y.: Study on the Design of Single Point Buoy Mooring, Technical Note of PHRI, No.829, 1996. (in Japanese)

3) Hiraishi, T. and Y. Tomita: Model Test on Countermeasure to Impulsive Tension of Mooring Buoy, Technical Note of PHRI, No.816, p.18, 1995. (in Japanese)

4) JSCE: Guideline and Commentary for Design of Offshore Structures (Draft), 1973. (in Japanese)

5) U.S. Navy Bureau of Yards and Docks: Mooring Guide, Vol.1, p.61, 1954.

4 Mooring Piles

[Ministerial Ordinance] (Performance Requirements for Mooring Piles)

Article 28

The performance requirements for mooring piles shall be as prescribed respectively in the following items:

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

(2) Damage, etc. due to the actions of berthing, traction by ships, etc. shall not impair the function of the mooring piles, and shall not adversely affect the continuous use of the mooring piles

[Public Notice] (Performance criteria of Mooring Piles)

Article 54

The performance criteria of mooring piles shall be as prescribed respectively in the following items:

(1) The mooring piles shall have the dimensions required for the usage conditions.

(2) The following criteria shall be satisfied under the variable situation, in which the dominating actions are ship berthing and traction by ships:

(a) For mooring piles with superstructures, the risk of impairing the integrity of the members of the superstructure shall be equal to or less than the threshold level.

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

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

[Interpretation]

11. Mooring Facilities

(8) Performance criteria of mooring piles (Article 28 of the Ministerial Ordinance and the interpretation related to Article 54 of the Public Notice)

The performance requirement for mooring piles under the variable situation in which the dominating actions are ship berthing and/or traction by ships shall be serviceability. In addition, the performance verification items and the standard indexes for the determination of the limit values to such actions shall be as shown in **Attached Table 11-20**.

Attached Table 11-20 Performance Verification Items and Standard Indexes for Determination of Limit Values for Mooring Piles in Each Design State (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
28	–	2	54	–	2a	Serviceability	Variable	Ship berthing and/or traction by ships	Self-weight	Sectional failure of superstructure [*1)]	Design resistance force of section
					2b					Axial force on pile	Ratio of bearing-capacity-related action on pile to resistance force (pushing, pulling)
					2c					Yielding of pile	Design yield stress

[*1)]: Only for structures with a superstructure.

(1) In setting the sectional dimensions and ancillary facilities in regard to the performance verification of mooring piles, consideration shall be properly given to their expected usage conditions.

(2) In the implementation of the performance verification of mooring piles, **Part III, Chapter 5, 5.2 Open-type Wharves on Vertical Piles** and **5.3 Open-type Wharves on Coupled Raking Piles of this Chapter** shall be referred to in accordance with the characteristics of the facilities.

5 Piled Piers

[Ministerial Ordinance] (Performance Requirements for Piled Piers)

Article 29

1 The performance requirements for piled piers shall be as prescribed respectively in the following items in consideration of the structural type:

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

(2) Damage to the piled pier due to self-weight, earth pressure, Level 1 earthquake ground motions, berthing and traction by ships, surcharge load, etc. shall not impair the functions of the piers and shall not adversely affect its continuous use.

2 In addition to the provisions of the previous paragraph, the performance requirements for piled piers listed in the following items shall be as prescribed respectively in those items:

(1) "Performance requirements for piled piers for the purpose of environmental conservation" means that piled piers shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to contribute to conservation of the environment of ports and harbors without impairing the original functions of the piled piers.

(2) "Performance requirements for piled piers classified as high earthquake-resistance facilities" means that damage to piled piers, etc. due to Level 2 earthquake ground motions, etc. shall not affect the restoration through minor repair works of functions required for the quatwalls in the aftermath of the occurrence of Level 2 earthquake ground motion. Provided, however, that for the performance requirements for the piled piers which requires further improvements in earthquake-resistant performance due to environmental conditions, social conditions, etc. to which the piled piers are subjected, damage due to Level 2 earthquake ground motions, etc. shall not impair the functions necessary for the quatwalls in the aftermath of the occurrence of Level 2 earthquake ground motion, and shall not adversely affect the continuous uses of the piled piers.

[Public Notice] (Performance Criteria of Piled Piers)

Article 55

1 The provisions of Article 48 apply mutatis mutandis to the performance criteria of piled piers.

2 In addition to the provisions of the preceding paragraph, the performance criteria of the access bridge of piled piers shall be as prescribed respectively in the following items:

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

(a) The access bridge of piled piers shall have the dimensions necessary for enabling the safe and smooth loading, unloading, embarkation and disembarkation, etc. in consideration of the usage conditions.

(b) The access bridge of piled piers shall not transmit horizontal loads to the superstructure of the piled pier, and shall not fall down even when the piled pier and the earth-retaining part are displaced owing to the actions of earthquakes, etc.

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

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

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

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

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

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

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

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

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

3) The provisions of Article 49 through Article 52 apply mutatis mutandis to the performance criteria of the earth-retaining parts of piled piers in consideration of the structural type.

[Interpretation]

11. Mooring Facility

(9) Performance Criteria of Piled Piers

① **Piled piers which are classified as high earthquake-resistance facilities** (Article 29 paragraph 2 item 2 of the Ministerial Ordinance and the interpretation related to Article 55 paragraph 1 of the Public Notice)

a) In regard to the interpretation concerning performance requirements and performance criteria of piled piers that are high earthquake-resistance facilities, the interpretation concerning performance requirements and performance criteria of quay walls that are high earthquake-resistance facilities is applied, excluding performance verification items and standard indexes to provide limit values.

b) Verification items and standard indexes to provide limit values of piled piers that are high earthquake-resistance facilities to accidental situations with a dominating action of Level 2 earthquake ground motion shall be in accordance with **Attached Table 11-21**.

Attached Table 11-21 Performance Verification Items and Standard Indexes to Provide Limit Values of Piled Piers That Are High Earthquake-resistance Facilities

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to provide limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non- dominating action		
29	2	2	55	1	–	Restorability and Serviceability	Accidental	Level 2 earthquake ground motion	Self weight, surcharges	Deformation of face line	Residual deformation
										Cross-sectional failure of the superstructure	Design cross-sectional resistance
										Damage to piles	Limit curvature
										Axial forces in the piles	Bearing power of piles

c) In **Attached Table 11-21**, the standard index to provide the limit value for the deformation of the face line shall apply to gravity-type mooring quay walls that are high earthquake-resistance facilities.

d) In **Attached Table11-21**, the following performance verification shall be carried out concerning damage to piles of piled piers that are high earthquake-resistance facilities in consideration of the types of high earthquake-resistance facilities.

i) Specifically designated (emergency supply transport) and specifically designated (trunk line cargo transport)

It shall be verified that no pile which reaches the limit curvature at two locations exists in the cross section of the piled pier concerned.

ii) Standard (emergency supply transport)

It shall be verified that at least one pile which reaches the limit curvature at less than two locations on a pile exists among the piles comprising the piled pier concerned. (It shall be verified that all the piles existing in the cross section of the piled pier concerned are not in a state such that the limit curvature at two or more locations is reached on a pile.)

e) The verification items and standard indexes to provide limit values of the high earthquake-resistance facilities of open-type wharves on vertical piles shall be applied for piled piers that are high earthquake-resistance facilities of structures with stiffened members.

② **Main structure of piled piers** (Interpretation related to Paragraph 1, Article 29 of the Ministerial Ordinance and Paragraph 2, Article 55 of the Public Notice)

a) The performance requirement for piled piers under a variable situation where the dominating actions are Level 1 earthquake ground motions, berthing and traction by ships, surcharges, and variable waves shall be serviceability. Performance verification items and the standard indexes to provide limit values to these actions concerning the superstructure and the piles of piled piers are shown in **Attached Tables 11-22** and **11-23**.

Attached Table 11-22 Performance Verification Items and Standard Indexes to Provide Limit Values in Each Design Situation (Excluding Accidental Situations) Concerning Superstructure of Piled Piers

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to provide limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non- dominating action		
29	1	2	55	2	2a	Serviceability	Variable	Berthing and traction by ships	Self weight, surcharges	Cross-sectional failure of superstructure	Design cross-sectional resistance
								Level 1 earthquake ground motion	Self weight, surcharges		
								Surcharges (including surcharges during cargo handling)	Self weight, wind acting on cargo handling equipment and ships		
								Surcharges (including surcharges during cargo handling)	Self weight, wind acting on cargo handling equipment and ships	Crack width of superstructure cross-section	Limit value of bending crack width
								Repeatedly applied surcharges	Self weight	Fatigue failure of superstructure	Design fatigue strength
					3b			Variable waves	Self weight	Cross-sectional failure of superstructure	Design cross-sectional resistance

Attached Table 11-23 Performance Verification Items and Standard Indexes to Provide Limit Values in Each Design Situation (Excluding Accidental Situations) Concerning Piles of Piled Piers

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to provide limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non- dominating action		
29	1	2	55	2	2b	Serviceability	Variable	Berthing, traction by ships	Self weight, surcharges	Axial forces in piles	Action-resistance ratio concerning bearing capacity of piles
								Level 1 earthquake ground motion	Self weight, surcharges		
								Surcharges (including surcharges during cargo handling)	Self weight, wind acting on cargo handling equipment and ships		
					2c			Berthing and traction by ships	Self weight, surcharges	Yielding of piles	Design yield stress of piles
								Level 1 earthquake ground motion	Self weight, surcharges		
								Surcharges (including surcharges during cargo handling)	Self weight, wind acting on cargo handling equipment and ships		
					3c			Variable waves	Self weight	Axial forces acting in piles	Action-resistance ratio concerning bearing capacity of piles

b) The performance verification item and standard index to provide the limit value concerning access bridges of piled piers under the variable situation in which the dominant action is variable waves is shown in **Attached Table 11-24**. In addition to that shown in **Attached Table 11-24**, performance verification items and standard indexes to provide limit values concerning access bridges of piled piers shall be adequately established as necessary under the variable situation in which the dominant action is surcharges.

Attached Table 11-24 Performance Verification Item and Standard Index to Provide Limit Value in Each Design Situation (Excluding Accidental Situations) Concerning Access Bridges of Piled Piers

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to provide limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non- dominating action		
29	1	2	55	2	3a	Serviceability	Variable	Variable waves	Self weight	Uplift force on access bridge	Design cross-sectional resistance

c) Performance verification items and the standard index to provide a limit value concerning piled piers of structures with stiffening members under the variable situation in which the dominating actions are Level 1 earthquake ground motion, berthing and traction by ships, surcharges, and variable waves shall comply with those of piled piers, and are shown in **Attached Table 11-25**.

Attached Table 11-25 Performance Verification Items and Standard Indexes to Provide Limit Values in Each Design Situation (Excluding Accidental Situations) Concerning Piled Piers of Structures with Stiffening Members

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to provide limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
29	1	2	55	2	4	Serviceability	Variable	Berthing and traction by ships	Self weight, surcharges	Yielding of stiffening members	Design yield stress Design shear force resistance
								[Level 1 earthquake ground motion]	(Self weight, surcharges)	Failure of connections at joints	Design shear force resistance
								[Surcharges (including surcharges during cargo handling)]	(Self weight, surcharges, and wind acting on ships)	Punching shear failure at joints	Design shear force resistance
								Repeatedly acting surcharges	Self weight	Fatigue failure of joints	Design fatigue strength
								Variable waves	Self weight	Failure of connections at joints	Design shear force resistance

* Items within square brackets [ ] in the column "Dominating action" indicate that the design situation replaces the dominating actions.
* Items within parentheses ( ) in the column "Non-dominating action" indicate that this shall be read according to dominating actions.

③ **Earth-retaining sections of piled piers** (Interpretation related to Paragraph 1, Article 29 of the Ministerial Ordinance and Paragraph 3, Article 55 of the Public Notice)

The performance criteria and interpretation concerning earth-retaining sections of piled piers shall, in consideration of the structural types, comply with the criteria and their interpretation in Article 49 "Performance Criteria of Gravity-type Quay Walls" through Article 52 "Performance Criteria of Cell Type Quay Walls" of the Public Notice.

④ **Symbiosis piled pier** (Interpretation related to Item 1, Paragraph 2, Article 29 of the Ministerial Ordinance and Paragraph 1, Article 55 of the Public Notice)

a) A piled pier for environmental conservation is called a "symbiosis piled pier". The following are applied together with the criteria for piled piers:

b) The performance requirement for symbiosis piled piers shall be serviceability. Here, serviceability indicates the performance required to contribute to the preservation of the port environment, such as wildlife and the ecosystem, without impairing the original functions of the piled pier concerned.

c) The dimensions of piled piers for environmental conservation include the structure, cross-sectional dimensions, and ancillary facilities. In establishing the structure and cross-sectional dimensions and installing ancillary facilities in the performance verification of piled piers for environmental conservation, contributing to the preservation of the port environment, including wildlife and the ecosystem, without impairing the original functions of the piled piers concerned shall be considered adequately.

5.1.1 Dimensions of piled piers

(1) Performance verification items held in common to multiple piled piers may be in accordance with **[Facilities] 2.1 Common Items for Quay walls** in this Chapter.

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

(3) Ancillary facilities

In the performance verification of piled piers, it is necessary to appropriately consider ancillary facilities for the piled pier to be safely and efficiently used.

(4) Access bridges

In setting the structure and cross-sectional dimensions of access bridges in the performance verification of piled piers, it is necessary to appropriately consider the conditions of use of the concerned piers in order for the piled pier to be safely and efficiently used.

Also, in setting the structure and cross-sectional dimensions of access bridges in the performance verification of piled piers, it is necessary to appropriately consider the amount of relative deformation between the main structures of the piled pier.

5.1.2 Symbiosis piled piers

(1) Symbiosis piled piers[1)] are piled piers which contribute to creating a hospitable environment in ports and which aim for wildlife inhabitation at beaches, etc. according to the natural circumstances where the facilities concerned are located (**[Reference (Common)] Chapter 3. 2. Symbiosis Port Facilitie**s). It is also possible to add a wildlife inhabitation function when an existing piled pier is improved in order to convert it to a symbiosis piled pier.

(2) The effect on the target of wildlife inhabitation (**[Reference (Common)] Chapter 3. 2. Symbiosis Port Facilities**) shall be grasped with environmental surveys and numerical models, etc. In performance verification, it shall be confirmed that the structure, the cross-section, and the ancillary facilities are suitable to achieve the goal.

(3) The performance requirement for symbiosis piled piers shall be possessing inhabitation functions for wildlife. The impact (dominating actions) shall be ensuring the environment necessary for the inhabitation of wildlife, the existence of the foundation for wildlife, and the external forces such as waves and flows. The environment necessary for wildlife inhabitation includes, for example, water depth and clarity, which affect the amount of light necessary for photosynthesis, and water temperature, which affects the activities of life. More precisely, if the goal is the inhabitation of sessile organisms, it is necessary that the structure and the cross-section of the piled pier and the foundation and the slope of the ancillary facilities be suitable for the target sessile organisms to attach to.

(4) The performance verification of symbiosis piled piers shall be carried out by confirming on the basis of existing knowledge that the environment of the place where wildlife shall live symbiotically is within the range in which the target wildlife can live. For example, in the performance verification for a piled pier which shall serve for the inhabitation of seaweed beds, the light amount, which affects photosynthesis and respiration, and water temperature shall be considered and the performance verification shall confirm that the environment is within the range in which the target seaweed beds can thrive. In cases in which it is possible to estimate the change in the environmental conditions after the symbiosis piled pier is installed or to estimate future environmental changes, etc., verification to confirm that the environment is suitable for wildlife using numerical models of growth is also possible.

(5) In the performance verification of symbiosis piled piers, **[Facilities] Chapter 4. 4. Symbiosis breakwater, [Reference (Common)] Chapter 3. 2. Symbiosis port facilities and Guidelines[1)] for maintenance of symbiosis port facilities** can be referred to.

5.2 Open-type Wharves on Vertical Piles

5.2.1 General

(1) An example of a cross-section of an open-type wharf on vertical piles is shown in **Fig. 5.2.1.**

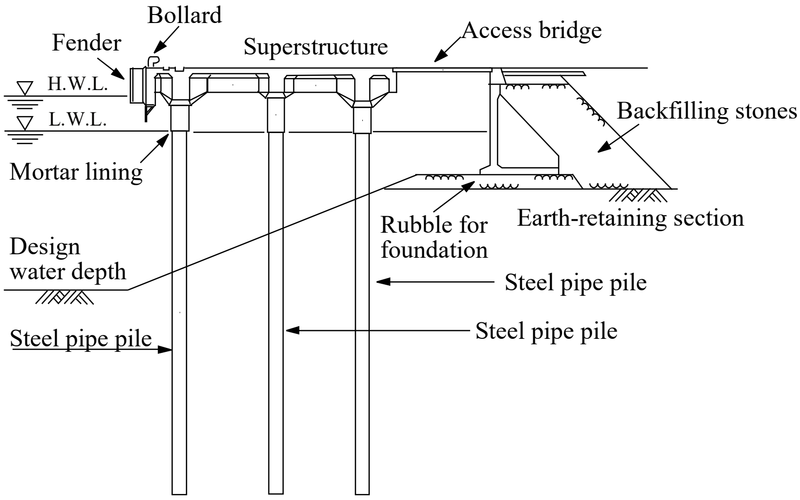
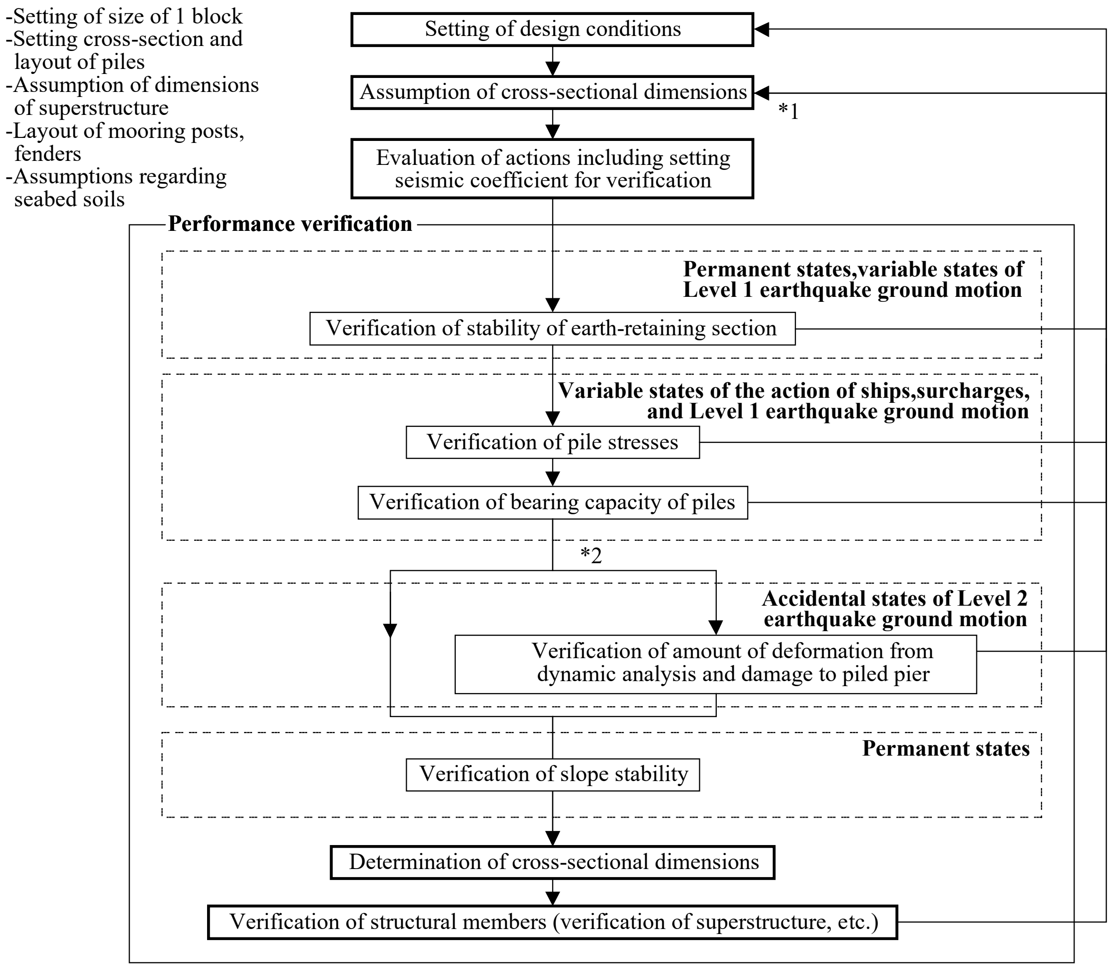


Fig. 5.2.1 Example of a Cross-section of an Open-Type Wharf on Vertical Piles

(2) The following refers to open-type wharves on vertical piles using either steel pipe piles or steel sections; however, it may also be applied to similar facilities when their dynamic characteristics are taken into account.

(3) For the performance verification procedure of open-type wharves on vertical piles, it is possible to refer to **Fig. 5.2.2**. However, the evaluation of the effect of liquefaction owing to earthquake ground motion is not shown in **Fig. 5.2.2;** therefore, it is necessary to appropriately investigate the potential for liquefaction and the measures against it (refer to **Part II, Chapter 7 Ground Liquefaction**).



*1: Evaluation of the effect of liquefaction and settlement is not shown in the diagram; therefore, it is necessary these effects separately.
*2: Verification shall be carried out for high earthquake-resistance facilities against Level 2 earthquake ground motion.

Fig. 5.2.2 Example of the Sequence of Performance Verification of a Piled Pier

(4) In principle, the performance verification method presented in this section will be examined based on the condition that the effect of deformation of the earth-retaining section and other such sections will not be transmitted to the frame. Therefore, it is necessary to adapt the structure dimensions and construction by considering this fact. For example, as the earth-retaining section or the reclaimed land sinks, a part or the whole of the piled pier may cave in or lateral flow may occur; therefore, it is necessary to take measures to ensure that the actions caused by this incident will not transmit to the main body of the piled pier. It is also necessary to take various measures to ensure that the actions caused by deformation of the earth-retaining section and other such sections at the time of an earthquake ground motion will not be transmitted to the superstructure of the piled piers through the access bridges and that the significant deformation of the ground around a pile to the sea side will not negatively affect the piles.

(5) For the variable situation in case of Level 1 earthquake ground motion, it is possible to perform verification by obtaining the natural periods of the piled pier based on frame analysis and by further calculating the seismic coefficient for verification based on the obtained natural periods and the acceleration response spectrum. For open-type wharves on vertical piles other than high earthquake-resistant facilities, the verification of the accidental situation in case of Level 2 earthquake ground motion can be omitted.

(6) While verifying the performance of open-type wharves on the vertical piles, the cross-section is normally set with respect to actions other than that of Level 2 earthquake ground motion; further, the seismic performance is verified with respect to Level 2 earthquake ground motion. This is because the performance verification is performed based on the yield stress of the steel pipe piles for the verification of a variable situation with respect to the action of ships and Level 1 earthquake ground motion; however, a verification method that considers the extent of damage to the

piled pier is used for performing the seismic performance verification of seismic-resistant structures with respect to Level 2 earthquake ground motion.

(7) When cargo-handling equipment, such as container cranes, is to be installed on an open-type wharf on vertical piles, it is preferable to install it in such a manner that all its feet are positioned on either the pile-supported section or the earth-retaining section. For example, if one foot of the cargo-handling equipment is positioned on the pile-supported section and another foot is positioned on the earth-retaining section, the equipment will become susceptible to the adverse effects caused by uneven settlement and ground motions owing to the difference between the response characteristics of the two sections. When it is unavoidable to position one foot on the pile-supported section and another foot on the earth-retaining section, sufficient foundation, such as foundation piles, should be provided to prevent uneven settlement owing to the settlement on the earth-retaining section. In this case, in general, the fixed foot of the cargo-handling equipment, such as the portal crane, should not be installed on the pile-supported section. While installing the cargo-handling equipment, such as the container cranes, on the pile-supported section, seismic response analysis should be performed by considering the coupled oscillation of the cargo-handling equipment and the open-type wharf.

(8) In general, the open-type wharves on vertical piles possess structural types that can be adversely affected by the performance of the members because of material deterioration, including the chloride-induced corrosion of the concrete material and the corrosion of the steel material. Therefore, it is necessary to take maintenance through the service life into account sufficiently at the design stage.

① The superstructures of the piled piers are the concrete structures located in a severe chloride-induced corrosion environment, and it is necessary to pay close attention to maintenance through their service life. If the distance between the superstructure of a piled pier and the sea level is less and if it is difficult to secure work space, inspection and diagnosis, and measures are difficult to be conducted during the service life. In general, with respect to the superstructures of the piled piers, the members of ocean-side tend to deteriorate relatively early[2)]. However, depending on the distance between the superstructures of the piled piers and the sea level or on the form or layout of the earth-retaining sections, the superstructure’s members which are located around the earth-retaining sections may easily deteriorate owing to the wave-breaker actions. It is necessary to consider such factors while setting the maintenance level during the designing stage of the superstructures of piled piers. The superstructures of the piled piers shall be designed based on **Part III, Chapter 2, 1.2.4 Examination Concerning the Time Worn of Performance.**

② The steel pipe piles and steel sections, which compose the piled piers, are located in a severe corrosive environment. Therefore, when the steel members are designed for making the piled piers, appropriate corrosion protection must be performed based on **Part III, Chapter 2, 1.3.4 Examination Concerning the Change of Performance Over Time.**

③ With respect to the piled piers located in a severe environment, it is desirable that labor saving and rationalization should be considered for maintenance. Therefore, inspection holes and inspection scaffolding may be arranged, and sensors for monitoring may be installed to ensure that inspection and diagnosis, and measures may be easily conducted during the service life. Please refer to **reference 3)**, which includes case studies that consider maintenance at the design stage.

④ If the backfilling material could be eliminated from the earth-retaining sections, preventive measures will be adopted according to the structural types of earth-retaining sections by referring to **Part III, Chapter 5, 2.2. Gravity-type Quaywalls** and **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls.**

5.2.2 Setting of the Basic Cross-section

(1) The size of a deck block, the distances between the piles, and the number of pile rows shall be appropriately determined by considering the following:

① apron width;
② location of the sheds;
③ seabed (especially, the slope stability);
④ existing revetments;
⑤ matters related to construction work, including the concrete casting capacity; and
⑥ surcharges (especially, the crane specifications).

(2) The larger the size of one deck block, the bigger the rigidity of the structure will be to the fender reaction force, tractive force, and so on. Even though this is preferable, it becomes weaker against uneven settlement on the other hand. In addition, the size is limited because of the concrete casting capacity. The normal length of one deck block of large-scale wharves is approximately 20 to 30 m in Japan.

(3) In general, the distances between the piles and the number of pile rows will be decided based on the shape of the cross-section of piles, economic comparison between various cases, and examination of construction restrictions. In case of a wharf that is planned to be equipped with a rail mounted crane, an unloader, and so on, construction is often restricted because of the gauge and action status of such equipment. In a case in which large quay cranes are to be installed for ships of the 10,000 DWT class, the piles are usually designed to be placed in intervals of 5 m with 3–4 pile rows in the cross-section. If the superstructures are manufactured from cast-in-place reinforced concrete, the distance between the piles shall be ca. 4–6 m because it is restricted by the concrete casting construction. The distance between the piles could be larger in case of pre-stressed concrete.

(4) The dimensions of the superstructure of the open-type wharf will be appropriately determined by considering the following:

① the distances between the piles, number of pile rows, and shapes and dimensions of piles;
② construction problem of shattering forms and scaffold;
③ ground conditions;
④ arrangement of the mooring posts; and
⑤ arrangement, shape, and dimensions of the fenders.

(5) If the concrete pavement is placed on piled piers after the construction of the piled pier floorboards, cracks may occur in the concrete pavement (refer to **Part III, Chapter 5, 9.18 Apron**).

(6) Assumptions Regarding the Seabed Condition

① Determination of the slope gradient

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

(b) It is necessary to examine the stability of the slope with respect to the circular slip failure. When an earth-retaining structure is installed behind the slope, it is preferable that the structure is not constructed in front of the slope surface from the toe of the slope at the slant angle indicated by **equation** (**5.2.1**) (see **Fig. 5.2.3**).

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

where

α : angle between the slope and the horizontal surface (°),

ϕ: : angle of shear resistance of the main material forming the slope (°),

$\varepsilon = \tan^{-1} k_h'$, and

k_h' : apparent horizontal seismic coefficient

When the slope contains a hard mudstone or rock, **equation (5.2.1)** may not be applied. For the seismic coefficient for verification to calculate the apparent horizontal seismic coefficient, the value calculated in the analysis of the earth-retaining section may be used. Refer to **Reference (Part III), Chapter 1, 1 Particularities Concerning the Seismic Coefficient for Verification** for the calculation of the seismic coefficient to verify the presence of the earth-retaining section.

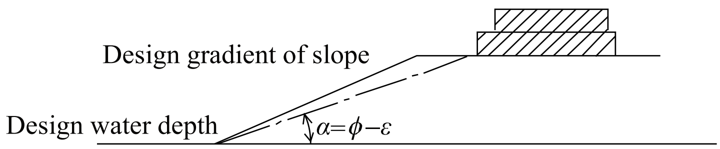


Fig. 5.2.3 Position of the Earth-Retaining Structure on the Slope

(c) The angle set in **equation (5.2.1)** in (b) is a restriction when a structure will be constructed behind the top of this slope. The slant angle as an actual design cross-section is usually steeper than α. For example, if the foundation of the earth-retaining section and the slope face is comprised of rubble, the ratio is usually set to be approximately 1:1.5 to 1:2. This is to take the top of the slope at the front of the earth-retaining structure as much as possible to ensure that the effect of scouring or the effect of local collapse does not reach the front toe of the structure.

② **Virtual ground surface**

(a) In the calculation of the lateral resistance and the bearing capacity of the piles, a virtual ground surface will be assumed at an appropriate elevation shown in the following step for each pile.

(b) When the inclination of the slope is considerably steep, the virtual ground surface for each pile to be used in the calculation of the lateral resistance or the bearing capacity may be set at an elevation that corresponds to half of the vertical distance between the surface of the slope at the pile axis and the seabed as shown in **Fig. 5.2.4**.

(c) The calculation method of lateral resistance of piles used for analysis of open-type wharves on vertical piles is primarily for the horizontal ground surface. Therefore, if the lateral resistance of the piles, which are driven on the slopes as piles of open-type wharves, will be calculated, some kind of correction will be required. To simplify the calculation method, this will generally be set as shown in (b).

(d) The application of this method is not appropriate for wide piers having a width of more than 20 m and having an extremely long slope. In such cases, it is preferable that other methods, such as that denoted in **reference 4)**, will be adopted.

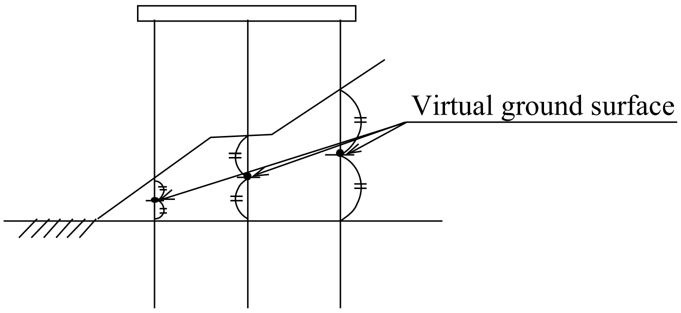


Fig. 5.2.4 Virtual Ground Surface

(7) Coefficient of the Lateral Subgrade Reaction

① In the calculation of the lateral resistance of piles, it is preferable to obtain the coefficient of the lateral subgrade reaction of the subsoil through lateral loading tests of the piles in-situ. If no tests are conducted, the coefficient of the lateral subgrade reaction of the subsoil may be estimated by appropriate analytical methods derived from the lateral resistance tests as shown in ③.

② Some measured data are available about the coefficient of the lateral subgrade reaction that is obtained by the loading tests in which the lateral loads are applied to the piles of open-type wharves up to the yield points. Although some of these data are related to the *N*-value, the coefficient of lateral subgrade reaction cannot be precisely estimated from the *N*-value. Thus, it is preferable to estimate the coefficient by performing the lateral loading tests in-situ.

③ When lateral loading tests of piles are not conducted because of small-scale construction works or time constraints, the coefficient of the lateral subgrade reaction of the subsoil may use the mean value of the minimum value and central value obtained from the lateral resistance tests. While using Chang's method, **equation (5.2.2**) may be utilized, and **Part III, Chapter 2, 3.4.6 Deflection of Piles under the Action of Lateral Load** can be referenced. However, some in-situ measurement data indicate that the coefficient of the lateral subgrade reaction of the rubble stones is smaller than the estimate by **equation (5.2.2)** using Chang's method. In this case, it is recommended to set the coefficient of lateral subgrade reaction as 3,000–4,000 kN/m^3 in Chang's method.

$$k_{CH} = 1{,}500N \tag{5.2.2}$$

where

k_{CH} : coefficient of horizontal subgrade reaction (kN/cm^3) and

N : average N-value of the ground to a depth of approximately $1/\beta$.

④ The coefficient of the lateral subgrade reaction denoted in **equation (5.2.2)** is a static coefficient of the subgrade reaction and may be used while verifying the stress of piles by static frame analysis and other such methods. However, it cannot be used for liquefied ground. In addition, if the natural periods of the piled piers are to be calculated, doubling **equation (5.2.2)** and using the actual ground surface instead of the virtual ground surface will yield values nearer to the actual values [6) 7)]. For the calculation of the natural periods of piled piers, refer to **Part III, Chapter 5, 5.2.3 Actions.**

⑤ Because soil is not an elastic body, the relation between the lateral loads to piles and displacement is generally nonlinear. As the lateral loads become larger, k_{CH} decreases. To derive **equation (5.2.2)** from the result obtained using the lateral resistance tests, it is preferable to provide relatively low k_{CH}.

(8) Virtual Fixed Point

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

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

where

k_{CH} : lateral subgrade reaction coefficient (kN/m^3) (calculated by **equation (5.2.2)**),

D : diameter or width of the pile (m), and

EI : flexural rigidity of the pile (kN·m^2).

② The method of usage of virtual fixed points is used for simple calculation of the pile head moments. If the projected length of the piles from the sea bottom is long, **equation (5.2.3)** can be approximately applied[8)]. The pile head moments and other such factors may be derived using methods other than those shown here.

③ If the virtual fixed-point method based on Chang's method is used, the locations of the virtual fixed points can be decided using the following methods corresponding to Chang's method:

(a) the method in which the first immobile point in Chang's method is set as the virtual fixed point;

(b) the method in which the virtual fixed point is set so that the pile head reaction and the pile head flexural moment using Chang's method are the same as those in beams with fixed ends;

(c) the method in which the virtual fixed point is set so that the pile head displacement and the pile head flexural moment using Chang's method are the same as those in beams with fixed ends; and

(d) the method in which the virtual fixed point is set so that the pile head reaction and the pile head displacement using Chang's method are the same as those in beams with fixed ends.

① is based on the step (b) of the aforementioned method. As the depth of the virtual fixed points calculated by the methods (a) to (d) exhibits a little variation, the pile head displacement also exhibits some difference. However, the depth is within a sufficiently permissible range from an engineering perspective.

④ The virtual fixed-point method based on the PHRI method uses the PHRI method instead of Chang's method to calculate the lateral resistance of each pile. It sets the virtual fixed points so that the pile head displacement calculated by the PHRI method becomes equal to the head displacement of the beam with fixed ends and a virtual fixed point. As the PHRI method considers the nonlinearity of the lateral subgrade reaction of piles, the virtual fixed points stated here are described to depend on the horizontal forces acting on the vertical piles [9) 10) 11) 12)].

(9) It is desirable that the fenders and mooring posts are arranged so that the actions inclining to one deck block do not occur when possible.

(10) If the fender reaction force through fenders or the tractive force transmitted from the mooring posts is added with eccentricity, the reactions by the rotation of deck blocks become larger. Therefore, it is reasonable to install a fender or a mooring post at the center of a deck block so that the inclined action does not occur when possible (refer to **Fig. 5.2.5**).

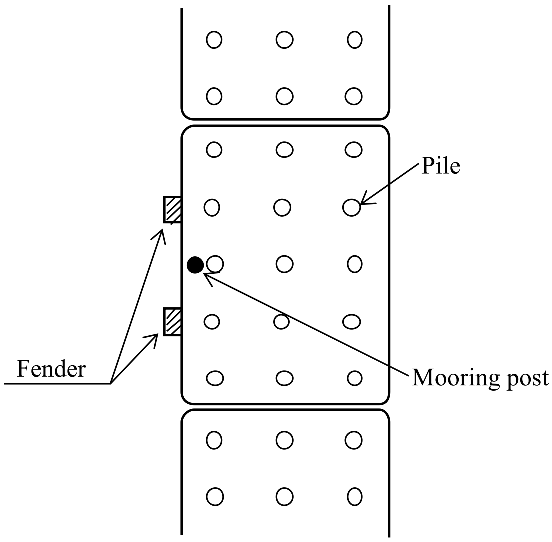


Fig 5.2.5 Arrangement of the Ancillary Facilities

5.2.3 Actions

(1) Types of actions to be taken account of for each design state

The following actions will be considered in the stability verification of open-type wharves on vertical piles with respect to each design state.

① **Permanent situation**

If the earth-retaining sections are to be constructed behind piled piers, the actions, which will be considered in a permanent situation, will be adequately set according to the structure types of the earth-retaining sections.

② **Variable situation**

The following actions shall be considered to be the dominating actions:

(a) Level 1 earthquake ground motion;

(b) ship tractive force and ship berthing force;

(c) surcharge (including surcharge during cargo handling);

(d) surcharge that repeatedly generates actions; and

(e) variable waves.

For understanding Level 1 earthquake ground motion, readers can refer to **Part II, Chapter 6, 1.2 Level 1 Earthquake Ground Motions Used in Performance Verification of Facilities**. With respect to the ship tractive force and ship berthing force, **Part II, Chapter 8, 2 Actions Caused by Ships** can be referred. With respect to the surcharges during cargo handling, **Part II, Chapter 10, 3 Surcharge** can be referred. With respect to the variable waves, **Part II, Chapter 2 Setting of Wave Conditions** can be referred.

③ **Accidental situations**

Level 2 earthquake ground motion will be considered as the dominating action. With regard to the setting of Level 2 earthquake ground motion, **Part II, Chapter 6, 1.3 Level 2 Earthquake Ground Motions used in Performance Verification of Facilities** can be referred.

(2) Points to be noticed regarding setting the actions

① With respect to the actions on the earth-retaining sections, each chapter of this part can be referred to according to their types, and the self-weight of the access bridges is added.

② For calculating the self-weight of the reinforced concrete superstructures, each part of the dimensions is assumed based on the dimensions of the superstructure, and the volume is calculated on them. The self-weight can be obtained by multiplying the unit weight obtained from **Part II, Chapter 10, 2 Self-Weight** by the volume. However, by considering that the superstructure is established not at the design stage of the basic cross-sections but during the performance verification of the structure members, if the performance verification of the piles of open-type wharves on vertical piles is conducted, 21 kN per 1.0 m^2 of the deck area of the superstructure of the piled pier may be assumed for the calculation of the self-weight of the reinforced concrete superstructures. Further, this weight per unit area of the superstructure is calculated from the case examples of large-scale mooring facilities. However, this value can be used for the small-scale mooring facilities as well if the distances between the piles and the superstructure dimensions are not drastically changed.

③ In peculiar piled piers, it is inappropriate to use the self-weight of the superstructures shown above. If the distances between the piles are larger than the normal values, if large cargo-handling equipment, such as container wharves, will be installed on the piled piers, or if piles on the sea side or the land side of the piled piers are steel sheet piles that are also used as earth retainer, it is preferable to decide the self-weight of the superstructure by separately calculating the volume.

④ At the site expected to be subjected to waves, the following items should be examined with respect to the wave uplift on the superstructure of the piled pier and the access bridge:

(a) stability of the access bridges and pulling resistance of the piles against the uplift

(b) member strength of the superstructures and access bridges against uplift.

For uplift, refer to Part II, Chapter 2, 6.4 Wave Action Acting on Structure near the Water Surface.

⑤ The static loads may be determined in accordance with **Part II, Chapter 10, 3.1 Static Load**. The earthquake inertia forces owing to static loads may normally be considered to act on the upper surface of the deck slab. However, when the center of gravity of the static loads is located at an especially high elevation, it is important to consider the height of the center of gravity as the point of application of the horizontal force.

⑥ Live loads should be determined in accordance with **Part II, Chapter 10, 3.2 Live Load**. The seismic force that can be attributed to a rail mounted crane should be calculated by multiplying its self-weight with the seismic coefficient for verification, and the force can be considered to be transmitted from the wheels of the crane to the pile-supported section. It is also necessary to perform seismic response analysis by considering the coupled oscillations of the cargo-handling equipment and the open-type wharf (refer to **Part III, Chapter 7, 2.2.3 Verification of Earthquake-Resistant Performance**). In this case, the ground motion will be applied in the form of a time-series seismic wave profile. Further, the wind load acting on the crane may be determined in accordance with **Part II, Chapter 2, 2.3 Wind Pressure**.

⑦ It is generally preferable to decide the coexistence of static and live loads by considering the utility form of the wharf.

⑧ The fender reaction force used for the performance verification of piled piers can be calculated in accordance with **Part II, Chapter 8, 2.2 Actions Caused by Ship Berthing** and **Part II, Chapter 8, 2.3 Actions Caused by Ship Motions**, and **Part III, Chapter 5, 9.2 Fender System**.

⑨ The tractive force of the vessels can be determined in accordance with **Part II, Chapter 8, 2.4 Actions due to Traction by Ships**. In several cases, one bollard is installed on one deck block.

⑩ When rubber fenders are installed as a damper on an ordinary large wharf with a unit deck block of 20 to 30 m in length, a common practice is to provide two rubber fenders on one block. In several cases, fender intervals of 8 to 13 m are used. The berthing behavior of various sizes of ships has been examined by installing 1.5-m long rubber fenders on an ordinary large wharf. The results of the examination have revealed that it is appropriate to calculate the berthing force based on the assumption that the ship's berthing energy is absorbed by one fender. Therefore, the reaction force may be basically calculated on the assumption that the berthing energy is absorbed by one fender while using rubber fenders as the damper. However, this is not applicable when fenders are continuously installed along the face line of a wharf.

⑪ The berthing energy is absorbed by the displacement of the main structure of the pier. However, it is a common practice to not consider this because the energy absorbed by the main structure of the pier accounts for less than 10% of the total berthing energy in several cases.

⑫ **Fig. 5.2.6** denotes an example of the displacement–energy curve and the displacement–reaction force curve of the rubber fender. If a single fender absorbs the berthing energy E_1, the corresponding fender deformation δ_1 can be obtained. Further, using the other curve, the corresponding reaction force acting on the pier can be obtained as $H_1(\delta_1 \rightarrow C \rightarrow H_1)$. However, if fenders are installed too close to each other and if the berthing energy is absorbed by two fenders, the berthing energy acting on one fender becomes $E_2 = E_1/2$, and the corresponding fender deformation becomes δ_2. As can be obtained from **Fig. 5.2.6** ($\delta_2 \rightarrow D \rightarrow H_2$), the reaction force acting on the pier in the two-fender case is almost the same as that generated in the single fender case because of the characteristics of the rubber fender. Thus, if two fenders are installed close to each other, the horizontal reaction force acting on the pier becomes $2H_2 \approx 2H_1$, which indicates that the horizontal force to be used in the performance verification doubles. Therefore, it is preferable to carefully consider this behavior of the reaction force in the performance verification and determination of the location of fenders while using the fenders that have such characteristics.

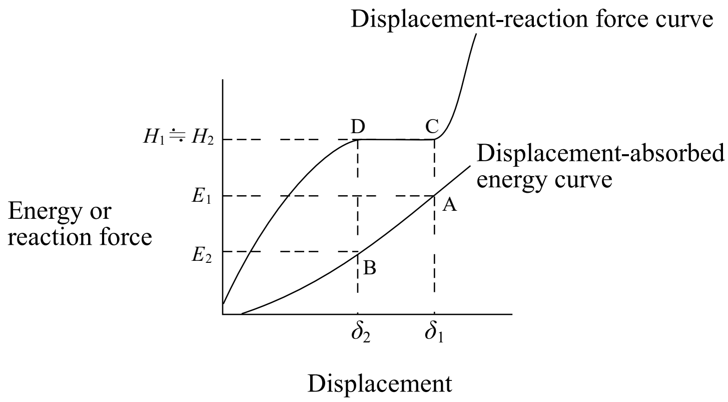


Fig. 5.2.6 Curve of Rubber Fender Characteristics

⑬ When it is considered necessary to examine the rotation of the piled pier unit while evaluating the actions, the verification should take this into consideration. In this case, the distribution of forces on each pile may be evaluated as described below.

(a) When the symmetry axis of the piled pier unit is perpendicular to the face line of the wharf and when the direction of action of the horizontal force is parallel to the symmetry axis as depicted in **Fig. 5.2.7**, the horizontal force may be calculated using **equation** (**5.2.4**). Further, the horizontal force without considering the rotation of the piled pier unit may be calculated by **equation** (**5.2.4**) with e = 0.

$$H_i = \frac{K_{H_i}}{\sum_i K_{H_i}} H + \frac{K_{H_i} x_i}{\sum_i K_{H_i} x_i^2} eH \qquad \textbf{(5.2.4),}$$

where

H_i : horizontal force on pile (kN),

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

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

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

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

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

H : horizontal force acting on the unit (kN),

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

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

The subscript *i* refers to the *i*-th pile.

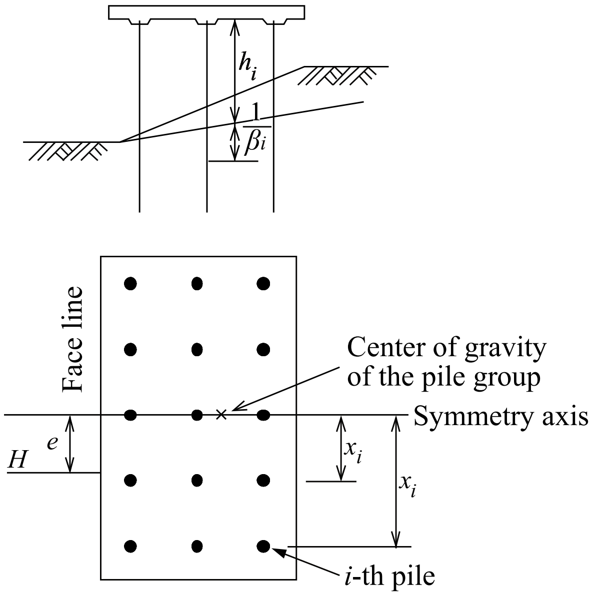


Fig. 5.2.7 Distance between the Center of Gravity of the Pile Group and Individual Piles

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

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

(d) For the methods to obtain the axial force of a piled pier, the pile head moment, and other such factors, the method in Reference 13) can be referred.

⑭ **Ground Motion used in the Performance Verification of Seismic-resistant and Seismic Coefficient for Verification**

(a) The ground motion used in the performance verification of seismic resistance is set by considering the effect of the surface strata using ground seismic response analysis. It is necessary to use a seismic response analysis code capable of appropriately evaluating the amplification of ground motions in soft ground (refer to **Part II, Chapter 6, 1.2.3 Seismic Response Analysis of Surface Ground**).

(b) Using a one-dimensional seismic response analysis as described in **Part II, Chapter 6, 1.2.3 Seismic Response Analysis of Surface Ground**, the acceleration time history at a position $1/\beta$ below the virtual ground surface can be calculated with the acceleration time history of the ground motion set at the seismic bedrock as the input ground motion. While calculating the acceleration time history, the average depth of the $1/\beta$ ground point for each pile may be obtained, as depicted in **Fig. 5.2.8**. From the acceleration response spectrum obtained in this manner, the response accelerations corresponding to the natural periods of the piled pier are calculated, and the value obtained by dividing this with the gravitational acceleration can be regarded as the characteristic value of the seismic coefficient for verification. A damping factor of 0.2 may be used while calculating the acceleration response spectrum.

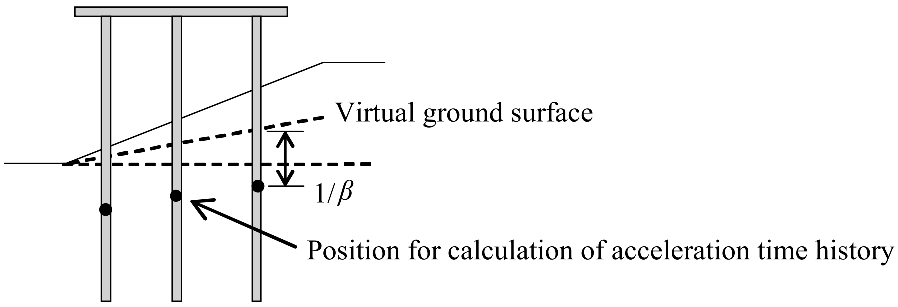


Fig. 5.2.8 Positions for the Calculation of the Earthquake Ground Motions

(c) An example of a typical procedure for setting the seismic coefficient for performing verification is denoted in **Fig. 5.2.9**. While verifying the seismic performance of the earth-retaining parts using the seismic coefficient method, the structural characteristics are observed to be different from those of the piled pier; therefore, the seismic coefficient indicated here may not be used. To calculate the seismic coefficient for verification of the earth-retaining parts, refer to (g).

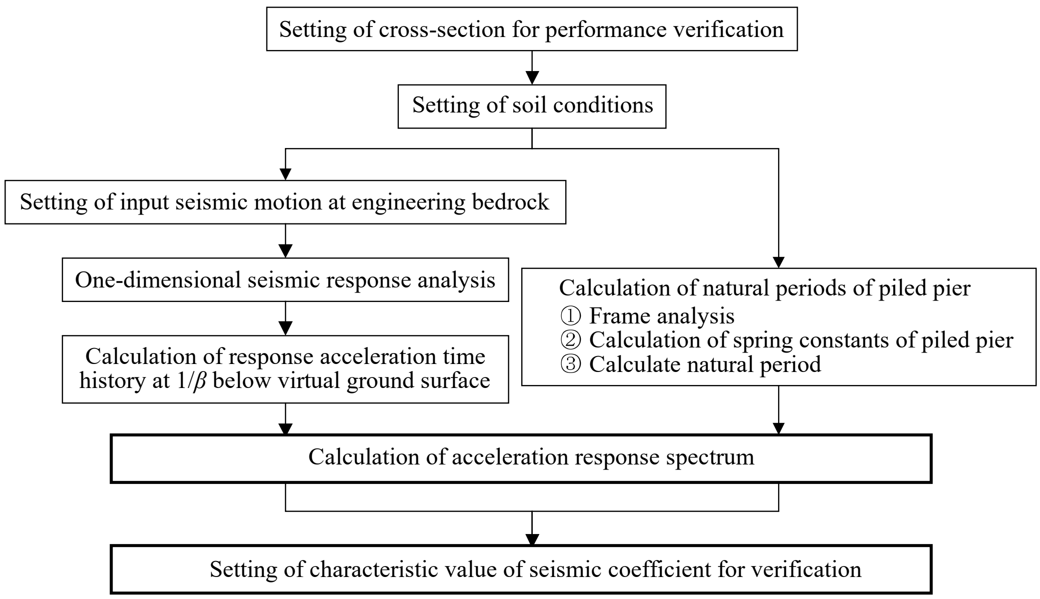


Fig. 5.2.9 Typical Procedure for Setting of the Seismic Coefficient for Verification

(d) The natural periods of the piled pier may be calculated using frame analysis. If the relation between the displacement and the load is obtained from the frame analysis, as depicted in **Fig. 5.2.10**, when minute loads act on the piled pier, the spring constants of the piled pier can be set, and the natural periods can be obtained from **equation (5.2.5)**. While calculating the natural periods, the values that are twice the ground spring constants obtained using **equation (5.2.2)** are often used.

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

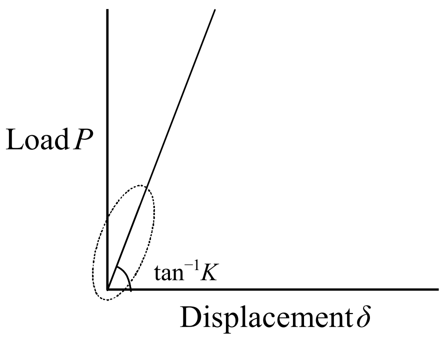
where

T_s : natural period of the piled pier (s),

W : self-weight and static load during an earthquake per one row of pile group (kN),

g : gravitational acceleration (m/s^2), and

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

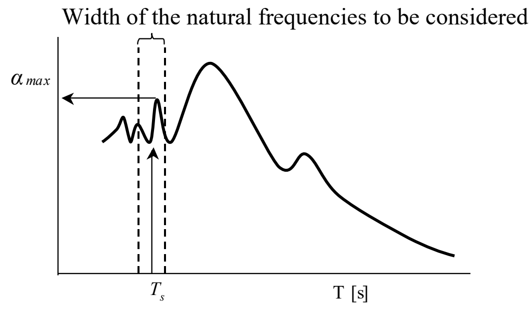


※ Coefficient of lateral subgrade reaction can be obtained from equation (5.2.2)

Fig. 5.2.10 Relation between Load and Displacement from Frame Analysis

(e) If it is judged that the ratio of the sum of the self-weight of the piles, the weight of water in the piles, and the weight corresponds to the mass around the piles added by the earthquake ground motion to the sum of the self-weight of the superstructures of piled piers and surcharges at the time of earthquake is relatively large, it is preferable that all these masses are considered in a flame analysis to estimate the natural period and the cross-section force.

(f) The natural period of the piled pier obtained from the spring constants of the piled pier by frame analysis usually involves some amount of errors. Therefore, if the value in the acceleration response spectrum corresponding to the natural period is a local minimum, the seismic coefficient for verification may be underestimated, and this value should not be applied as it is. In addition, **Reference 14)** indicates the possibility that the natural periods based on **equations** (**5.2.2**) and (**5.2.5**) may be approximately twice as long as the natural periods calculated by the two-dimensional earthquake response analysis etc.. Therefore, it is preferable that the spectral value should be determined to calculate the seismic coefficient for verification with a certain range of natural periods. However, this does not deny the importance of avoiding a local maximum in the acceleration response spectrum caused by the site effects. In case the natural period of the piled pier corresponds to a local maximum in the acceleration response spectrum, it is very likely that the cross-section will not be optimum from the viewpoint of the seismic resistance performance and cost. It is necessary to devote attention to this point for setting the cross-section for verification.



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. 5.2.11 Consideration of the Natural Period in the Acceleration Response Spectrum

(g) Seismic coefficient for verification used in the performance verification of the seismic resistance of the earth-retaining sections

The performance verification of the seismic resistance of the earth-retaining sections can be performed by directly evaluating the deformation of the earth-retaining section using nonlinear effective stress analysis etc.. However, simple methods, such as the seismic coefficient method, can also be used. In this case, it is necessary to appropriately set the seismic coefficient for verification used in the performance verification corresponding to the amount of deformation of the facility by considering the effect of the frequency characteristics of the ground motion and the duration. The normal procedure for calculating the seismic coefficient for verification is as shown in **Reference (Part III), Chapter 1, 1 Particularity Concerning Seismic Coefficient for Verification**.

5.2.4 Performance Verification Regarding Open-Type Wharves on Vertical Piles

(1) Performance verification items to be considered in the performance verification of open-type wharves on vertical piles

① In the performance verification of the open-type wharves on vertical piles, the necessary items will be appropriately investigated and set as necessary. Performance verification under Level 2 earthquake ground motion shall be in accordance with **Part III, Chapter 5, 2.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situation**. For the cross-sectional forces in the superstructure and fatigue failure, refer to **Part III, Chapter 5, 2.6 Performance Verification of Structural Members**.

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

③ In the performance verification for Level 1 earthquake ground motion, the seismic coefficient for verification is calculated from the acceleration response spectrum values corresponding to the natural periods of the piled pier; thus, when the dimensions of the piles are not determined, it is not possible to determine the natural periods of the piled pier. Therefore, the dimensions of the piles are assumed, and the seismic coefficient for verification is calculated from the acceleration response spectrum corresponding to the natural periods; further, the verification is conducted. If the performance requirements are not satisfied, the pile dimensions are changed, and the same calculation has to be repeated.

④ The performance verification of the deformation may be conducted by setting an appropriate limiting value considering the dynamic deformation of the piled pier. For example, the amount of deformation required to ensure that the access bridge does not fall down may be considered to be the limiting value. In this case, it is appropriate to use the response displacement by considering the dynamic action, such as the displacement response spectrum, and not the static action.

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

① The examination of the structural stability of the earth-retaining section of the open-type wharf on vertical piles can be performed in accordance with the performance criteria prescribed in **Part III, Chapter 5, 2.2 Gravity-type Quaywalls,** and **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls** depending on its structural type.

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

③ The stability of the earth-retaining section of the open-type wharf on vertical piles should be examined against the circular slip failure by applying **Part III, Chapter 2, 4.2.1 Stability Analysis by Circular Slip Failure Surface.**

(3) Performance verification for the stresses in the piles under variable situation (surcharge, ship berthing force, tractive force by ship, and Level 1 earthquake ground motion)

① The stresses occurring in the piles of a piled pier may be verified using **equation** (**5.2.6**). In the following equations, γ denotes the partial factor corresponding to the suffix, where the suffixes k and d indicate the characteristic value and the design value, respectively. As for the partial factors in the relevant equations, the values shown in **Table 5.2.1** can be used. The values shown as "-" in **Table 5.2.1** indicates that the values may be verified using the values enclosed in parentheses ( ) to ensure convenience. If the axial forces are tensile, S_k and R_k can be calculated using **equations** (**5.2.6 (b-1)**) and (**5.2.6 (b-2)**), respectively, and each value should satisfy **equation** (**5.2.6**).

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \qquad R_d = \gamma_R R_k \qquad S_d = \gamma_S S_k \tag{5.2.6}$$

(a) When the axial forces are compressive,

$$S_k = \left(\frac{\sigma_{c_k}}{red} + \sigma_{bc_k} \right) \qquad R_k = \sigma_{by_k} \tag{5.2.6 (a)}$$

(b) When the axial forces are tensile,

$$S_k = \sigma_{t_k} + \sigma_{bt_k} \qquad R_k = \sigma_{ty_k} \tag{5.2.6 (b-1)}$$

$$S_k = -\sigma_{t_k} + \sigma_{bc_k} \qquad R_k = \sigma_{by_k} \tag{5.2.6 (b-2)}$$,

where

red : coefficient defined as the value of the axial compressive yield stress (refer to **Table 5.2.2**) divided by the characteristic value of the yield stress,

σ_t and σ_c : tensile stress due to the axial tensile forces acting on the cross-section and compressive stress due to the axial compressive forces, respectively (N/mm^2),

σ_{bt} and σ_{bc} : maximum tensile stress and maximum compressive stress because of the flexural moment acting on the cross-section, respectively (N/mm^2),

σ_{ty} and σ_{cy} : axial tensile yield stress and axial compressive yield stress, respectively (N/mm^2),

σ_{by} : bending compressive yield stress (N/mm^2),

R : resistance term (N/mm^2),

S : load term (N/mm^2),

γ_R : partial factor that is to be multiplied with the resistance term,

γ_S : partial factor that is to be multiplied with the load term, and

m : adjustment factor.

Table 5.2.1 Partial Factor Used for Verification of the Stresses Occurring in the Piles of a Piled Pier

Verification target	Installation water depth	Partial factor to be multiplied with resistance term γ_R	Partial factor to be multiplied with load term γ_S	Adjustment factor m
Stress occurring in the piles of a piled pier (variable action due to surcharge (during work))	All water depth	– (1.00)	– (1.00)	1.67
Stress occurring in the piles of a piled pier (variable action due to surcharge (during storm))	All water depth	– (1.00)	– (1.00)	1.12
Stress occurring in the piles of a piled pier (variable action due to tractive force by ship)	All water depth	– (1.00)	– (1.00)	1.67
Compressive stress occurring in the piles of a piled pier (variable action due to ship berthing force)	Less than 12.0m	0. 97	1.34	- (1.00)
	12.0m and above	1. 01	1.29	
Tensile stress occurring in the piles of a piled pier (variable action due to ship berthing)	All water depth	– (1.00)	– (1.00)	1.67

Verification target	Installation water depth	Partial factor to be multiplied with resistance term γ_R	Partial factor to be multiplied with load term γ_S	Adjustment factor m
Stress occurring in the piles of a piled pier (variable action due to Level 1 earthquake ground motion)	All water depth	– (1.00)	– (1.00)	1.12

② The partial factors are used for the verification of the compressive stresses occurring in the piers of a piled pier at the time when a ship berth in **Table 5.2.1** is the coefficient obtained by the conducted code calibrations so that the obtained dimensions are averagely equivalent as the cross-sections of open-type wharves on vertical piles designed using the previous design methods[12) 16)]. In addition, partial factors related to other design states set by referring to the allowable stresses of the steel members in the previous design methods.

③ Each stress in the equation shown in ① can be calculated using **equation** (**5.2.7**). The suffix k indicates the characteristic value.

$$\sigma_{t_k} = \frac{P_k}{A}, \quad \sigma_{c_k} = \frac{P_k}{A}, \quad \sigma_{bt_k} = \frac{M_k}{Z}, \quad \sigma_{bc_k} = \frac{M_k}{Z} \tag{5.2.7}$$

where

A : cross-sectional area of the piles (mm^2),

P : axial force on the pile (N),

Z : section modulus of the piles (mm^3), and

M : flexural moment of the piles (N·mm).

④ For the yield stress of piles, refer to **Part II, Chapter 11, 2 Steel**. The axial compressive yield stress may be calculated from the equation in **Table 5.2.2**. As for the effective buckling length of the members, the distance from the lower end of the superstructure to $1/\beta$ under the virtual ground surface may be used as denoted in **Fig. 5.2.12**. If verification is necessary for local buckling, the S**pecification and Commentary for Highway Bridges**[23)] may be referred.

Table 5.2.2 The Axial Compressive Yield Stresses

SKK400		SKK490	
a) When $\frac{\ell}{r} \leq 19$	235	a) When $\frac{\ell}{r} \leq 16$	315
b) When $19 < \frac{\ell}{r} \leq 93$	$235 - 1.4\left(\frac{\ell}{r} - 19\right)$	b) When $16 < \frac{\ell}{r} \leq 80$	$315 - 2.1\left(\frac{\ell}{r} - 16\right)$
c) When $\frac{\ell}{r} > 93$	$\dfrac{2.0 \cdot 10^6}{6.7 \cdot 10^3 + \left(\frac{\ell}{r}\right)^2}$	c) When $\frac{\ell}{r} > 80$	$\dfrac{2.0 \cdot 10^6}{5.0 \cdot 10^3 + \left(\frac{\ell}{r}\right)^2}$

SM490Y		SM570	
a) When $\frac{\ell}{r} \leq 15$	355	a) When $\frac{\ell}{r} \leq 13$	450
b) When $15 < \frac{\ell}{r} \leq 76$	$355 - 2.6\left(\frac{\ell}{r} - 15\right)$	b) When $13 < \frac{\ell}{r} \leq 67$	$450 - 3.7\left(\frac{\ell}{r} - 13\right)$
c) When $\frac{\ell}{r} > 76$	$\dfrac{2.0 \cdot 10^6}{4.4 \cdot 10^3 + \left(\frac{\ell}{r}\right)^2}$	c) When $\frac{\ell}{r} > 67$	$\dfrac{2.0 \cdot 10^6}{3.5 \cdot 10^3 + \left(\frac{\ell}{r}\right)^2}$

ℓ: effective buckling length of the member (mm) and r: radius of gyration of the member gross cross-section (mm)

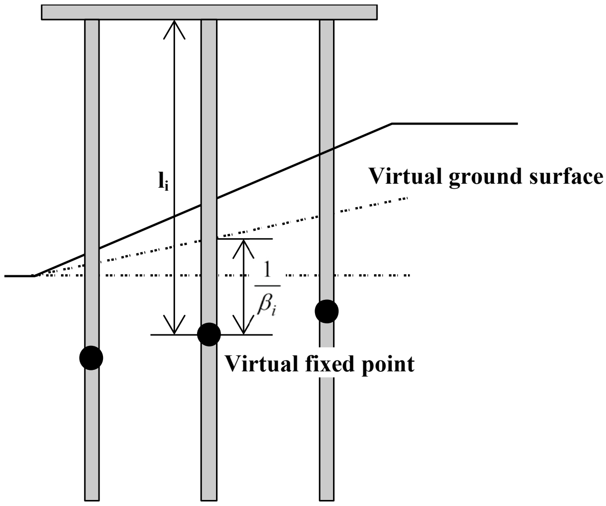


Fig. 5.2.12 Setting of the Effective Buckling Length

⑤ It is preferable to calculate the flexural moments on the piles for the directions both normal and parallel to the face line of the wharf. As in the example shown in **Fig. 5.2.1**, if the ground surface under the floor slab of the piled pier has a sloping surface, the flexural moments in the frontmost row of piles are mostly maximized when the ground motion acts in the direction parallel to the face line.

⑥ The superstructures of the piled piers were equipped with joints for each block interval; however, the horizontal displacements actually transmit each other. Thus, various actions, such as the tractive force and the berthing force, which do not simultaneously occur at individual locations of the whole mooring facility, occur not only in one block but are distributed in a certain section of the piled pier; therefore, it can be assumed that the stress of a pile would not become so dangerous as the stress verified for the normal direction to the face line. However, the earthquake ground movement actions work simultaneously in the whole piled piers and should, therefore, be considered.

(4) Performance verification of the bearing capacity in piles under design situations apart from the accidental situations considering Level 2 earthquake ground motion

① The bearing capacity of the piles in the piled piers can be verified using **equation** (**5.2.8**). The symbol γ is the partial factor corresponding to the suffix, where the suffixes k and d indicate the characteristic value and the design value, respectively. As for the partial factors in the relevant equation, the values shown in **Table 5.2.3** can be used. The values denoted as "-" in **Table 5.2.3** indicates that the values may be verified using the values enclosed in parentheses ( ) to ensure convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \qquad \textbf{(5.2.8)}$$

where

R : resistance term (kN/m),

S : load term (kN/m),

γ_R : partial factor that is to be multiplied with the resistance term,

γ_S : partial factor that is to be multiplied with the load term, and

m : adjustment factor.

Table 5.2.3 Partial Factors Used for Performance Verification Regarding the Bearing Capacity in Piles

Verification target	Type of piles	Partial factor to be multiplied with resistance term γ_R	Partial factor to be multiplied with load term γ_S	Adjustment factor m
Bearing capacity of the open-type wharves on vertical piles (variable situation for surcharges during ship actions)	Pulling pile	– (1.00)	– (1.00)	3.00
	Pushing pile	– (1.00)	– (1.00)	2.50
Bearing capacity of the open-type wharves on vertical piles (variable situation during storms, high waves, and Level 1 earthquake ground motion)	Pulling pile	– (1.00)	– (1.00)	2.50
	Pushing pile (bearing pile)	– (1.00)	– (1.00)	1.50
	Pushing pile (friction pile)	– (1.00)	– (1.00)	2.00

② The characteristic value of the bearing capacity of the piles in piled piers can be appropriately estimated in accordance with **Part III, Chapter 2, 3.4.3 Axial Pushing Resistance of Piles** and **Part III, Chapter 2, 3.4.4 Axial Pulling Resistance of Piles**, corresponding to the ground characteristics and an analysis method for pile lateral resistance. In this case, for calculating the characteristic value of the bearing capacity of piles on a sloping surface, the soil strata below the virtual ground surface can be considered to be the effective bearing strata.

③ With respect to the virtual ground surface, refer to **Part III, Chapter 5, 5.2.2 Setting the Basic Cross-section**.

(5) Examination of the Embedment Length for Lateral Resistance

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

② The embedment lengths of the vertical piles are generally set to be $3/\beta$ below the virtual ground surface based on the results of the pile lateral resistance analyses. The value of β can be set in accordance with **Part III, Chapter 5, 5.2.2 Setting of Basic Cross-Section**.

③ Even though the methods for analyzing a single pile receiving lateral force include the PHRI method and Chang's method, it is preferable to use the PHRI method for the estimate described in **Part III, Chapter 2, 3.4.6 Deflection of Piles Receiving Lateral Load**. However, the analytical result of the behavior of piles when the actions occur is almost the same using both Chang's method and the PHRI method for piles with free length-like piled pier structure. Therefore, the virtual fixed-point method based on Chang's method may be used. In the virtual fixed-point method based on Chang's method described here, Chang's method is adapted for calculating the lateral resistance of a single pile. Further, to ensure that the pile head reaction and the pile head flexural moment set by the aforementioned method are the same as those of the beams with fixed ends, the leg length of the lower end embedded rigid frames should be set for each pile.

④ The method mentioned above in ② can be applied as an analysis method for a single pile receiving force from the perpendicular direction by setting $1/\beta$ in Chang's method as the virtual fixed point if stability analysis is conducted under horizontal force. Namely, Chang's method is the obtained solution if the pile length under the ground is assumed to be infinite. In addition, the range of finite underground pile length to which the method can be applied was examined, and it was observed that no large error occurs even if piles with finite lengths are treated as those with infinite lengths if the embedment length of piles is $3/\beta$ or longer.

If the range of approximation between the piles with infinite lengths and the piles with finite lengths is widened in Chang's method, an embedment length of up to $2/\beta$ can be accepted for each vertical pile. However, it is preferable to avoid an embedment length shorter than $2/\beta$ using the virtual ground surface.

⑤ Even when Chang's method is used, if the solution method obtained using the boundary condition of finite embedment length is adopted, it does not have to be in accordance with the method ②.

⑥ If the lateral resistance of the piles is analyzed based on the PHRI method, the minimum embedment length of piles may become $1.5l_{ml}$. Here, l_{ml} is generally the depth from the ground surface to the flexural bending moment second zero point of the fixed head piles.

(6) Other examination

① **Examination of the Pile Joints**

(a) When a pile joint is required in a pile, it is preferable to ensure that the pile can maintain its stability against the impact stress generated in the joint during driving.

(b) The location of the pile joint shall be carefully determined to avoid the portion with excessive stress.

(c) For the method for joining piles, refer to **Part III, Chapter 2, 3.4.12 Particulars.**

② Change of the Plate Thickness or Material of Steel Pipe Pile

(a) Any change in the plate thickness or material along the same steel pipe pile must be made in accordance with **Part III, Chapter 2, 3.4.12 Particulars.**

(b) The strengths of the joints and portion with change in steel thickness should be examined carefully because there are some examples[17)] in which the piles of open-type wharves buckled at these portions because of ground deformation in a deep ground at which no bending stresses were generated under normal load conditions.

5.2.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situations

① The performance verification of the open-type wharves on vertical piles in case of Level 2 earthquake ground motion in accidental situations should be appropriately conducted by considering the situation and importance of facilities in question, and the accuracy of analysis method, and so on.

② The examples of performance verification methods include (a) a method in which the cross-sections for verification shall be set with the dynamic analysis of the lumped mass system; further, one-body analysis of the piled piers and the ground shall be conducted with the nonlinear seismic response analysis by considering the three-dimensional dynamic interaction of the piles and the ground and (b) a method in which the cross-sections for verification shall be set in the same way as above; further, using the ground deformation around the piles that have been separately calculated, the response displacement method will be performed using the frame structure of the piled piers.

③ The deformation of the ground at the time of earthquakes, which leads to the damage of the piled piers, is largely caused by the presence of the soil-retaining sections and soil improvement and the specifications of these sections. For example, for piled piers without soil-retaining sections, if it is judged that the ground is good, that the deformation of the ground would not damage the piled piers, that the ground is not in a good condition, that the deformation is controlled by soil improvement, etc., the verification of the piled piers will be conducted using only the dynamic analysis of the lumped mass system. It is necessary to perform verification using appropriate methods in accordance with the situations.

④ For setting the cross-section for verification, nonlinear dynamic analysis of a spring-mass model with single mass or double masses may be used if a container crane has been installed. The system comprises a spring equivalent to the modeled load–displacement relation of the piled pier structure obtained from the elasto–plastic analysis.

⑤ If container cranes or other cargo-handling equipment are installed on a piled pier, the seismic response characteristics of the piled pier may be considerably altered depending on the ratio of the mass of the cargo-handling equipment to that of the piled pier and the ratio of their natural periods. Therefore, it is necessary to perform a seismic response analysis that considers the coupled oscillations of the cargo-handling equipment and the piled pier. For details, refer to **Part III, Chapter 7, 2.2.3 Performance Verification of Earthquake-Resistance.**

⑥ Apart from the inertia forces acting on the superstructure of the piled pier, the factors that exhibit an adverse effect on the piles include the transmission of the deformation of the ground around the earth-retaining section to the superstructure through the access bridge and the transmission of the forces to the piles when the soil around the piles moves toward the sea owing to the deformation of the soils at this location. Therefore, the structure of the access bridge should be such that deformation of the soils around the earth-retaining section does not adversely affect the superstructure of the piled pier.

⑦ When performance verification using nonlinear seismic response analysis etc. is conducted, it is preferable that the effect of inertial force and added mass for pipe water mass in piles of piled piers should be adequately considered.

⑧ As for the bending strength of the steel pipe piles, as the ratio of diameter D to plate thickness t (D/t) increases, the bending strength becomes lower than the fully plastic moment calculated with cross-section calculation. This tendency is observed to become stronger as the axial force ratio increases. Therefore, when the models of steel pipe piles used for nonlinear seismic response analysis are set, it is necessary to consider the D/t ratio of pipes.

For modeling the steel pipe members in the seismic response analysis, the relation between the bending moment and the curvature, which was obtained by three-dimensional FEM analysis with shell elements, can be replaced with the relation of bilinear type using beam element as depicted in **Fig. 5.2.13**. The ultimate curvature ϕ_u is the curvature at the time when bending moment reached to the maximum bending strength in the three-dimensional FEM analysis. In the bilinear type (beam element analysis), the colored area in the figure under the curve up to the point at which the maximum bending strength M_{max} occurs in the three-dimensional FEM analysis was calculated, and the curvature at the point at which the area under the bilinear lines is the same is considered to be the ultimate curvature ϕ_u.

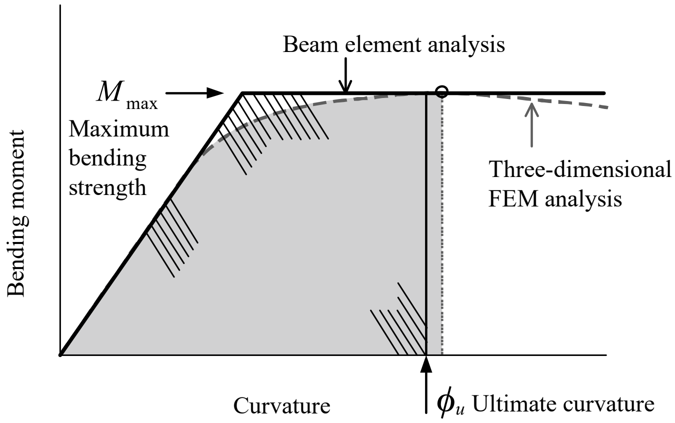


Fig. 5.2.13 Calculation Method of the Ultimate Curvature of the Beam Element (Bilinear Type)

In the beam element based on the infinitesimal deformation theory, if the relation between the bending moment and the curvature of piles are defined in the bilinear type, the ultimate curvature ϕ_u can be calculated from **equations** (**5.2.9**) and (**5.2.10**) by considering the diameter-to-thickness ratio[18].

If the axial force is compressive ($N \geq 0$),

$$\left.\begin{aligned} \phi_u &= \mu \phi_y{}' \\ \phi_y{}' &= \frac{\sigma_y{}' Z}{EI}\left(1 - \frac{N}{N_{yc}{}'}\right) \end{aligned}\right\} \qquad \textbf{(5.2.9)}$$

If the axial force is tensile ($N < 0$),

$$\left.\begin{aligned} \phi_u &= \mu \phi_y \\ \phi_y &= \frac{\sigma_y Z}{EI}\left(1 + \frac{N}{N_{yt}}\right) \end{aligned}\right\} \qquad \textbf{(5.2.10)}$$

where

ϕ_u : the ultimate curvature (1/mm),

ϕ_y : the curvature corresponding to the yield moment (1/mm),

ϕ_y' : the curvature corresponding to the yield moment in consideration of the reduction in yield stress in the direction of the axial compression (1/mm),

EI : the bending rigidity (N·mm^2),

N_{yc}' : the yield axial force in consideration of the reduction in yield stress in the direction of the axial compression (positive value, N),

N_{yt} : the yield axial force when a steel pipe is subjected to the tensile axial force (negative value, N),

Z : the section modulus (mm^3),

σ_y : the yield stress (N/mm^2),

σ_y' : the yield stress in the direction of the axial compression (N/mm^2), and

μ : ductility factor.

$$\left.\begin{array}{ll} \mu = \gamma\left[(-1.24l/r+209)t/D-0.0119l/r+1.46\right] & \text{(with a retained circular shape)} \\ \mu = \gamma\left[(-4.72l/r+440)t/D+0.0413l/r-2.55\right] & \text{(with an unretained circular shape)} \end{array}\right\}$$

t : the wall thickness (mm)

D : the diameter (mm)

l : the effective member length (mm) (refer to **Reference (Part III), Chapter 1, 2.5.2 Modeling Method of Piles**)

r : the radius of gyration of the cross-section (mm); and

γ : the correction coefficient with respect to yield stress

$$\gamma = \sqrt{235/\sigma_y}$$

As for the correlation coefficient with respect to yield stress, the applicability of up to 450 N/mm^2 has been confirmed[19]. The symbol " ′ " (prime) indicates that the yield stress in the axial compression direction is reduced corresponding to the diameter-to-thickness ratio in accordance with **equation (5.2.11)** [20]. Because the purpose of this equation is reduction corresponding to the diameter-thickness ratio, the upper limit of the reduction coefficient shall be 1, and σ_y' should not become larger than σ_y.

$$\sigma_y' = \sigma_y\left(0.86+5.4t/D\right) \qquad \textbf{(5.2.11)}$$

⑨ The standard limit value of deformation in an accidental situation related to Level 2 earthquake ground motion shall be adequately set, and refer to **Part III, Chapter 5, 1.5 Matters concerning High Earthquake-resistance Facilities.**

⑩ With respect to the performance verification of the ultimate curvature of the piles of the open-type wharves on vertical piles for Level 2 earthquake ground motion, if the concerned piled pier is high-earthquake-resistant facilities (designated (emergency supply transport) and designated (trunk line cargo transport)); if no pile, which reaches the ultimate curvature at two points in a pile, exists in the cross-section of the concerned piled pier, it is generally considered that the performance requirements of the piles of the concerned piled pier are satisfied.

With respect to high earthquake-resistance facilities (standard (emergency supply transport)), at least one pile, which reaches the ultimate curvature at less than two points per one pile, exists in the cross-section of the concerned piled pier; therefore, it is generally considered that the performance requirements of the piles of the concerned piled pier are satisfied. **Fig. 5.2.14** schematically depicts the above.

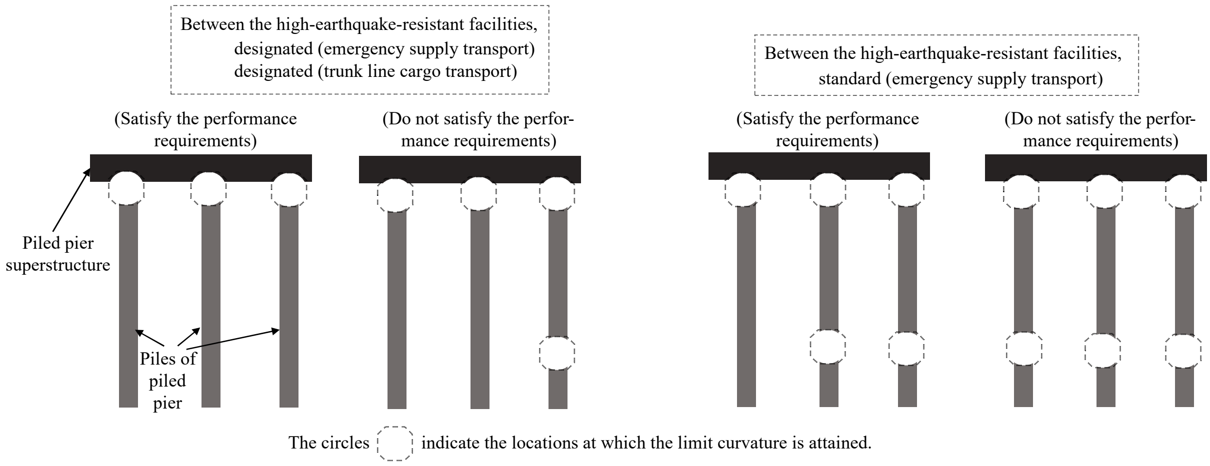


Fig. 5.2.14 Conceptual Diagram of the Counting Piles Reaching the Ultimate Curvature

5.2.6 Performance Verification of the Structural Members

(1) Performance verification can be conducted in accordance with **Part III, Chapter 2, 2 Members of Structure**.

(2) It is necessary to confirm that there will be no loss of the required function because of the deterioration of the concrete superstructure and the steel pipe pile substructure owing to material degradation during the design service life. In particular, there have been several cases where the performance requirements of the concrete superstructures have not been achieved owing to chloride-induced corrosion; therefore, a detailed maintenance management plan should be prepared and followed.

(3) It is necessary to verify that the flexural moment, axial force, and shear force acting on the connections between the steel pipe piles and the superstructure do not reach the ultimate limit state. If the buckling of reinforcing steel or the flaking of covering concrete occurs at the pile heads, the rigid-connecting condition assumed in the calculation of the response values shall not be satisfied, and the actual response value would differ from the response value expected from the calculation. Therefore, attention is required.

(4) In the performance verification of the piled piers, the analysis is performed by assuming the formation of rigid connections between the pile heads and the concrete beams. Further, it is necessary that the pile head flexural moment can be smoothly distributed to the pile head and the concrete beam. The flexural moment that can be distributed to the beam M_{ud} may be calculated using **equation** (**5.2.12**), ignoring the reinforcement connection plates or the vertical ribs that are provided, as necessary. In the following equation, the suffix *d* indicates the design value.

$$M_{ud} = \frac{DL^2 f'_{cd}}{6\gamma_b} \quad \textbf{(5.2.12),}$$

where

M_u denotes the flexural moment that can be distributed to the part of the pile embedded in the beam (N.mm),

D : diameter of the steel pipe pile (mm),

L : embedded length of the steel pipe pile (mm),

f'_{cd} : design value of the compressive strength of beam concrete (N/mm^2), and

γ_b : member factor (may be considered to be 1.15).

(5) It is assumed that the axial forces are distributed by only the bond among the outer peripheral surface of the piles and the vertical ribs, which are provided, as necessary, as well as the concrete. In this case, the axial force that can be distributed, P_{ud}, can be calculated from **equation** (**5.2.13**). In the following equation, the suffix *d* indicates the design value.

$$P_{ud} = \frac{1}{\gamma_b}\left(L\varphi + 2A_p\right) f_{bo_d} \quad \textbf{(5.2.13)}$$

where

P_u : axial force that can be distributed to the part of the pile embedded (N),

L : embedded length of the steel pipe pile (mm),

φ : outer perimeter of the steel pipe pile (mm), and

f_{bod} : design value of the bond strength between the pile and the concrete (N/mm²).

$f_{bod} = 0.11\, f'_{ck}{}^{2/3} / \gamma_c,$

where

f'_{ck} : characteristic value of the compressive strength of the concrete (N/mm²),

γ_c : material coefficient of concrete (= 1.3),

A_p : area of vertical ribs that bonds with concrete (mm²), and

γ_b : member factor (may be considered to be 1.0).

(6) It is necessary to verify that failure due to punching shear forces in the horizontal direction shall not occur in the beam at the end of which the steel pipe pile is embedded.

(7) With regard to the reinforcing steels of bars, it is necessary that the fixation shall be ensured using measures, such as welding to the steel plates installed at the pile heads of the steel pipe piles, and that the transmission of force between the beams and steel pipe piles should be smooth.

5.3 Open-type Wharves on Coupled Raking Piles

5.3.1 General

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

(2) The open-type wharf on coupled raking piles is a structure that resists the horizontal force acting on the wharf, such as the seismic actions, fender reaction force, and tractive force of ships with coupled raking piles. Therefore, this type of wharf must be constructed on ground that yields sufficient bearing capacity for coupled raking piles.

(3) Because the coupled raking piles are so laid out to resist the horizontal forces in the direction normal to the face line of the wharf, the horizontal displacement in that direction is smaller than that of open-type wharves on vertical piles. Coupled raking piles are seldom laid out to resist the horizontal forces in the direction of wharf face line. Therefore, it is preferable to examine the strength of the wharf against the horizontal force parallel to the face line in the same manner as the examination for open-type wharves on vertical piles.

(4) For the procedure for performance verification of open-type wharves on coupled raking piles, refer to **Fig. 5.2.2** of **Part III, Chapter 5, 5.2.1 General** for open-type wharves on vertical piles.

(5) Verification for the variable situations in respect of Level 1 earthquake ground motion may be carried out by obtaining the natural periods of the piled pier with frame analysis and calculating the seismic coefficient for verification with the acceleration response spectrum corresponding to the natural periods. However, as for high earthquake-resistance facilities, appropriate dynamic analysis methods, such as nonlinear seismic response analysis taking account of the 3-dimensional dynamic interaction between the piles and the ground, may be used for verification. For Open-type wharves on coupled raking piles that are not high earthquake-resistance facilities, verification in accidental situations in respect of Level 2 earthquake ground motion can be omitted.

(6) For consideration of the maintenance of open-type wharves on coupled raking piles, refer to **Part III, Chapter 5, 5.2.1 General** for wharves on vertical piles. In addition, as the head parts of coupled piles of open-type wharves on coupled raking piles are narrow, inspection and diagnosis, and countermeasures during the working life are difficult to carry out without special care.

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

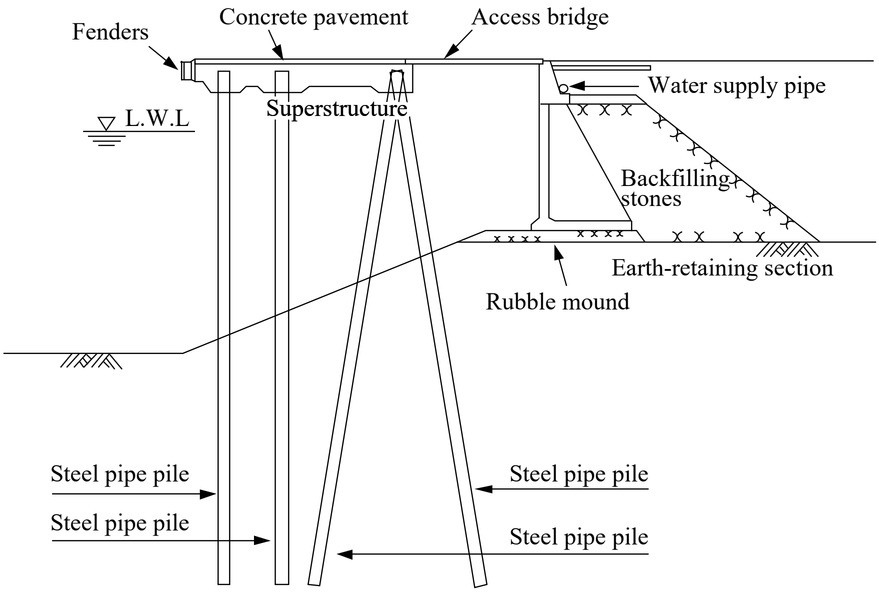


Fig. 5.3.1 Example of Cross-section of an Open-Type Wharf on Coupled Raking Piles

5.3.2 Setting of Basic Cross-section

(1) In the case of open-type wharves on coupled raking piles, the piles come close to adjacent vertical piles and the earth-retaining section; therefore, it is preferable that the layout of the piles be carefully determined considering the construction conditions and the conditions of use.

(2) A large wharf for a design ship size of 10,000 DTW class has one or two sets of coupled raking piles behind one vertical pile in the direction normal to the wharf face line. The distance between piles or between centers of coupled raking piles is usually set to be 4 to 6 m in consideration of loading conditions and construction work.

(3) It is preferable to use a small raking angle of coupled piles from the viewpoint of securing resistance against horizontal force, but in many cases an inclination of 1 : 0.33 to 1 : 0.2 (around 11 to 18 degree from the vertical surface) is used because of constraints related to the required distances from other piles and construction work-related constraints, such as the capacity of the pile driving equipment available.

(4) The beam widths of superstructures of open-type wharves on coupled raking piles depend on how the heads of coupled piles are connected. In general, they tend to be wider than those of open-type wharves on vertical piers.

(5) For setting the basic cross-section of open-type wharves on coupled raking piles, refer to **Part III, Chapter 5, 5.2.2 Setting of Basic Cross-section** in this Chapter.

5.3.3 Actions

(1) Regarding actions to be considered in performance verification of open-type wharves on coupled raking piles, **Part III, Chapter 5, 5.2.3 Actions** can be referred to.

(2) The characteristic value of the seismic coefficient for verification used in the performance verification of open-type wharves on coupled raking piles for the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics of the wharf. For calculation of the seismic coefficient for verification of open-type wharves on coupled raking piles, refer to **Part III, Chapter 5, 5.2.3 (2) ⑭ Ground Motion and Seismic Coefficient for Verification used in Performance Verification of Seismic-resistant**.

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

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

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

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

$$H_i = \frac{C_i}{\sum_i C_i} H + \frac{C_i x_i}{\sum_i C_i x_i^2} eH \qquad \textbf{(5.3.1)}$$

where,

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

H : horizontal force acting on the block (N/m)

H_i : horizontal force distributed to each pile (N/m)

e : distance between center line of pile group and the acting horizontal force (m)

x_i : distance from each pile group to the center line of a pile group (m)

□$_i$: total pile length (m), being substituted the pile length of the friction pile □ when pulling-out forces are acting.

A_i : cross-sectional area of each pile (m²)

E_i : Young's modulus of each pile (N/m²)

θ_{i1}, θ_{i2} : angle of each pile with the vertical direction (°)

The subscript i refers to the ith pile.

The subscripts 1, 2 refer to each pile in one pile group.

The center line of a pile group may be obtained from $\Sigma C_i\xi_i/\Sigma C_i$. ξ_i are the coordinates from an arbitrary coordinate origin of each pile group in face line direction.

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

1) Sandy soil

Equation (5.3.1) is used, substituting, $\frac{2l_i + \lambda_i}{3}$ for □$_i$.

2) Cohesive soil

Equation (5.3.1) is used, substituting, $\frac{l_i + \lambda_i}{2}$ for □$_i$.

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

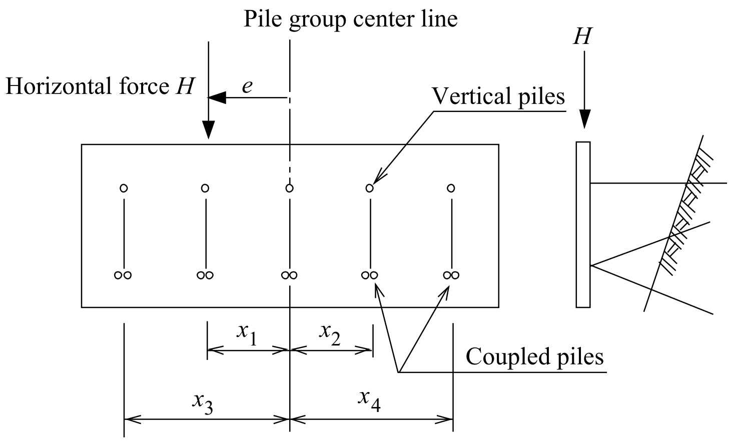


Fig. 5.3.2 Pile Group Center Line and Distance from each Pile Group

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

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

where, n: number of coupled piles

5.3.4 Performance Verification of Open-type Wharves on Coupled Raking Piles

(1) Items for the Performance Verification of Open-type Wharves on Coupled Raking Piles

The performance verification of open-type wharves on coupled raking piles shall apply **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers** and be based on the following.

(2) Performance Verification of Earth-retaining Sections

① For the performance verification of earth-retaining sections, refer to Part III, 5Chapter 5, .2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers.

② It is necessary to ensure that the action due to deformation of the earth-retaining section by earthquakes shall not be transmitted to the superstructure of the piled pier via the access bridge, and that the piles are not adversely affected by significant deformation of the soil around the piles toward the sea. Because earth-retaining sections and piles are close to each other in case of raking piles, special attention is required in deciding the location of earth-retaining sections.

(3) Verification of Stresses in Piles

① The cross-sectional stress in each pile may be calculated by applying **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers** in this Chapter for piles subject to axial forces or piles subject to axial forces and flexural moments.

② However, if partial factors are applied in performance verification regarding axial compressive stress of piles in the variable situation (ship berthing) of open-type wharves on coupled raking piles, **Table 5.2.1** in **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers** cannot be applied. In this case, the following **Table 5.3.1** can be used. Values shown as “-” in **Table 5.3.1** means that the values may be verified using values enclosed in parentheses () as a matter of convenience. These values are factors set referring to allowable stress and the like in the previous design methods.

Table 5.3.1 Partial Factors in Variable Situation by Ship Berthing of Open-type Wharves on Coupled Raking Piles

Verification target	Partial factor to be multiplied by resistance term γ_R	Partial factor to be multiplied by load term γ_S	Adjustment factor m
Compressive stress occurring in the piles of a piled pier (variable action due to ship berthing force)	– (1.00)	– (1.00)	1. 67

③ If coupled piles are laid out in the direction normal to the face line of the wharf, it is preferable that the stress of each pile in that direction is 20 to 30% lower than the yield stress based on **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-Type Wharves on Vertical Piles**, in order to deal with the flexural moment or secondary stress not considered in the verification. On the other hand, as for the direction of the wharf face line, stress can be calculated in accordance with **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-Type Wharves on Vertical Piles**.

④ As open-type wharves on coupled raking piles are mostly constructed on ground where sufficient bearing capacity can be expected, it is preferable that special attention is paid to examine impact stress by driving, buckling, etc. For examination, refer to **Part III, Chapter 2, 3.4 Pile Foundations**.

(4) Performance Verification of Bearing Forces on Piles

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

② The pushing-in and pulling-out forces on each raking pile are axial forces of each raking pile obtained with a frame analysis method, taking into consideration the effect of the raking angle of the pile as indicated in **Part III, Chapter 2, 3.4.8 Calculation of Deflection of Piles by PHRI Method**, calculating the ratio of the coefficient of lateral subgrade reaction, and appropriately correcting the coefficient of lateral subgrade reaction.

③ For verification of pushing-in and pulling-out forces in each raking pile, refer to **Part III, Chapter 2 3.4.3 Pushing-in Resistance Force in Axial Direction of Piles** and **Part III, Chapter 5, 3.4.4 Pulling-out Resistance Force in Axial Direction of Piles**.

④ For partial factors used for verification of bearing forces on piles, refer to **Table 5.2.3** in **Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers**.

(5) Verification of Pile Embedment

For the embedment length of raking piles, refer to Part III, Chapter 5, 5.2.4 Performance Verification Regarding Open-type Wharves on Vertical Piers.

(6) Analysis in the Face Line Direction

① If there are coupled raking piles in the face line direction, the analysis should be carried out in the same way as the direction perpendicular to the face line.

② In the sections of open-type wharves on coupled raking piles other than special locations like the pointed end, it is not rare that coupled piles are laid out with respect to actions in the face line direction of wharfs due to restrictions such as processes of driving raking piles and construction equipment. If coupled piles are not laid out in the face line direction, piles resist with lateral resistance to actions in the face line direction. Therefore, stresses, etc. of piles shall be examined in the same way as open-type wharves on vertical piers. In that case, the virtual ground surface, the virtual fixed point, and the like may be considered in the same way as open-type wharves on vertical piers.

③ Horizontal forces in the face line direction include actions of earthquake ground motion, tractive force by ships, and fender reaction force. Although the superstructures of piled piers were equipped with joints for each block interval, but horizontal displacements are actually transmitting each other. Thus, actions like the tractive force and the berthing force, which do not occur simultaneously at individual locations of the whole mooring facility, occur not only in one block but are distributed to a certain section of a piled pier and therefore the stress of a pile would not become so dangerous as the stress of coupled piles in the normal direction to the face line. However, the earthquake ground movement actions work simultaneously to the whole piled piers and therefore require consideration.

5.3.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situation

Performance verification of open-type wharves on coupled raking piles to Level 2 earthquake ground motion in accidental situations needs to be carried out appropriately, taking into account the situation in which the facilities in question, the importance and accuracy of the analysis method, and so on. Performance verification of open-type wharves on coupled raking piles in accidental situations can be conducted in accordance with **Part III, Chapter 5, 5.2.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situation**.

5.3.6 Performance Verification of Structural Members

For performance verification of structural members such as superstructures and access bridges, refer to **Part III, Chapter 5, 5.2.6 Performance Verification of Structural Members** in this Chapter.

5.4 Strutted Frame Type Pier

5.4.1 General

(1) For maintenance of strutted frame type piers, refer to **Part II, Chapter5, 5.2.1 General**, and **Part II, Chapter5, 5.3.1 General** in this Chapter for open-type wharves on vertical and coupled raking pile, respectively. When designing stiffening members and panel points of strutted frame type piers, appropriate corrosion control measures must be adopted in the same manner as that for steel pipe piles based on **Part II, Chapter 2, 1.3.4 Examination concerning the Change of Performance over Time**.

5.4.2 Actions

(1) Regarding actions to be considered in the performance verification of strutted frame type piers, refer to **Part II, Chapter5, 5.2.3 Actions**.

(2) The characteristic value of the seismic coefficient for verification used in the performance verification of strutted frame type piers against variable situations with respect to Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics. For calculating the seismic coefficient for verification of strutted type piers, refer to **Part II, Chapter5, 5.2.3(2) ⑭ Ground Motion used in Performance Verification of Seismic-resistant and Seismic Coefficient for Verification**.

5.4.3 Performance Verification

(1) For performance verification of strutted frame type piers, refer to **Part II, Chapter5, 5.2 Open-type Wharves on Vertical Piles** and **Part II, Chapter5, 5.3 Open-type Wharves on Coupled Raking Piles,** and also refer to **the Strutted Frame Method Technical Manual**[21].

(2) Verification of Level 2 Earthquake Ground Motion via the Dynamic Analysis Method

① The performance verification of strutted frame type piers in accidental situations with respect to Level 2 earthquake ground motion shall be appropriately conducted considering the concerned circumstances around the facilities, importance of the facility, and the accuracy of the method. Performance verification of strutted frame type piers may basically comply with that of jacket type piled piers.

② The required performance of strutted frame type piers in accidental situations with respect to Level 2 earthquake ground motion is basically the same as that of open-type wharves on vertical piles. In addition, conducting additional examination according to the structure of strutted frame type piers such as stiffening members and panel points is necessary.

5.5 Jacket Type Piled Piers

[Public Notice] (Performance Criteria for Piled Piers)

Article 55

1 The provisions of Article 48 apply mutatis mutandis to the performance criteria of piled piers.

2 In addition to the provisions of the preceding paragraph, the performance criteria of the access bridge of piled piers shall be as prescribed respectively in the following items:

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

(a) The access bridge of piled piers shall have the dimensions necessary for enabling the safe and smooth loading, unloading, embarkation and disembarkation, etc. in consideration of the usage conditions.

(b) The access bridge of piled piers shall not transmit horizontal loads to the superstructure of the piled pier, and shall not fall down even when the piled pier and the earth-retaining part are displaced owing to the actions of earthquakes, etc.

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

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

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

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

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

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

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

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

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

3 The provisions of Article 49 through Article 52 apply mutatis mutandis to the performance criteria of the earth-retaining parts of piled piers in consideration of the structural type.

5.5.1 General

(1) An example of cross-section of a jacket type piled pier is shown in **Fig 5.5.1**.

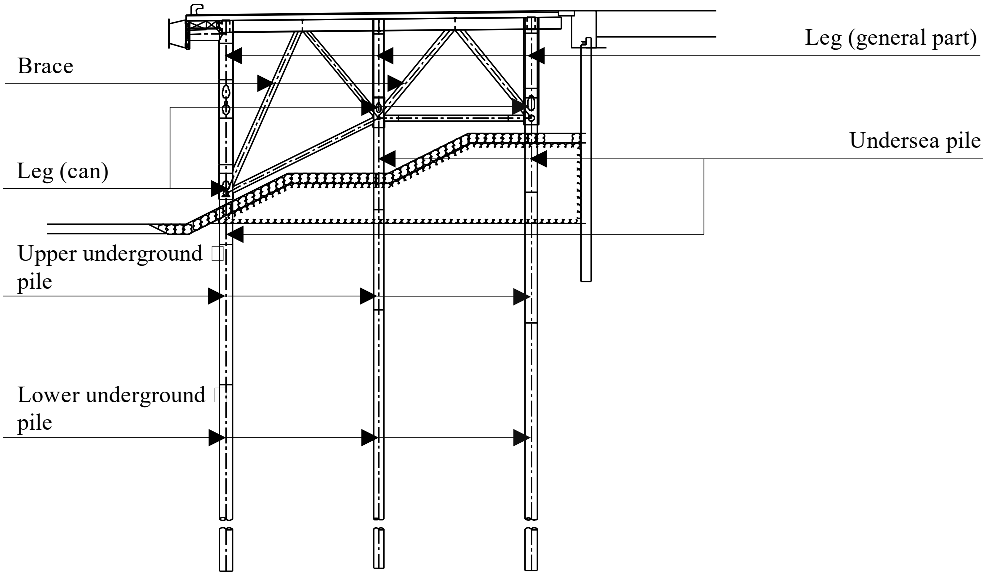


Fig. 5.5.1 Example of Cross-section of a Jacket Type Piled Pier

(2) In some cases, earth-retaining sections are integrated into jacket type piled piers. In one type of such earth-retaining walls, walings are installed and sheet piles are arranged on a straight line. In the other type, no waling is installed and sheet piles are arranged in an arc form.

(3) For consideration for maintenance of jacket type piled piers, refer to **Part III, Chapter5, 5.2.1 General**, **5.3.1 General**, and **Part III, Chapter 5, 5.4 Strutted Frame Type Pier** for open-type wharves on vertical piles, open-type wharves on coupled raking piles, and strutted frame type piers, respectively.

5.5.2 Actions

(1) For actions to be considered in performance verification of jacket type piled piers, refer to **Part III, Chapter5, 5.2.3 Actions**.

(2) The characteristic value of the seismic coefficient for verification used in the performance verification of jacket type piled piers in the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated considering the structural characteristics. For calculation of the seismic coefficient for the verification of jacket type piled piers, refer to **Part III, Chapter5, 5.2.3(2) Ground Motions used in Performance Verification of Seismic-resistant and Seismic Coefficient for Verification**.

5.5.3 Performance Verification

(1) For performance verification of jacket type piled piers or piled piers whose structure has stiffening members, refer to **Part III, Chapter5, 5.2 Open-type Wharves on Vertical Piles**, and **Part III, Chapter5, 5.3 Open-type Wharves on Coupled Raking Piles**; for details, refer to **the Jacket Method Technical Manual**[22].

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

① The performance verification of jacket type piled piers in accidental situations in respect of Level 2 earthquake ground motion shall be appropriately carried out considering the concerned circumstances around the facilities, importance of the facility, and preciseness of the method. The performance verification of jacket type piled piers may comply with that of open-type wharves on vertical piles, but the actions occurring in the members shall be appropriately set considering the structure of the trusses. The different points in the dynamic characteristics between jacket type piled piers and open-type wharves on vertical piles are as follows:

(a) The natural periods are short because jacket type piled piers have truss structure.

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

(c) Separate verification of the panel points is necessary.

② Examples of performance verification methods include a method in which cross-sections for verification shall be set with the dynamic analysis of lumped mass system, and then, using the ground deformation around the piles calculated separately, the response displacement method using the frame structure of piled piers can be applied.

③ As jacket type piled piers generally possess resistance even after buckling of stiffening members, elements that can express characteristics after buckling need to be used in modeling non-linear history of brace material of jackets. The buckling curve may be obtained with methods such as the finite element method and the method assuming plastic hinge. Simple models of approximation with several curves and straight lines may be used[23)].

④ The performance required of jacket type piled piers in accidental situations in respect of Level 2 earthquake ground motion is basically the same as that of open-type wharves on vertical piles. In addition, it is necessary to carry out additional examination according to the structure of jacket type piled piers such as joints between jackets and piles and panel points.

5.6 Dolphins

5.6.1 General

(1) The following may be applied to the performance verification of mooring facilities such as pile type, steel cell type, caisson type, and other types of dolphin structures. Depending on their function, the types of dolphin structures include breasting dolphins, which are used for ships' berthing; mooring dolphins, which are used to hook mooring ropes; and loading dolphins.

(2) It should be noted that the guidelines outlined in **Part III, Chapter 5, 5.6.3 Actions** and **Part III, Chapter 5, 5.6.4 Performance Verification** may be used in simplified verification methods. Therefore, it is preferable to adopt highly precise methods (model experiment or numerical analysis that can reproduce mechanisms).

(3) The performance verification of dolphins should be performed by considering the following items. For other items, it is preferable to appropriately perform performance verification in accordance with each structural form.

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

② Conventional torsion in the case of pile-type structures and rotation in the case of caisson-type structures have not been examined comprehensively. However, these factors may affect the stability of structures in certain cases; therefore, it is necessary to carefully consider these aspects.

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

(4) Consideration is required for the appropriate maintenance of dolphins in accordance with their structural forms.

① For pile type dolphins, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Piles**.

② For steel cell type dolphins, refer to **Part III, Chapter 5, 2.9 Cellular-Bulkhead Quaywalls with Embedded Sections**.

③ For caisson type dolphins, refer to **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls**.

(5) **Fig. 5.6.1** shows an example of a cross section of a pile-type dolphin.

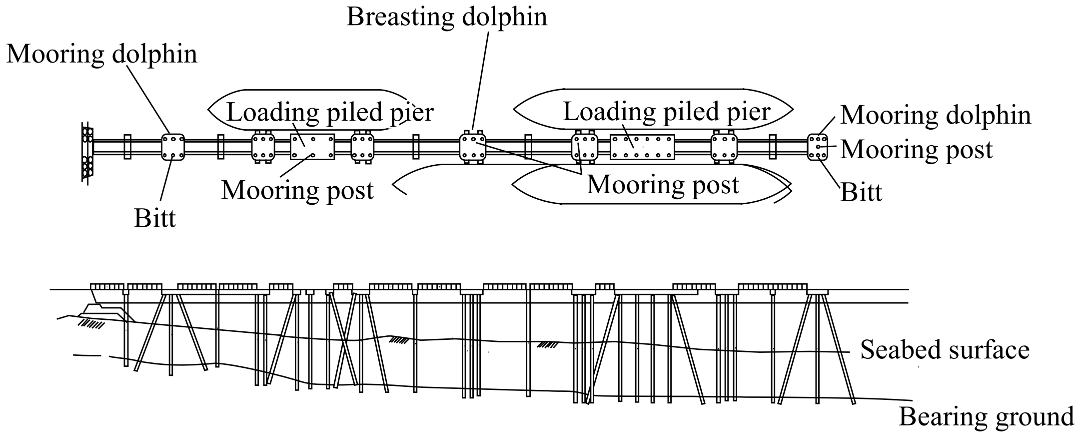


Fig. 5.6.1 Example of a Cross Section of a Pile-Type Dolphin

5.6.2 Layout

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

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

① **Dimensions of the design ship**

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

(b) When special cargo handling equipment is required for dolphins for oil handling, a cargo handling platform dolphin is installed midway between the breasting dolphins. In this case, it is preferable to locate the cargo handling platform with its seaside front slightly backward from that of the breasting dolphins so that the ship berthing force does not act directly on the cargo handling platform dolphin.

② The layout of dolphins should be designed in such a way that the longitudinal axis of dolphins becomes parallel to the prevailing directions of winds, waves, and tidal currents. This layout helps ease ship maneuvering during berthing and unberthing and reduces the external forces acting on the dolphins when the ship is moored. If the directions of winds, waves, etc. are different from those of coastlines, and when dolphins need to be constructed near the coast, the dolphins are usually laid out parallel to the coastline for the usage of the water area because dolphins that are designed for medium-sized or smaller ships near the coast should not have a large wind load. Furthermore, positions that can secure the planning depth at the water depth of the ground are advantageous.

③ Dolphins should be laid out in such a way that avoids adverse effects on the anchorage of other ships in anchorage areas and on the navigation of other ships in navigation channels.

(3) Although mooring posts may be sometimes installed on land when the coast is near to connect mooring ropes for berthing and mooring, mooring posts are usually installed on mooring dolphins. Mooring dolphins are normally set at a 45° angle from the rope bitts on the bow and stern of a ship and with a certain setback from the front face of the breasting dolphins. The number of mooring dolphins is decided on the basis of the tractive force of ships, but two to four units are usually adopted based on previous construction examples. It is also possible to install mooring posts on breasting dolphins and use them for the mooring of design ships and small ships.

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

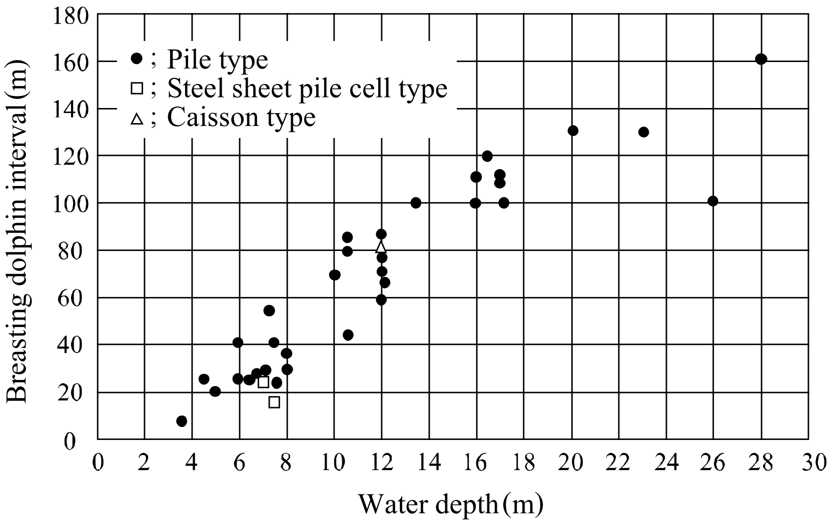


Fig. 5.6.2 Distance between Breasting Dolphins

5.6.3 Actions

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

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

(3) For the calculation of vertical loads due to self-weight and live load, refer to **Part II, Chapter 10, Self-Weight and Surcharge** and **Part III, Chapter 5, 5.2.3 Actions** for open-type wharves on vertical piles.

(4) For the action due to earthquakes, refer to **Part II, Chapter 6, Earthquakes** and **Part III, Chapter 5, 5.2.3 Actions** in this chapter for open-type wharves on vertical piles.

(5) For the calculation of dynamic water pressure during an earthquake, refer to **Part II, Chapter 4, 3.2 Dynamic Water Pressure**.

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

5.6.4 Performance Verification

(1) Pile Type Dolphins

① For the performance verification of pile type dolphins, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Pile**r.

② The characteristic value of the seismic coefficient for the verification used in the performance verification of pile type dolphins in variable situations with respect to Level 1 earthquake ground motion shall be appropriately calculated by considering the structural characteristics. For the calculation of the seismic coefficient for the verification of pile type dolphins, refer to **Part III, Chapter 5, 5.2.3(2)⑭ Ground Motions used in the Performance Verification of Seismic Resistance and Seismic Coefficient for Verification**.

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

④ Large tankers are usually berthed at a slant angle with the dolphin alignment line. Considering that the characteristics of fenders vary depending on the berthing angle, it is recommended to use the characteristics curve appropriate to the berthing angle. Furthermore, a slanting berthing may cause some of the fenders attached to a breasting dolphin to not absorb the berthing energy effectively. Therefore, it is preferable to examine carefully the fenders that will come into contact with the hull of the ship in consideration of the berthing angle.

⑤ For the structure of the joints of the pile heads, **Part III, Chapter 5, 5.2.6 Performance Verification of Structural Members** can be applied with modification as necessary. If steel pipe piles are used, it is preferable to improve the joints with superstructures and to apply filling with concrete around the L.W.L. to avoid pile deformation due to collisions with driftwood or small ships. If material other than concrete is used for the superstructures, verification should be performed appropriately in consideration of the material properties.

⑥ For ancillary facilities such as mooring posts, fenders, skirt guards, and ladders, refer to **Part III, Chapter 5, 9 Ancillary of Mooring Facilities** in this chapter. For connecting piled piers, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Piles**.

(2) Steel Cell Type Dolphins

① For the performance verification of steel cell type dolphins, refer to **Part III, Chapter 5, 2.9 Cellular-Bulkhead Quaywalls with Embedded Sections**.

② The characteristic value of the seismic coefficient for the verification for the performance verification of steel cell type dolphins in variable situations with respect to Level 1 earthquake ground motion shall be appropriately calculated by considering the structural characteristics. The characteristic value of the seismic coefficient for verification of steel cell type dolphins may be calculated in accordance with **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls** when soil pressure is the acting force or in accordance with **Part III, Chapter 4, 3.1 Gravity-Type Breakwaters (Composite Breakwaters)** when soil pressure is not the acting force.

③ For the foundations of cargo handling equipment and mooring posts, refer to **Part III, Chapter 2, 3.4 Pile Foundations** and **9.19 Foundations for Cargo Handling Equipment**.

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

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

where

B : equivalent wall width (m);

R : radius of cylindrical cell (m).

⑤ In general, steel cell type dolphins have reinforced concrete superstructures for the whole top area, and a crown made from steel sheet piles or steel plates is embedded on them. When heavy goods, such as cargo handling equipment, are placed on this structure, a significantly large compressive force acts on the steel sheet piles or steel plates via the superstructures. Buckling may occur on the crown if the weight is not relieved. One countermeasure involves driving piles in the inner filling of the cells for support as pile foundations.

⑥ For the performance verification of structural members, refer to **Part III, Chapter 5, 5.6.4(1) Pile Type Dolphins**. For the verification of members of steel cells, refer to **Part III, Chapter 5, 2.9 Cellular-Bulkhead Quaywalls with Embedded Sections**.

(3) Caisson Type Dolphins

① For the performance verification of caisson type dolphins, refer to **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls** in this chapter.

② The characteristic value of the seismic coefficient for the verification for the performance verification of caisson type dolphins in variable situations with respect to Level 1 earthquake ground motion shall be appropriately calculated by considering the structural characteristics. The characteristic value of the seismic coefficient for the verification of caisson type dolphins may be calculated in accordance with **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls** when soil pressure is the acting force or in accordance with **Part III, Chapter 4, 3.1 Gravity-Type Breakwaters (Composite Breakwaters)** when soil pressure is not the acting force.

③ The rotation of a caisson occurs when an eccentric external force acts on a dolphin. The examination of stability against rotation must be made even when the stability against sliding and overturning, as well as against the failure of the foundation ground due to insufficient bearing capacity, are found to be satisfactory because the confirmation of the stability with respect to these items does not guarantee that the caisson is safe against rotation. In this case, in calculating the resistance force, attention should be given to the friction force of the caisson bottom, which should be proportional to the bottom reaction force, as described in **Part III, Chapter 2, 2.2 Caissons**.

④ For the performance verification of structural members, refer to **Part III, Chapter 5, 5.6.4(1) Pile Type Dolphins**. Furthermore, for the verification of caisson members, refer to **Part III, Chapter 2, 2.2 Caissons**.

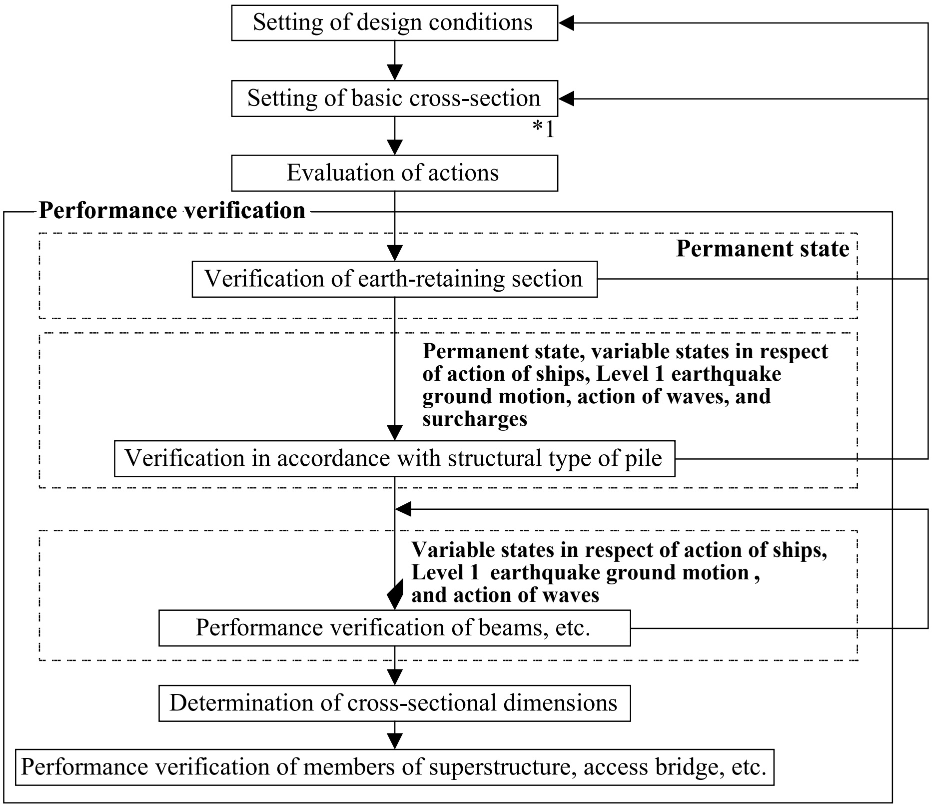
5.7 Detached Piers

5.7.1 General

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

(2) Detached piers are foundations such as a rail-mounted portal bridge crane, constructed at locations with adequate water depth, and used as mooring facilities. In general, a detached pier needs no floor structure and consists of a beam and piles that support the beam.

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



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

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

(4) For consideration for the maintenance of detached piers, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** and **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Piles** with paying due consideration to the structural characteristics.

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

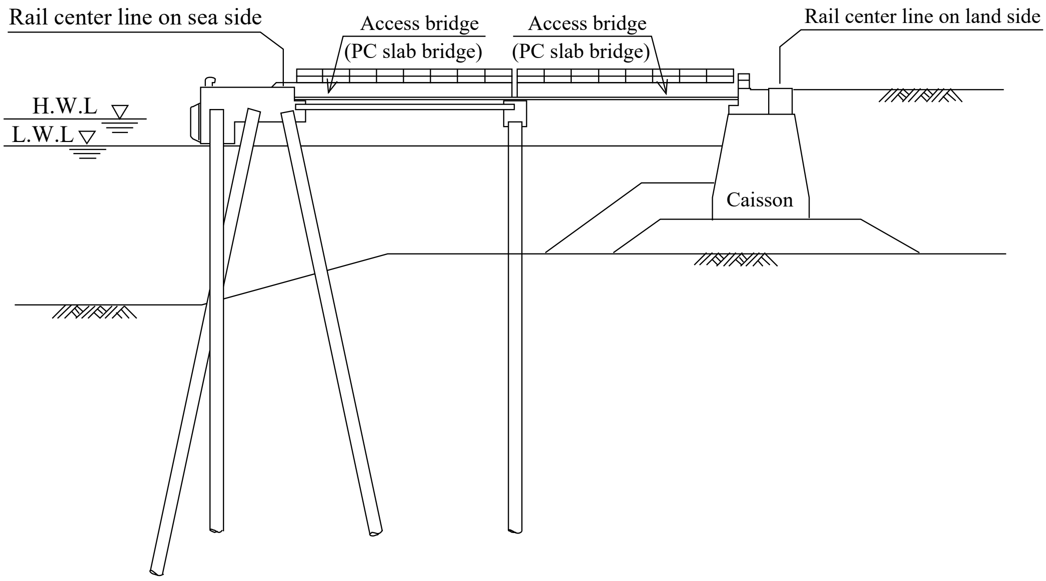


Fig. 5.7.2 Example of Cross-section of a Detached Pier

5.7.2 Setting of Basic Cross-section

(1) The distance between a detached pier and the land as well as the rail interval and the spacing of piles in the face line direction shall be set based on the economic efficiency, constructability, etc. with due consideration given to the dimensions of the mounted cranes, the sea bed, etc.

(2) In general, simple beams are adopted with due consideration given to the uneven settlement of piles.

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

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

(5) In some cases, both rails of cargo handling equipment are laid on a detached pier. In other cases, only one rail is laid on a detached pier while the other rail is laid on the earth-retaining section. In the latter case, it is preferable to construct the foundation of the fixed legs of cargo handling equipment on the land side.

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

5.7.3 Actions

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

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

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

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

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

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

(7) The characteristic value of the seismic coefficient for verification for the performance verification of the detached piers against the variable situations in respect of Level 1 earthquake ground motion shall be appropriately calculated with due consideration given to the structural characteristics. For the calculation of the seismic

coefficient for verification of detached piers, refer to **Part III, Chapter 5, 5.2.3(2) ⑭ Ground Motions used in the Performance Verification of Seismic Resistance and Seismic Coefficient for Verification**.

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

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

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

5.7.4 Performance Verification

(1) The performance verification of the piles of the detached piers may be carried out by appropriately selecting items from performance criteria of **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles**, **Part III, Chapter 5, 5.3 Open-Type Wharves on Coupled Raking Piles**, **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls**, and **Part III, Chapter 5, 2.9 Cellular-bulkhead Quaywalls with Embedded Sections** r, in accordance with the structure type. In addition, the performance verification of the earth-retaining part may be carried out by appropriately selecting items from performance criteria of **Part III, Chapter 5, 2.2 Gravity-type Quaywalls**, **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**, and **Part III, Chapter 5, 2.4 Cantilevered Sheet Pile Quaywalls** and, in addition, refer to the following.

(2) Performance Verification of Piles

Performance Verification of piles shall be carried out appropriately in accordance with the structure type.

(3) Performance Verification of Beams

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

② Structural elements with sufficient strength against the designated vertical and horizontal forces shall be used for the beams of a detached pier because the crane rails for a crane are directly installed on the beams. In the examination of vertical loads, due consideration shall be given to the increase in the wheel loads due to the wind load or seismic force on cargo handling equipment.

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

(4) Performance Verification of Earth-Retaining Sections

① Performance verification of earth-retaining sections shall be carried out appropriately in accordance with the structure type.

② If one rail of the cargo handling equipment is laid at the back of the earth-retaining section, performance verification may be carried out in accordance with **Part III, Chapter 5, 9.19 Foundations for Cargo Handling Equipment**.

5.7.5 Performance Verification of Structural Members

(1) Superstructure

For performance verification of superstructures, refer to **Part III, Chapter 5, 5.2.5 Performance Verification of Level 2 Earthquake Ground Motion in Accidental Situation**.

(2) Access Bridge

For performance verification of the access bridges, refer to the **Specifications and Commentary for Highway Bridges**[23)] and **Technical Standards and Commentary of Elevated Pedestrian Crossing Facilities**[24)].

5.7.6 Structural Detail

(1) Detached piers are generally equipped with ancillary facilities such as fenders, mooring posts, and access bridges.

(2) Access bridges are provided at one to two locations per berth. In addition, if there is not enough space for rope-handling work when ships depart, slabs are required for safety.

(3) For fenders and mooring posts, refer to **Part III, Chapter 5, 9 Ancillary of Mooring Facilities**.

[References]

1) MLIT, Ports and Harbours Bureau: Guidelines for the Development and Maintenance of Ecological Port Facilities, 2014

2) Kato, E., Yamamoto, K., Kawabata, Y. and Iwanami, M.: Application of Corrosion Monitoring Sensor to RC Superstructure of Open-Type Wharf, Technical Note of the Port and Airport Research Institute, No.1307, 2015 (in Japanese)

3) Iwanami, M., Kato, E. and Kawabata, Y.: Development of Structural Design Method of Piers Considering Maintenance Strategy, Technical Note of the Port and Airport Research Institute, No.1268, 2013 (in Japanese)

4) Suzuki, A., Kubo, K. and Tanaka, Y.: Lateral resistance of vertical piles embedded in sandy layer with sloping surface, Report of the Port and Harbour Research Institute, Vol.5, No.2, 1966 (in Japanese)

5) Kikuchi, Y., Ogura, T., Ishimaru, M. and Kondo, T.: Coefficient of lateral subgrade reaction of rubble ground, Proceedings of 53rd Annual Conference of JSCE, 1998 (in Japanese)

6) Yokota, H., Takehana, N., Minami, K., Takahashi, K. and Kawabata, N.: Consideration of Design Seismic Coefficients of An Open Type Wharf Based on Dynamic Response Analyses, Report of the Port and Harbour Research Institute, Vol.37, No.2, 1998 (in Japanese)

7) Kuwabara, N. and Nagao, T.: A fundamental study on the evaluation method of seismic coefficients of pile supported wharves considering the dynamic characteristics, Technical Note of National Institute of Land and Infrastructure Management, No.591, 2010 (in Japanese)

8) Yokoyama, Y.: Calculus and Numerical Examples of Piling Structures, Sankai-do, 1977 (in Japanese)

9) Yamashita, I.: Equivalent Rigid Frame to Vertical Pile Structure on the Basis of the PHRI Method, Technical Note of Port of Harbour Research Institute, No.105, pp.1-12, 1970 (in Japanese)

10) Kubo, K.: A New Method for the Estimation of Lateral Resistance of Piles, Report of the Port and Harbour Research Institute, Vol.2, No.3, pp.1-37, 1964 (in Japanese)

11) Yamashita, I. and Arata, M.: The Standard Curves for the Built-in Head Standard Pile Partially Embedded in the C-Type Soil, Technical Note of the Port and Harbour Research Institute, No.65, pp.13-25, 1969 (in Japanese)

12) Yamashita, I., Inatomi, T., Ogura, K. and Okuyama, Y.: New Standard Curves in the PHRI Method, Report of the Port and Harbour Research Institute, Vol.10, No.1, pp.107-168, 1971 (in Japanese)

13) The Ports and Harbours Association of Japan: the Technical Standards and Commentaries for Port and Harbour Facilities in Japan, last volume, pp.740-743, 1999 (in Japanese)

14) Katsumata, M., Fukunaga, Y., Takenobu, M., Miyata, M. and Kohama, E.: A Study of a Method of Calculating Seismic Coefficients of Open-type Wharves on Vertically Pile Considering the Nonlinearity of Soil Stiffness Accompanying Earthquake Ground Motion, Technical Note of National Institute of Land and Infrastructure Management, No.1020, 2017 (in Japanese)

15) The Ports and Harbours Association of Japan: the Technical Standards and Commentaries for Port and Harbour Facilities in Japan, last volume, pp.1109-1130, 2007 (in Japanese)

16) Katsumata, M., Takenobu, M., Miyata, M. and Murakami, K.: Considerations of Level 1 Reliability Design Method for Vertical Pile-supported Wharves under Berthing Condition (Part 2), Technical Note of National Institute of Land and Infrastructure Management, No.931, 2016 (in Japanese)

17) Minami, K., Takahashi, K., Yokota, H., Sonoyama, T., Kawabata, N. and Sekiguchi K.: Earthquake damage of Kobe Port T Pier and static and dynamic analysis, Kisoko, Vol. 25, No.9, pp. 112-119, 1997 (in Japanese)

18) Ohya, Y., Shiozaki, Y., Kohama, E. and Kawabata, Y.: Proposal of Modeling of Circular Steel Tube for Seismic Performance Evaluation, Report of the port and airport research institute, Vol.55, No.2, pp.3-33, 2017 (in Japanese)

19) Shiozaki, Y., Ohya, Y. and Kohama, E.: M-φ characteristics of high-strength steel pipe piles considering local buckling, Proceedings of the 37th JSCE Earthquake Engineering Symposium, paper ID:12-1242, 8pp., 2017 (in Japanese)

20) Kishida, H. and Takano, A.: The buckling of steel pipe piles and the method to strengthen the end of steel pipe piles, Transactions of the Architectural Institute of Japan, Vol.213, pp.29-38, 1973 (in Japanese)

21) Coastal Development Institute of Technology (CDIT): Technical Manual for Grid Strut Method, 2000 (in Japanese)

22) Coastal Development Institute of Technology (CDIT): Technical Manual for Jacket structures, 2000 (in Japanese)

23) Japan Road Association: Specifications and Commentary for Highway Bridges, Maruzen Publications, 2017 (in Japanese)

24) Japan Road Association: Technical Standards and commentary of elevated pedestrian crossing facilities, 1979 (in Japanese)

6 Floating Piers

[Ministerial Ordinance] (Performance Requirements for Floating Piers)

Article 30

1 The performance requirements for floating piers shall be as prescribed respectively in the following items in consideration of the structural type:

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

(2) Damage, etc. due to the actions of self-weight, variable waves, Level 1 earthquake ground motion, ship berthing, traction by ships, surcharge loads, etc. shall not impair the function of the floating piers, and shall not adversely affect the continuous use of the floating piers.

2 In addition to the provisions of the preceding paragraph, the performance requirements for floating piers in the place where there is a risk of having serious impact on human lives, property, or socioeconomic activity by damage to the floating piers shall be such that the structural stability of the floating piers is not seriously affected even in cases where the function of the floating piers is impaired by design tsunamis, accidental waves, etc.

[Public Notice] (Performance Criteria of Floating Piers)

Article 56

1 The provisions of Article 48, paragraph (1) (excluding item (ii)) apply mutatis mutandis to the performance criteria of floating piers.

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

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

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

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

(4) The following criteria shall be satisfied under the variable situation in which the dominating actions are variable waves, Level 1 earthquake ground motions, ship berthing, traction by ships, and surcharge loads:

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

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

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

4 The provisions of Articles 65 and Article 95 apply mutatis mutandis to the performance criteria of the access facilities of the floating body by taking into account the usage conditions.

[Interpretation]

11. Mooring facilities

(10) Performance Criteria of Floating Piers (Article 30, Paragraph 1 of the Ministerial Ordinance and the interpretation related to 56, Paragraph 2 of the Public Notice)

① **Common for floating piers**

(a) The performance requirement for floating piers shall be serviceability. The serviceability mentioned here indicates that the applicable floating pier should have the necessary dimensions such that the amount of motions of the floating body and the amount of its tilting are within the allowable range and that it should have the necessary freeboard based on the dimensions of design ships and its usage conditions.

(b) When the freeboard is set, the dimensions of the design ships and the envisaged usage conditions shall be considered to allow the safe and efficient embarkation and disembarkation of passengers and the safe and efficient handling of cargos.

(c) The performance requirement for floating piers under the variable situation in which the dominating actions are variable waves, L1 earthquake ground motions, ship berthing and traction by ships, and/or surcharges shall be serviceability. The performance verification items and standard indexes to determine the limit values for such actions shall be as shown in **Attached Table 11-26**. The indexes for determining the limit values for the performance verification items shall be appropriately set.

Attached Table 11-26. Performance Verification Items and Standard Indexes for Determination of Limit Values for the Structural Stability of Floating Piers and the Soundness of Members in Each Design State (Excluding Accidental Situations)

<table>
<tr><th colspan="3">Ministerial Ordinance</th><th colspan="3">Public Notice</th><th rowspan="2">Performance requirement</th><th colspan="3">Design state</th><th rowspan="2">Verification item</th><th rowspan="2">Standard index for determination of limit value</th></tr>
<tr><th>Article</th><th>Paragraph</th><th>Item</th><th>Article</th><th>Paragraph</th><th>Item</th><th>State</th><th>Dominating action</th><th>Non-dominating action</th></tr>
<tr><td rowspan="4">30</td><td rowspan="4">1</td><td rowspan="4">2</td><td rowspan="4">56</td><td rowspan="4">2</td><td>2</td><td rowspan="4">Serviceability</td><td rowspan="4">Variable</td><td>Variable waves</td><td>Self-weight, wind, water pressure, water flow</td><td>Capsizing of floating body</td><td>–</td></tr>
<tr><td>4a</td><td>[Level 1 earthquake ground motion]</td><td>(Self-weight, wind, water pressure, water flow)</td><td>Soundness of members of floating body</td><td>–</td></tr>
<tr><td rowspan="2">4b</td><td>[Ship berthing and traction by ships]</td><td>(Self-weight, support reactions of connecting facilities, wind, water pressure, water flow, surcharges)</td><td>Soundness of members of mooring equipment</td><td>–</td></tr>
<tr><td>[Surcharges]</td><td>(Self-weight, wind, water pressure, water flow)</td><td>Structural stability of mooring equipment</td><td>–</td></tr>
</table>

Note: The descriptions [ ] under dominating action indicate that such dominating action is used as a substitute for the design state.

Note: The descriptions ( ) under non-dominating action indicate that such action is used as a substitute based on the dominating action.

(d) Regarding **Attached Table 11-26**, for the performance verification against the capsizing of floating bodies, standard indexes for determining the limit values for capsizing shall be appropriately set by considering the conditions of use of the floating body and the natural conditions. For the performance verification of the structural members of a floating body and floating equipment, standard indexes for determining the limit values for their soundness and stability shall be appropriately set based on the structural type and material of the members.

(e) The performance verification items and standard indexes to determine the limit values for the soundness of the structural members of mooring equipment with mooring ropes under the variable situation in which the dominating actions are variable waves, ship berthing, and/or traction by ships shall be as shown in **Attached Table 11-27**.

Attached Table 11 -27. Performance Verification Items and Standard Indexes for Determination of Limit Values for the Soundness of the Structural Members of the Mooring Equipment of Floating Piers with Mooring Ropes in Each Design State(Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirement	Desig state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
30	1	2	56	2	4b	Serviceability	Variable	Variable waves	Self-weight, wind, water pressure, water flow	Yielding of mooring ropes	Design yield stress
								(Ship 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, etc. (horizontal, vertical)

Note: The descriptions [ ] under dominating action indicate that such dominating action is used as a substitute for the design state.
Note: The descriptions ( ) under non-dominating action indicate that such action is used as a substitute based on the dominating action.
Note: The verification of the stability of mooring anchors, etc. refers to verifying that the tensile forces acting on the mooring anchors do not exceed the resistance force.

(f) The term "mooring anchors, etc." shown in **Attached Table 11-27** is used as a general term for equipment placed on the seabed to retain floating bodies and includes sinkers in addition to mooring anchors.

② **Floating piers of facilities prepared for accidental incidents** (Article 30, Paragraph 2 of the Ministerial Ordinance and the interpretation related to Article 56, Paragraph 3 of the Public Notice)

(a) The performance requirement for floating piers under the accidental situation in which the dominating actions are design tsunamis and/or accidental waves shall be safety. The performance verification items and standard indexes to determine the limit values for such actions shall be as shown in **Attached Table 11-28**.

Attached Table 11-28. Performance Verification Items and Standard Indexes for Determination of Limit Values for the Floating Piers of Facilities Prepared for Accidental Incidents under Accidental Situations

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
30	2	–	56	3	–	Safety	Accidental	Design tsunami	Self-weight, wind, water pressure, water flow	Yielding of mooring ropes	Design yield stress
								Accidental waves		Stability of mooring anchors, etc.	Resistance force of mooring anchors, etc. (horizontal, vertical)

(b) The verification of the stability of mooring anchors, etc. under accidental situation in which the dominating actions are design tsunamis and/or accidental waves shall be examined with the drifting of floating structures taken into account, which may be caused by design tsunamis and/or accidental waves, in order to ensure that it will not make a serious impact on the surroundings.

③ **Access facilities** (Article 30, Paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 56, Paragraph 4 of the Public Notice)

The performance criteria for vehicle ramp, which is an ancillary equipment of the mooring facilities defined in Article 65 of the Standard Public Notice, and the performance criteria of fixed facilities for embarkation and disembarkation of passengers defined in Article 95 of the Standard Public Notice shall apply to the performance criteria of the access facilities of floating piers in accordance with the envisaged conditions of use of the floating pier. The access facilities of floating piers are those that function between a floating body and the land or between floating bodies as passageways of passengers or vehicles, such as access bridges, gang ways, and adjustment towers.

6.1 Fundamentals of Performance Verification

(1) The performance verification procedures for floating piers shall be applied to those with floating bodies that are moored by mooring ropes (e.g., chains, wires, etc.), dolphins, mooring piles, etc. (hereinafter “pontoons”). Furthermore, the Japanese Fire Service Law (Law 186 in 1948), Japanese **Building Standards Law** (Law 201 in 1950), and/or Japanese **Vessel Safety Law** (Law 11 in 1933) apply to some floating structures.

(2) The methods for verifying the performance of floating piers shall be applied to the floating piers installed in places where the actions from waves, tidal currents, and wind are relatively weak. Floating piers are not normally used in locations where the waves or currents are large but are frequently used in locations where the wave height is 1 m or less and where the current is 0.5 m/s or less.

(3) **Figs. 6.1.1** and **6.1.2** show the notation of the respective parts of a floating pier and a pontoon which is a main structure of the floating pier. As shown in the figure, a floating pier comprises pontoons, an access bridge that connects the pontoons with land, gang ways that interconnect the pontoons, mooring ropes that moor the pontoons, mooring anchors, and other elements.

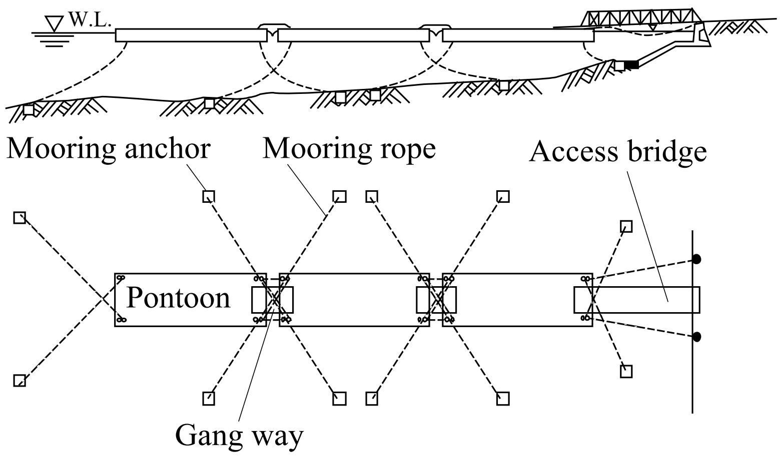


Fig. 6.1.1. Notation of Respective Parts of Floating Pier

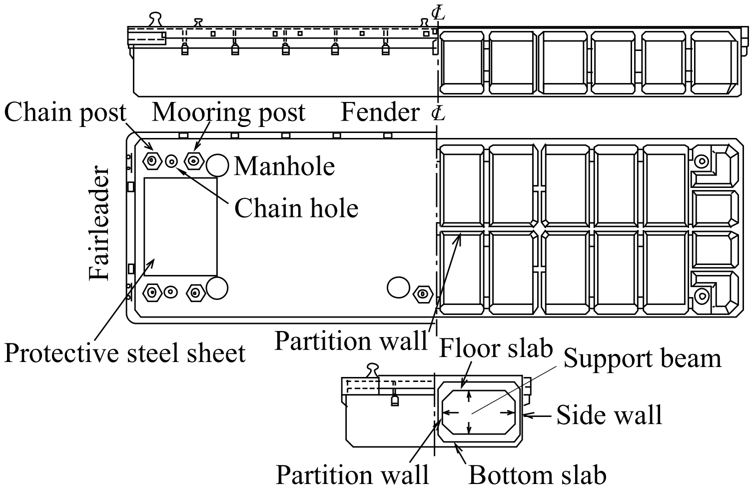


Fig. 6.1.2. Notation of Respective Parts of Pontoon

(4) Floating piers moored by methods other than mooring ropes (e.g., dolphin-fender method) and large-scale floating piers have recently been constructed. **Fig. 6.1.3** and **Fig. 6.1.4** illustrate examples of the overall structure of a large-scale floating pier and its pontoon structure, respectively.

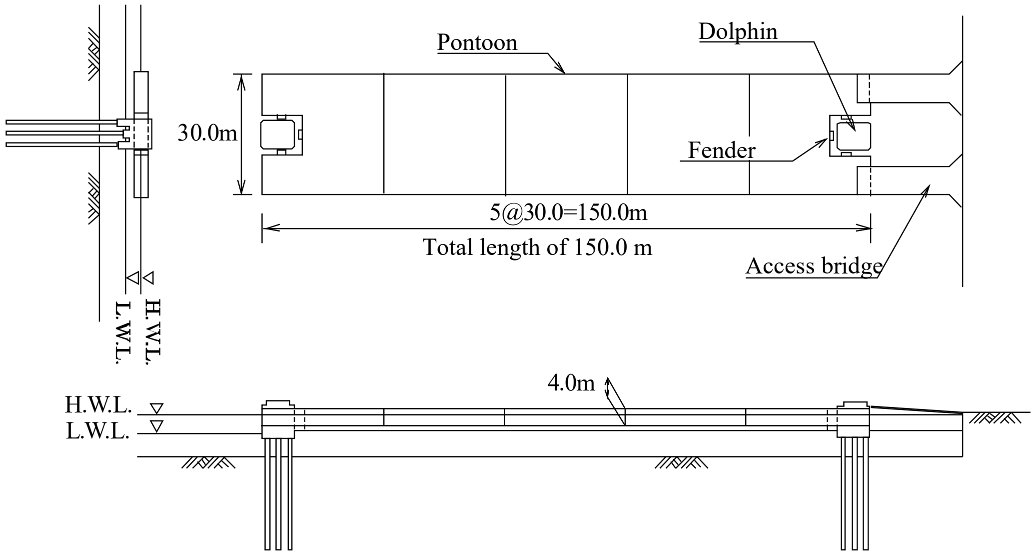


Fig. 6.1.3. Example of Overall Structure of a Large-Scale Floating Pier (Dolphin Mooring Method)

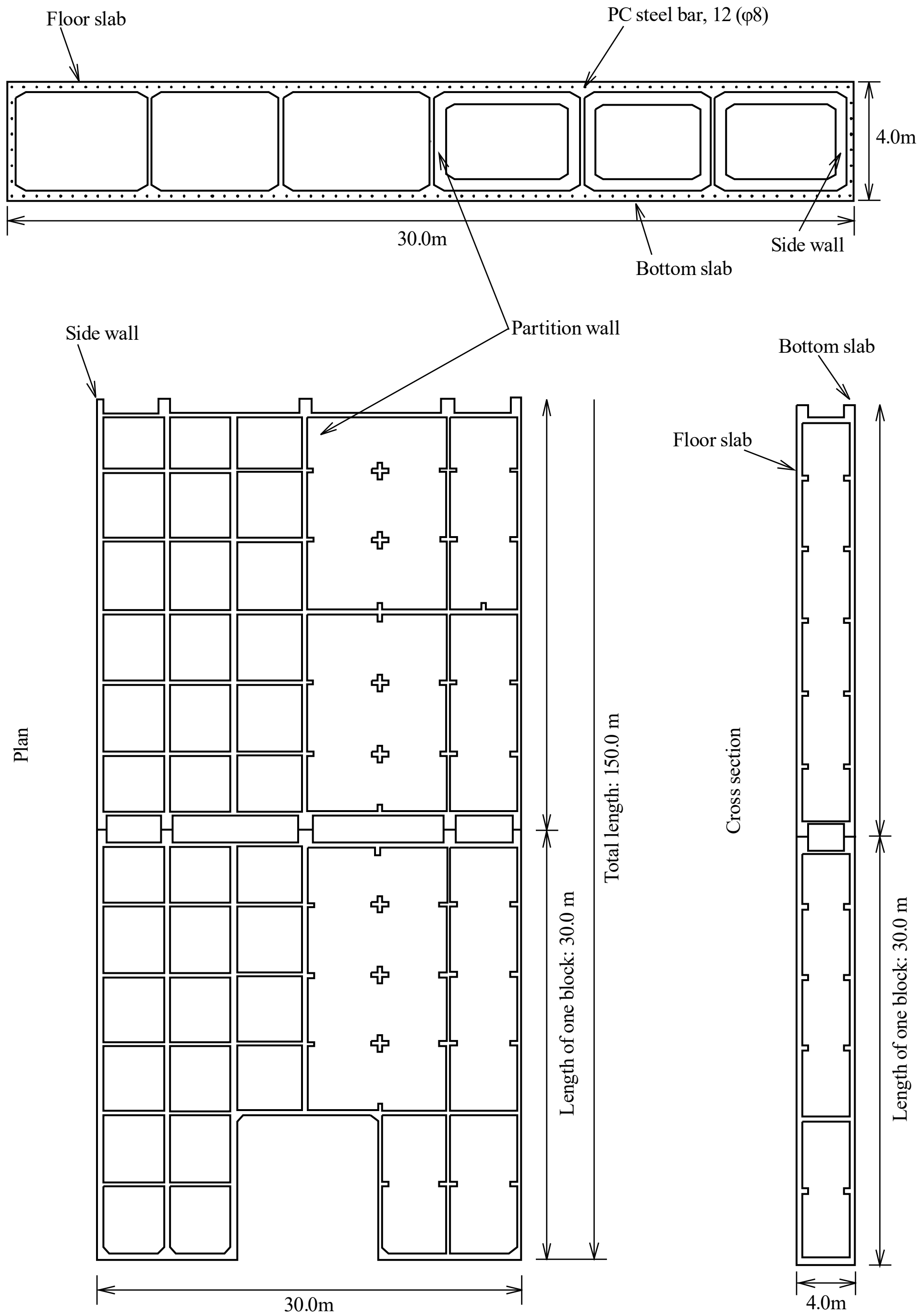


Fig. 6.1.4. Example of Pontoon Structure of a Large-Scale Floating Pier

(5) **The Technical Manual for Floating Structure**[1)] shall be used as a reference for the performance verification of floating piers. Furthermore, **Part II, Chapter 2, 6.4 Wave Force Acting on Structures near the Water Surface**, **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions**, and **Part III, Chapter 4, 3.10 Floating Breakwaters**, etc. shall be used as a reference as necessary. Regarding relatively small floating piers using mooring piles, the **Manual for Design and Construction of Floating Piers (draft)**[2)] can be used as a reference.

(6) For the performance verification of floating piers, the freeboard of the floating pier shall be appropriately set by considering the dimensions of the design ships and the envisaged conditions of use to secure the safety of people and cargos and to allow the efficient embarkation and disembarkation of passengers and the efficient handling of cargos.

(7) Floating piers shall be stable and safe, and their durability should be ensured because they are used for transporting cargos and passengers. Mooring systems such as mooring ropes and mooring anchors shall have sufficient resistance capacity against working actions.

(8) In setting the cross-sectional dimensions of floating bodies of floating piers, the amount of motions of the floating body, the amount of its tilting, etc. shall be appropriately verified to be within the allowable range in accordance with the envisaged conditions of use as necessary.

(9) **Fig. 6.1.5** shows an example of the procedure of the performance verification of floating piers.

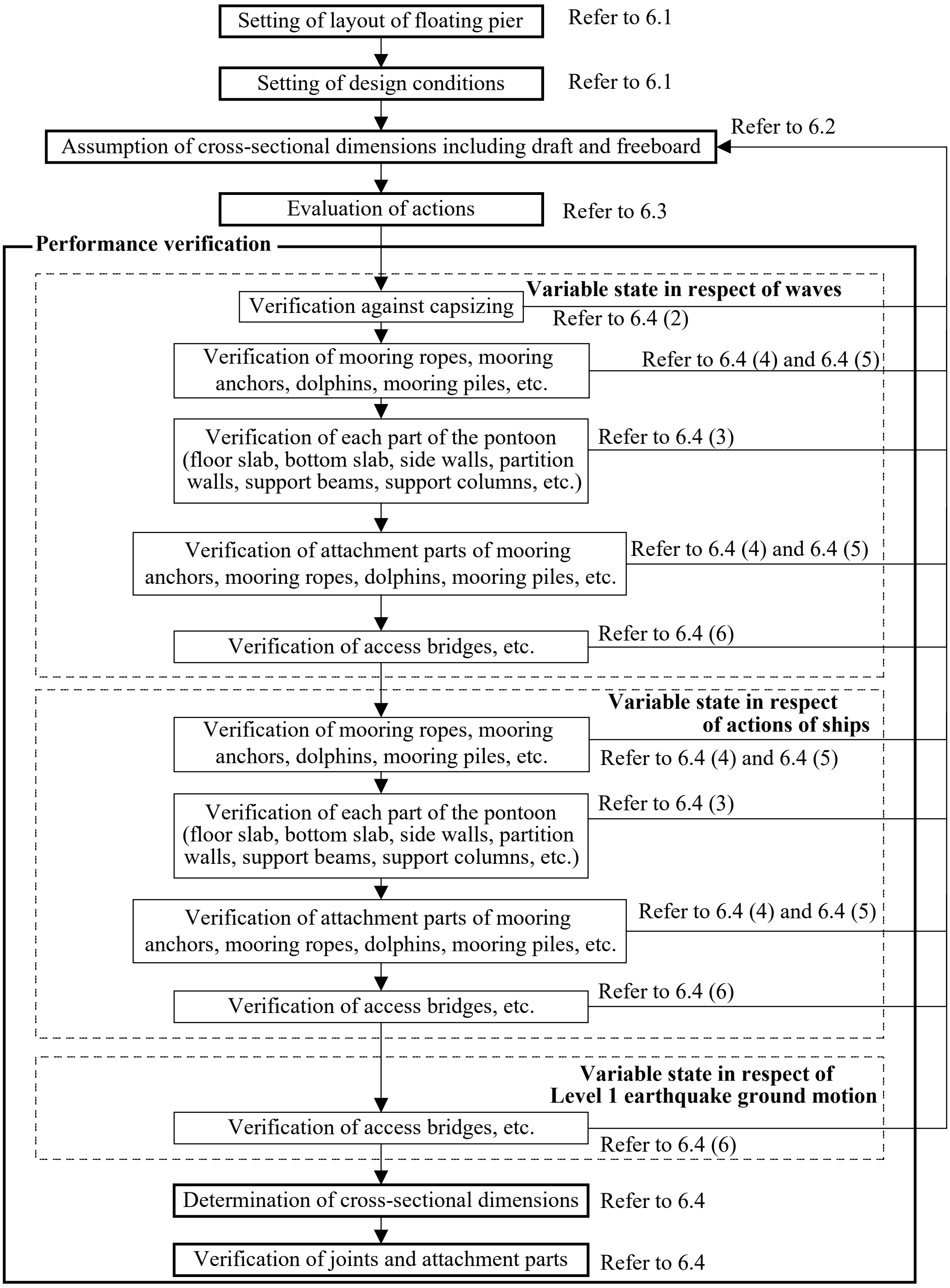


Fig. 6.1.5. Example of Performance Verification Procedure for Floating Piers

(10) The installation locations and arrangement of floating piers shall be determined by considering the types and sizes of design ships, depth of water, water flow, waves, wind, soil of the sea floor, and other factors. The arrangement of floating piers includes jetty type arrangement and parallel-type arrangement.[3] Generally, the jetty type arrangement is preferable due to the lower construction cost and ease of mooring ships. One to three pontoons are usually adopted in Japan when the pontoons are connected. However, recently, some large-scale floating piers have only one pontoon.

(11) When pontoons oscillate subjected to waves and when they roll due to deviations in the vertical directions of the two pontoons, one may hit the other and make a hole on it. To avoid such a problem, two pontoons shall be strongly united, or fenders shall be installed between pontoons. The length of the gang way between the pontoons may be approximately double the interval between the pontoons. Generally, chains are used to unite neighboring pontoons. At this time, the chains are installed onto the chain posts via the fairleaders.

(12) The types of pontoons are broadly classified as follows on the basis of raw materials: pontoons made from reinforced concrete, steel, PC, FRP, and wood and hybrid pontoons. The characteristics of each type are shown below:

① Reinforced concrete pontoons with excellent durability draw deep; therefore, they usually do not oscillate to a great extent. The construction, maintenance, and repair costs are lower than those for steel pontoons, but they are vulnerable to impact, and their imperviousness is rather low. Therefore, to bolster imperviousness, the content of concrete needs to be higher and reliable construction is required. Regarding reinforcing bars, the use of many small-diameter bars is recommended to enhance the resistance to impact.

② Impact-resistant steel pontoons are easier to manufacture and repair, but they corrode. Therefore, their durability is lower than that of reinforced concrete pontoons. However, they draw lighter than reinforced concrete pontoons; therefore, they are not significantly affected by water flow.

③ PC pontoons with excellent imperviousness resist cracking compared with reinforced concrete pontoons, and it is also an advantage that thickness of the members can be thinner.

④ Lightweight FRP pontoons draw light; therefore, they are unstable but are highly durable and easier to install. They are currently used for small-scale floating piers (e.g., marinas).

⑤ The construction cost of wood pontoons is low, but their imperviousness is inferior, and their durability is low because they are vulnerable to decay and insect damage. To secure their imperviousness and antisepticize them, they need to be often pulled up and repaired.

⑥ RC hybrid pontoons refer to steel floating bodies that are surrounded by protective reinforced concrete. The steel members and concrete members work as one body to resist loads. Their imperviousness is high, and they resist corrosion. The weight of this type of a floating body is between that of a reinforced concrete pontoon and that of a steel pontoon.

⑦ For PC hybrid pontoons, PC inner members (support beams and partition walls) are combined with steel materials. The weight of a floating body in this type can be lighter than that of a PC pontoon. Their imperviousness is high, and they resist corrosion.

(13) For the raw materials of pontoons, the **Standard Specifications for Concrete Structures [Design]**,[4] the **Rules for the Survey and Construction of Steel Ships; Guidance for the Survey and Construction of Steel Ships Part Q (Steel Barges)**,[5] and other standards shall be used as a reference.

(14) There are several types of mooring methods such as the chain or wire method, in which mooring ropes are used, and the dolphin-fender method, in which dolphins are used.[1] For small-scale floating piers, a mooring method that uses mooring piles is often used. The method to be applied shall be appropriately selected on the basis of the performance required for floating piers and other factors.

(15) Access bridges are movable bridges. There are two types of such bridges: One bridge has a land side that is secured with hinges, and its other side with rollers is on a pontoon; the other bridge is hung with an adjustment tower.[3] At a place where motions due to waves is minimal, the former type can be used without using adjustment towers. The length and width of an access bridge shall be appropriately determined by considering the conditions of use of the floating pier.

6.2 Setting of Basic Cross-section

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

(2) The length of many pontoons is 20 to 40 m, the width is less than 15 m, and the height is 2 to 4 m. Recently, large-scale floating piers have been constructed.

(3) As standard dimensions of various sections in a pontoon, the length of a single side of a floor slab, side wall, bottom slab, and partition wall is 1 to 3 m. The thickness of the side wall and bottom slab of a reinforced concrete

pontoon is often 15 to 20 cm and that of a floor slab and partition wall is 10 to 20 cm. For steel pontoons, the thickness of them is often 6 to 10 mm. The ratio of the side to the length of each slab shall desirably be a value close to one.

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

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

where

h' : freeboard (m)

d : pontoon height (m)

W_1 : pontoon weight (kN)

γ_W : unit weight of seawater (kN/m^3)

A : horizontal cross-sectional area of the pontoon (m^2)

(5) In the case of a reinforced concrete pontoon, the dimensions shall be determined by considering the imperviousness of the concrete.

(6) To enable the facilities to exert its functions, the mooring method of pontoons of a floating pier shall be appropriately determined on the basis of the scale of the pontoons, water depth at the installation location, soil of the sea floor, and other natural conditions.

(7) Regarding the types of mooring methods, normally a chain method or a wire method is used in fairly deep water. For shallow water depths, an intermediate buoy method, an intermediate sinker method, or a dolphin-fender method are mainly used.[1)] The mooring method shall be selected on the basis of a comparison of the functions and stability of the floating pier and the characteristics of the mooring facilities.

(8) The dimensions and incline of access bridges and gang ways shall be appropriately determined by considering the capacity required from floating piers for transporting passengers and cargos.

(9) For the width and incline of access bridges and gang ways on which vehicles pass, **Part III, Chapter 5, 9.8 Vehicle Loading Facilities** shall be referred to.

(10) The incline of access bridges and gang ways on which passengers pass is often 5% to 20% at the L.W.L.

(11) Regarding the dimensions of access bridges and gang ways, the width is often 2 to 6 m, the span length for access bridges is 10 to 30 m, and the span length for gang ways is 2 to 6 m, according to the examples constructed in the past.

6.3 Actions

(1) Among the actions applied to pontoons, the live load shall be determined on the basis of the type of passengers and cargos that will be transported using pontoons.

(2) When pontoons are subject to a reaction force from the access bridges, they tilt; therefore, ballast is placed as a counterweight in some cases. The weight shall be determined such that it matches the reaction force caused by the self-weight of an access bridge, and it works to make the pontoon horizontal.

(3) The fender reaction force, wave force, current force, etc. do not need to be considered unless necessary. However, when there is an anticipated risk that the pontoon may be subjected to wave actions, the following forces shall be considered: the wave forces exerted upon the stationary pontoon that is assumed to be rigidly fixed in position and the fluid forces due to the motions of the pontoon.[6)] For these forces, **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions** shall be referred to. In this case, the mooring forces should be calculated by considering the motions of the pontoon.

(4) A sidewalk live load of 5.0 kN/m^2 is commonly used for floating piers, which are mainly used for passenger ships. For the live load, the vehicle load shall be taken into account when floating piers are used for ferries and when

vehicles are allowed to get on them. For vehicle loads used for performance verification of floor slabs, the T-load specified in the **Specifications and Commentaries for Highway Bridges, Part I Common**[7] can be generally used.

(5) The fender reaction forces used in the performance verification of mooring ropes, etc. can be calculated by referring to **Part II, Chapter 8, 2.2 Actions Caused by Ship Berthing** and **Part II, Chapter 8, 2.3 Actions Caused by Ship Motions**. Furthermore, for the tractive forces of ships, refer to **Part II, Chapter 8, 2.4 Actions Caused by Traction of Ships** shall be referred to.

(6) The wave forces used in the performance verification of mooring ropes, etc. can be calculated by an appropriate method by referring to **Part II, Chapter 2, 6.4 Wave Force Acting on Structures near the Water Surface** and **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions**. At this time, the drag coefficient for cubes may be used. The area over which the drag force acts can be taken to be the area below the still water surface. The abovementioned wave forces are those that act on a stationary pontoon. However, if the natural period of the motions of the pontoon is close to the period of the waves, resonance may occur, thus causing large forces in the mooring ropes. This point should be carefully considered. In particular, for floating piers located in places where it is envisaged that swells and other long period waves penetrate, it is preferable that a motion analysis of the moored floating bodies be performed using a numerical simulation method.[8]

(7) In the performance verification of mooring ropes, etc., when the mooring systems are not tense, the influence of wave drift force (higher-order component) is large in addition to wave-exciting force; therefore, it should be appropriately evaluated.

(8) The water flow velocity used in the performance verification of mooring ropes, etc. can be determined on the basis of the instructions in **Part II, Chapter 2, 7 Water Currents**, but it is desirable to determine it through actual measurements. Furthermore, the tidal current force can be calculated by referring to **Part II, Chapter 2, 7.2 Fluid Force due to Currents**. The drag coefficient can be calculated in a similar way as that for wave forces.

6.4 Performance Verification

(1) Normally the following items shall be examined for the verification of the stability of floating piers.

① Pontoon stability

② Stability of each part of the pontoon

③ Stability of the mooring system (e.g., mooring ropes, mooring anchors, dolphins, and mooring piles)

④ Stability of access bridges and gang ways

(2) Performance Verification of the Stability of Pontoons

① The structural stability levels required for the pontoons shall be appropriately secured in accordance with the conditions of use and other conditions. In examining the stability of a pontoon, the following requirements must be satisfied:

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

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

② Generally, the height of the water accumulated inside the pontoon by leakage shall be 10% of the height of the pontoon in the examination of pontoon stability. In most cases, the freeboard to be maintained is approximately 0.5 m.

③ When subjected to a uniformly distributed load, the pontoon can be regarded stable if **equation (6.4.1)** is satisfied. **Fig. 6.4.1** illustrates the stability of a pontoon subjected to an eccentric load.

$$\frac{I\gamma_W}{W} - \overline{\mathrm{CG}} > 0 \qquad \textbf{(6.4.1)}$$

where

I : geometrical moment of the inertia of the cross-sectional area at the still water level with respect to the longitudinal axis of the pontoon (m^4)

W : weight of the pontoon and uniformly distributed load (kN)

γ_W : unit weight of seawater (kN/m^3)

C : center of buoyancy of the pontoon

G : center of gravity of the pontoon

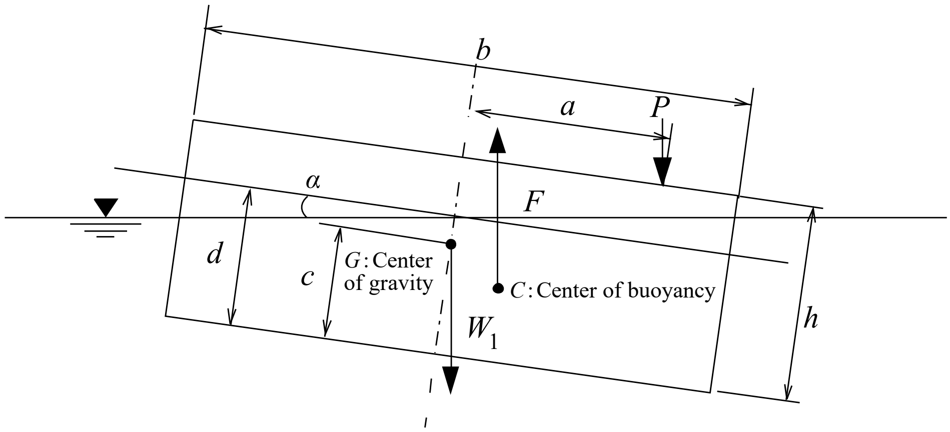


Fig. 6.4.1. Stability of Pontoon Subjected to Eccentric Load

When the pontoon is partially filled with water by leakage, the pontoon can be regarded stable when **equation (6.4.2)** is satisfied. *W*, *I*, C, and G of the equation shall refer to the states for water leakage inside the pontoon.

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

where

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

When subjected to an eccentric load, it shall be checked if the value of tan α obtained by solving **equation (6.4.3)** satisfies **equation (6.4.4)**. α is generally very small; therefore, $\cos^2\alpha$ in **equation (6.4.3)** can be $1-\tan^2\alpha$ approximately.

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

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

where

W_1 : weight of the pontoon (kN)

P : total force of the eccentric load (kN)

b : width of the pontoon (m)

h : height of the pontoon (m)

d : draft of the pontoon when P is applied to the center of the pontoon (m)

c : height of the center of gravity of the pontoon measured from the bottom (m)

a : deviation of P from the center axis of the pontoon (m)

α : inclination angle of the pontoon (°)

④ Regarding concepts on the stability of pontoons by water leakage, the **Theoretical Naval Architect**[9)] can be used as a reference.

(3) Performance Verification of the Stability of Each Part of a Pontoon

① The stresses, etc. generated in the structural parts of the pontoon shall be examined by using an appropriate method selected by considering the usage conditions of the pontoon, external actions on the respective parts, their structural characteristics etc.

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

(a) When only a static load acts on a pontoon

(static load) + (self-weight)

(b) When a live load acts on a pontoon

(live load) + (self-weight)

(c) When the supporting point of an access bridge is set on a pontoon without adjustment tower

(supporting point reaction force of an access bridge) + (self-weight)

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

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

⑤ A partition wall can normally be verified for performance as a slab fixed on four sides against water pressure acting when one compartment has become fully waterlogged.

⑥ The support beams of the floor slab, bottom slab, and side walls and the center support columns can normally be calculated as a rigid frame box against water pressure acting when the maximum load is on the floor slab of the pontoon and the draft of the pontoon is equal to its height. Furthermore, the performance of the center support columns and the support beams of the side walls can be verified as members that receive bending force and compressive force in the axial direction. Support beams can be regarded T-beams, but their performance can be generally verified as rectangle beams.

⑦ The effective spans of slabs shall be intervals between the centers of support beams, floor slabs, side walls, bottom slabs, etc.

⑧ For the bending moment of a four-side fixed slab that receives a uniformly distributed load, [**Reference (Part III), Chapter 4, 2 Numerical Table for Bending Moment of Slab** shall be used. The bending moment of a four-side fixed slab, when it receives trapezoidal water pressure, can be evaluated as follows: the water pressure is divided into a uniformly distributed load of 5.0 kN/m^2 and a triangular load that is zero at the upper end of the pontoon and that increases by 10.0 kN/m^2 for every 1.0 m, and then both loads are added.

⑨ The live load can be regarded a T-load, and performance can be verified by handling the live load as a concentrated load. For the performance verification of slabs, the **Specifications and Commentaries for**

Highway Bridges, Part II Steel Bridges and Steel Members[10] and the **Specifications and Commentaries for Highway Bridges, Part III Concrete Bridges and Concrete Members**[11] shall be referred to.

⑩ The supporting point reaction force of an access bridge can be acted at the center of the slab as a concentrated load. Furthermore, the weight of a steel plate to be constructed under the supporting points of an access bridge shall be considered as self-weight.

⑪ When the wave actions are considered, the calculations of section forces can be made using Müller's equation[12] and other rules. When it is necessary to consider the motions of the floating body, wave parameters, the effect of water depth, etc., the method with cross-sectional division by Ueda et al. [6) 13) 14)] can be used.

⑫ The imperviousness of concrete must be fully ensured. Surface coating with epoxy resin and polyurethane resin can be applied as measures for imperviousness.

(4) Performance Verification of the Stability of Mooring Ropes and Other Mooring Equipment

① The structure of mooring ropes shall be examined by using an appropriate method in such a way that the ropes can hold securely a pontoon in position under the action of whichever force is the largest among the fender reaction force generated during berthing, the tractive force of the ship, and the wave force, with the addition of the tidal current force to each of the aforementioned forces.

② Fundamentally, the chain method is described here because it is assumed to be the most applied. For other mooring methods, **Reference 1)** shall be referred to.

③ Chains are usually installed onto the pontoon's chain posts at the four corners through the chain holes, and the pontoon is secured to the sea floor with mooring anchors.

④ The chains are normally crossed under a pontoon as shown in **Fig. 6.1.1** such that they do not hinder ships from berthing. However, the chains may rub against each other and may abrade depending on the crossing way; therefore, attention is required.

⑤ The length of a mooring rope is usually five times the water depth plus the tidal range. When a chain is stretched, the following points shall be considered:

(a) The chain shall not be overstretched during high tide because overstretching can exert excessive tension force on the chain.

(b) There shall be no interference with ship berthing during high tide.

(c) Sufficient anchor holding power shall be ensured for mooring anchors during high tide.

(d) The amount of horizontal movement of the pontoon during low tide shall be small.

⑥ The anchor holding power of steel mooring anchors is significantly reduced when the angle between the chain at the attachment part and the horizontal surface is 3° or higher.

⑦ There are chains with studs and without studs. Appropriate chains shall be selected for floating piers. For example, **Flash Butt Welded Anchor Chains (JIS F 3303)** can be used.

⑧ The diameters of chains shall be determined such that they will not break owing to the actions specified in **Part III, Chapter 5, 6.3 Actions** at high tide. The allowable tension of chains at this time can be one-third of the breaking test load specified in **JIS F 3303** mentioned above.[1] The weight of chains can be the minimum weight specified in **JIS F 3303** mentioned above.

⑨ In the determination of the diameter of the chain, careful consideration shall be given to the abrasion, corrosion, and biofouling of the chain. Furthermore, appropriate maintenance work shall be performed on the chain, including periodical checks and replacement as necessary.

⑩ When determining the chain diameter with a numerical simulation of motions, the characteristics of the displacement-restoration force relationship of the mooring system shall be determined using an appropriate method such as catenary theory, etc.[15]

⑪ The maximum tension acting on each chain is ideally calculated by using dynamic analysis on the chain and the pontoon, but static analysis may also be used. A chain can normally be verified for performance on the condition that only one chain is assumed to resist all external actions as shown in **Fig. 6.4.2**.

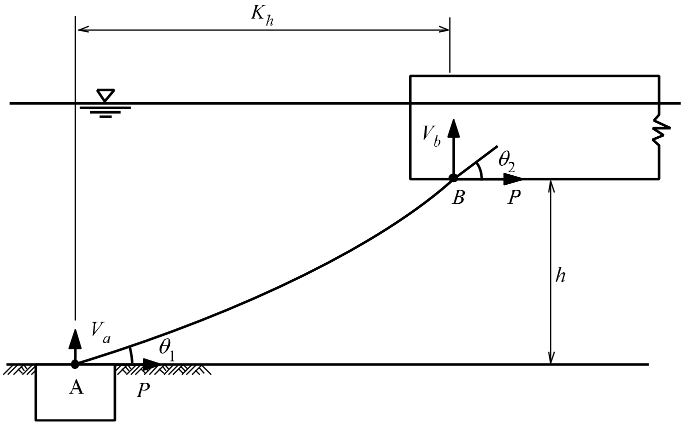


Fig. 6.4.2. Performance Verification of Mooring Rope

Assuming that the chain forms a catenary line, the maximum tension acting on the chain is given by **equation (6.4.5)**.

$$T = P \sec \theta_2 \tag{6.4.5}$$

where

T : maximum tension acting on the chain (kN)

P : horizontal external action (kN)

θ_2 : angle that the chain makes with the horizontal plane at the attachment point between the chain and the pontoon (°)

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

$$V_a = P \tan \theta_1 \tag{6.4.6}$$

where

V_a : vertical force acting on the mooring anchor (kN)

θ_1 : angle that the chain makes with the horizontal plane at the attachment point between the mooring anchor and the chain (°)

The vertical force acting on the attachment point between the chain and the pontoon can be expressed by **equation (6.4.7)**.

$$V_b = P \tan \theta_2 \tag{6.4.7}$$

where

V_b : vertical force acting on the attachment point between the chain and the pontoon (kN)

Angles θ_1 and θ_2 can be calculated using **equation (6.4.8)** by assuming a chain length l and a chain weight per unit length w.

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

where

l : length of the chain (m)

h : water depth under the bottom of the pontoon (m)

w : weight per unit length of the chain in water (kN/m)

The horizontal distance between a mooring anchor and the pontoon can be expressed by **equation (6.4.9)** when a horizontal force acts on the pontoon. By using this equation, the amount of the horizontal movement of the pontoon from its stationary position under no horizontal force can be calculated.

$$K_h = \frac{P}{w}\left\{\sinh^{-1}\left(\tan\theta_2\right) - \sinh^{-1}\left(\tan\theta_1\right)\right\} \quad \textbf{(6.4.9)}$$

where

K_h : horizontal distance between the mooring anchor and the attachment point between the pontoon and the chain (m)

Considering that the catenary line of the chain with normal diameter can be approximately represented with a straight line, it can be assumed in **equations (6.4.5)** to **(6.4.9)** that $\theta_2 = \theta_1 = \sin^{-1}(h/l)$, and K_h can be approximately expressed by : $K_h = \sqrt{l^2 - h^2}$

(5) Performance Verification of the Stability of Mooring Anchors

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

② For the performance verification of the stability of mooring anchors, **equation (6.4.10)** can be used. Furthermore, the adjustment factor can be taken to be an appropriate value equal to or greater than 1.2.

$$\left.\begin{aligned} R_h &\geq mP \\ R_v &\geq mV_a \end{aligned}\right\} \quad \textbf{(6.4.10)}$$

where

R_h : horizontal resistance force of the mooring anchor (kN)

R_v : vertical resistance force of the mooring anchor (kN)

P : horizontal force acting on the mooring anchor (kN)

V_a : vertical force acting on the mooring anchor (kN)

m : adjustment factor

$V_a = P\tan\theta_1$ can be used for the calculation.

③ The following forces are normally considered the resistance forces of a mooring anchor, but in-situ stability tests are preferred for a mooring anchor:

(a) In the case of concrete block:

1) For clay ground:

- Horizontal resistance force R_h: cohesion of the surfaces of the bottom and sides, the difference between the passive and active earth pressures
- Vertical resistance force R_v: weight in water, effective overburden weight in water

2) For sand ground:

- Horizontal resistance force R_h: bottom friction force, the difference between the passive and active earth pressures
- Vertical resistance force R_v: weight in water, effective overburden weight in water

The vertical force used in the calculation of the bottom friction force is the difference between the weight of the block in water and the vertical component of the chain tension acting on the block.

(b) In the case of steel mooring anchor:

- Horizontal resistance force R_h: holding power
- Vertical resistance force R_v: weight in water

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

$$\left.\begin{array}{ll} \text{On soft mud:} & T_A = 17W_A^{2/3} \\ \text{On hard mud:} & T_A = 10W_A^{2/3} \\ \text{On sand:} & T_A = 3W_A \\ \text{On flat rock:} & T_A = 0.4W_A \end{array}\right\} \qquad \textbf{(6.4.11)}$$

where

T_A : holding power of the mooring anchor (kN)

W_A : weight of the mooring anchor in water (kN)

④ Generally, concrete blocks are used as mooring anchors. Steel mooring anchors are often used for sandy soil.

⑤ When concrete blocks are used as mooring anchors, they should be embedded under the sea floor. The weight of many concrete blocks is approximately 150 to 700 kN. Concrete blocks that were reinforced with additional bars were used in some cases.

⑥ When a rectangular solid anchor block is deeply embedded in cohesive soil, Hansen obtained **equation (6.4.12)** for the horizontal resistance force by assuming the slip surface around the block. [16)]

$$P = 11.4ch \qquad \textbf{(6.4.12)}$$

where

P : resistance force of the block per unit width (kN/m)

c : cohesion of the cohesive soil (kN/m^2)

h : height of the block (m)

Mackenzie experimentally obtained **equation (6.4.13)** for blocks embedded to a depth of 12 times or more the height of the block. [16)]

$$P = 8.5ch \qquad \textbf{(6.4.13)}$$

⑦ For the mooring anchors of concrete blocks, anchor rings are generally installed as shown in **Fig. 6.4.3**. For the performance verification of anchor rings, **Part III, Chapter 2, 2 Structural Members** shall be referred to.

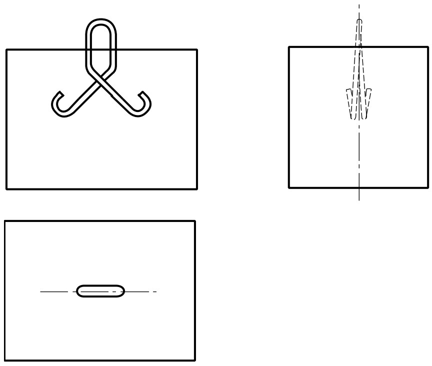


Fig. 6.4.3. Example of Concrete Mooring Anchor

(6) Performance Verification of the Stability of Access Bridges and Gang Ways

① The performance of access bridges and gang ways can be verified by referring to the **Specifications and Commentaries for Highway Bridges, Part II Steel Bridges and Steel Members**[10] and the **Technical Standard and Commentary of Grade Separation Facilities for Pedestrians**.[17]

② In the performance verification of access bridges and gang ways, attention shall be desirably paid so that elderly people and physically impaired persons can safely move on wheelchairs and other similar equipment. Furthermore, for floating piers on which low-floor vehicles may pass, sufficient separation distance shall be provided such that the floors of vehicles do not come into contact with the access bridges and gang ways.

③ Generally, steel access bridges are used for connections to the land. Pony trusses or plate girders are often used. Gang ways between pontoons are often steel-plate girders, I-beams, pony trusses, or slabs.

④ For vehicle loads, the **Specifications and Commentaries for Highway Bridges, Part I Common**[7] shall be referred to.

⑤ As bearings of access bridges connecting to the land, when the pontoons do not oscillate considerably, hinges are used on the land side, roller bearings are used on the pontoon side, or adjustment towers are used for hanging. For the pontoons that greatly oscillate, access bridges are simply placed between steel protection plates installed onto the pontoons and the land coasts, and chains are normally used as anchors for the pontoons and the land coasts to prevent the access bridges from falling down. To improve passage from the access bridges to the pontoons, apron plates are installed at the ends of the access bridges on the pontoon side when necessary. Steel plates are usually installed for protection onto pontoon floor slabs with which roller bearings come into contact. Furthermore, as bearings of gang ways between pontoons, hinges are used on one side, and roller bearings are used on the other side in most cases. There were some cases in which the hinges broke because forces acted on the access bridges within the horizontal planes owing to the motions of the pontoons; therefore, attention is required.

⑥ For access bridges connecting to the land, adjustment towers are installed on the access bridges on the pontoon side in some cases to reduce the supporting point reaction force acting on the pontoons and to wind up the access bridges in stormy weather. There are some types of adjustment towers: one is an adjustment tower that is manually modified on the basis of tide level at all times; another is an adjustment tower for which the most weight of an access bridge is hung with the counterweights of the adjustment tower, and a part is placed on the pontoon to allow the bridge to automatically move up and down on the basis of tide level. Many adjustment towers are made from reinforced concrete and steel frames. An adjustment tower has steel pulleys, counterweights, and hanging materials. **Fig. 6.4.4** illustrates an example of an adjustment tower. For adjustment towers, the stability required to satisfy the functions against the reaction force of access bridges and the actions of earthquake ground motion shall be secured.

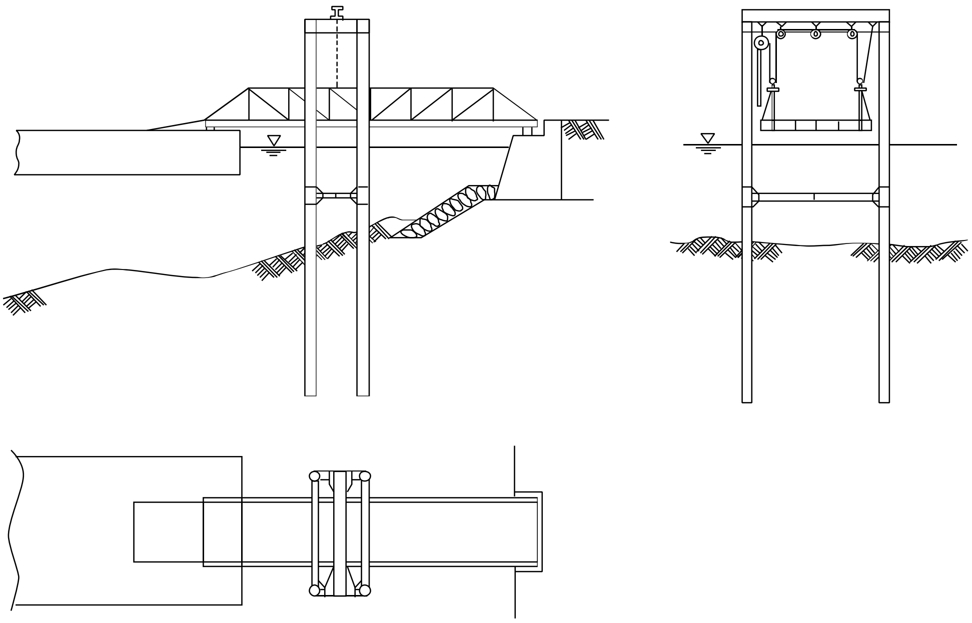


Fig. 6.4.4 Example of Adjustment Tower

6.5 Structural Specifications

Handrails shall be installed onto access bridges and gang ways as safety facilities. Curbing shall be installed on them as necessary.

[References]

1) Coastal Development Institute of Technology: Technical Manual for Floating Structure, 1991. (in Japanese)
2) Association for Innovative Technology on Fishing Ports and Grounds: Manual for Design and Construction of Floating Piers (draft), p.79, 2015. (in Japanese)
3) Yonekawa, M.: Design and Calculation Examples of Port Facilities (Enlarged and Revised Edition), Kazama Publishing, 1983. (in Japanese)
4) JSCE: Standard Specifications for Concrete Structures [Design], 2012. (in Japanese)
5) Nippon Kaiji Kyokai (ClassNK): Rules for the Survey and Construction of Steel Ships; Guidance for the Survey and Construction of Steel Ships Part Q (Steel Barges), 2017. (in Japanese)
6) Ueda, S., S. Shiraishi and K. Kai: Calculation Method of Shear Force and Bending Moment Induced on Pontoon Type Floating Structures in Random Sea, Technical Note of PHRI, No.505, 1984. (in Japanese)
7) Japan Road Association: Specifications and Commentaries for Highway Bridges, Part I Common, 2017. (in Japanese)
8) Ueda, S.: Analysis Method of Ship Motions Moored to Quay Walls and the Applications, Technical Note of PHRI, No.504, 1984. (in Japanese)
9) Oogushi, M: Theoretical Naval Architecture (First Volume) Revised Edition, Kaibundo Publishing, 1993. (in Japanese)
10) Japan Road Association: Specifications and Commentaries for Highway Bridges, Part II Steel Bridges and Steel Members, 2017. (in Japanese)
11) Japan Road Association: Specifications and Commentaries for Highway Bridges, Part III Concrete Bridges and Concrete Members, 2017. (in Japanese)
12) Müller, J.: Structural Considerations and Configurations II, Seminar on Concrete Ships and Vessels, University of California Extension, Berkeley, 1975.
13) Ueda, S., S. Shiraishi and T. Ishisaki: Calculation Method of Forces and Moments Induced on Pontoon Type Floating Structures in Waves, Rept. of PHRI, Vol. 31, No.2, 1992. (in Japanese)

14) Ueda, S., S. Shiraishi and T. Ishisaki: Example of Calculation of Forces and Moments Induced on Pontoon Type Floating Structures and Figures and Tables of Radiation Forces, Technical Note of PHRI, No.731, 1992. (in Japanese)

15) Ueda, S. and S. Shiraishi: Determination of Optimum Mooring Chain and Design Charts Using Catenary Theory, Technical Note of PHRI, No.379, 1981. (in Japanese)

16) Leonards, G.A.: Foundation Engineering, McGraw Hill Book Co., p.467, 1962.

17) Japan Road Association: Technical Standard and Commentary of Grade Separation Facilities for Pedestrians, 1979. (in Japanese)

7 Shallow Draft Wharves

(English translation of this section from Japanese version is currently being prepared.)

7.1 Common for Shallow Draft Wharves

(English translation of this section from Japanese version is currently being prepared.)

7.2 Actions

(English translation of this section from Japanese version is currently being prepared.)

7.3 Performance Verification

(English translation of this section from Japanese version is currently being prepared.)

8 Boat Lift Yards

[Ministerial Ordinance] (Performance Requirements for Boat Lift Yards)

Article 32

The performance requirements for boat lift yards shall be as prescribed respectively in the following items in consideration of the structural type:

(1) The requirements specified by the Minister of Land, Infrastructure, Transport and Tourism shall be satisfied so as to enable the safe and smooth lifting and launching of boats.

(2) Damage to boat lift yards, etc. due to self-weight, earth pressure, water pressure, variable waves, berthing and traction of boats, Level 1 earthquake ground motions, surcharge loads, etc. shall not impair the function of the boat lift yards and shall not adversely affect the continuous use of the boat lift yard.

[Public Notice] (Performance Criteria for Boat Lift Yards)

Article 58

1 The performance criteria for boat lift yards shall be as prescribed respectively in the following items:

(1) The boat lift yard shall have the necessary water depth and length in consideration of the dimensions of the design ships.

(2) The boat lift yard shall have the necessary crown height in consideration of the tidal range, the dimensions of the design ships and the usage conditions.

(3) The boat lift yard shall have the necessary ancillary equipment in consideration of the usage conditions.

2 The provisions of Article 49 through Article 52 apply mutatis mutandis to the performance criteria of the front wall portion of the boat lift yard in consideration of the structural type.

3 The performance criteria for the pavement of the boat lift yard shall be as prescribed respectively in the following items:

(1) The pavement of the boat lift yard shall have the dimensions necessary for enabling the safe and smooth handling of boats.

(2) The risk of impairing the integrity of the pavement under the variable situation, in which the dominating action is the surcharge load, shall be equal to or less than the threshold level.

(3) The risk of impairing the integrity of the pavement of the slip way under the variable situation, in which the dominating actions are water pressure and variable waves, shall be equal to or less than the threshold level.

[Interpretation]

11. Mooring facilities

(12) Performance Criteria for Boat Lift Yards (Article 32 of the Ministerial Ordinance and the interpretation related to Article 58 of the Public Notice)

① The provisions of Article 49 (performance criteria of gravity-type wharfs) through Article 52 (performance criteria of cell-type wharfs) in the Reference Public Notice and their interpretations shall be applied with modification as necessary to the performance criteria and interpretations for the front wall portions of boat lift yards in consideration of the structural type. Necessary performance verification items for the front wall portions of boat lift yards shall appropriately be selected from those defined in the articles.

② The pavements of boat lift yards shall have serviceability as their performance requirement in variable situations where the dominating actions are surcharges. **Attached Table 11–29** shows performance verification items and standard indexes to determine limit values for such actions. For the performance verification of the pavements of boat lift yards, standard indexes to determine limit values for the integrity shall be appropriately set based on the material quality and other factors.

Attached Table 11–29 Performance Verification Items and Standard Indexes to Determine Limit Values for the Pavements of Boat Lift Yards in Various Design Situations (excluding accidental situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
32	1	2	58	3	2	Serviceability	Variable	Repeated surcharges	–	Soundness of the pavement	–
					3			Water pressure [Variable waves]	–	Soundness of the pavement of the slip way	–

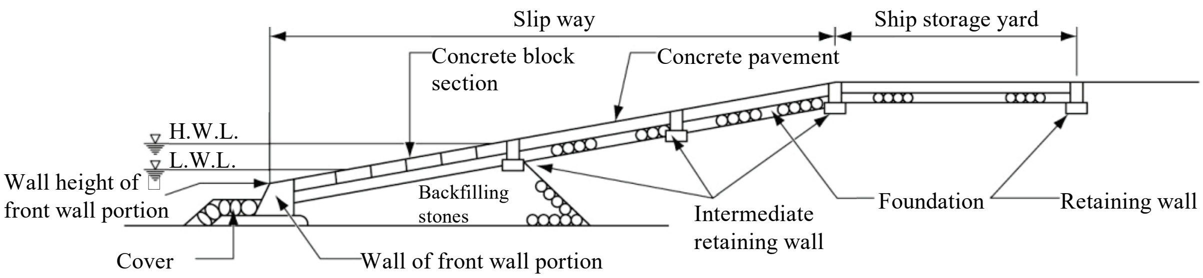
Note: The descriptions [xx] under Dominating action indicate that such dominating action is used as a substitute for the design situation.

8.1 Fundamentals of Performance Verification

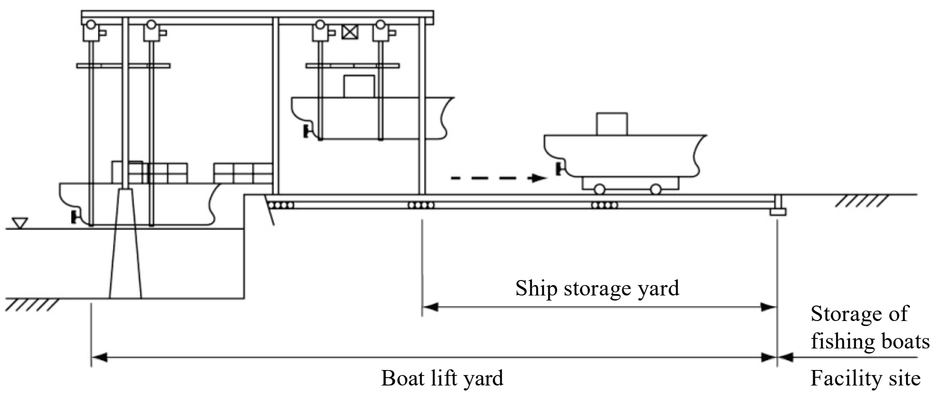
(1) A boat lift yard is a facility used to retrieve ships to the land and launch to the sea for such purposes as repair, refuge from storm waves and storm surges, and land storage of ships during winter.

(2) In many cases, rails or cradles are employed in the retrieving and launching of ships of 30 tons or larger in gross tonnage, but the provisions in this section can be applied to the performance verifications of the facilities used to lift and launch ships smaller than 30 tons in gross tonnage directly over the slope of slip way.

(3) The structure of boat lift yards is broadly divided into the pull-up type (slip way type) and lifting type. **Fig. 8.1.1** shows main components of both structural types.



(a) Pull-up type (slip way type)



(b) Lifting type

Fig. 8.1.1 Boat Lift Yard

(4) As fishing boats have been becoming more energy efficient, small ships have also been following this trend. Ships smaller than 30 tons in gross tonnage require rail-type slip ways in some cases. Rails are required because larger keels are used because of larger propellers and such keels get out from the bottoms of ships. This makes it impossible to directly pull up such ships onto a slope and lower them.

(5) In addition to improved energy efficiency of small ships, the number of lift-type docking and undocking facilities has also been increasing to make docking and undocking quick and to ensure safety.

8.2 Performance Verification

For performance verification of boat lift yards, **Design Guidelines for Fishing Port and Fishing Ground Facilities, Part 6 Mooring Facilities**[1] can be referred to.

8.3 Location Selection of Boat Lift Yard

(1) Location of boat lift yards needs to be determined in such a way that the following requirements are satisfied:

① The front water area is calm.

② The front water area is free from siltation or scouring.

③ Navigation and anchorage of other ships are not hindered.

④ There is an adequate space in the background for the work for ship lifting and launching as well as for ship storage.

(2) A boat lift yard is a slope, so waves easily go up it and they hinder the usage and moreover may cause disaster, so a calm place shall be selected to install a yard.

(3) A place for which the water area at the front would be easily buried because of littoral drift or river sediment load or it would be easily scoured because of water flow or waves requires maintenance and repair work, so such place shall be avoided as much as possible.

(4) Usually, on the back of a boat lift yard, hoists, rails, repair facilities, and other equipment are installed and fishing equipment is temporarily placed. Also, vehicles drive in there or the space is used for other purposes, so a sufficiently large site shall desirably be secured.

(5) The water area at the front shall be sufficiently large such that retrieving and launching of ships and installed rails and sliding way do not hinter the sailing and anchoring of other ships.

(6) At a place where waves going up the slip way in stormy weather may flow into the space on the back, scouring prevention work is required on the back of the slip way.

8.4 Dimensions of Each Part

8.4.1 Requirements for Serviceability

(1) Water depth and length

(a) Length

In setting the length of slip ways for the performance verification, the dimensions of the design ships shall be appropriately considered.

(b) Water depth

In setting the water depth of slip ways for the performance verification of boat lift yards, the dimensions of the design ships and the envisaged conditions of use of the facility shall be appropriately considered.

(2) Crown height

In setting the crown height of slip ways for the performance verification of boat lift yards, the dimensions of the design ships and the envisaged conditions of use of the facility shall be appropriately considered to enable the slip ways to be safely and efficiently used.

(3) Ancillary facilities

In the performance verification of boat lift yards, appropriate consideration is required for ancillary facilities to enable the boat lift yards to be safely and efficiently used. The provisions of **Reference Ministerial Ordinance, Article 33** (Performance Required for Facilities Incidental to Mooring Facilities) shall be applied with modification as necessary to the performance required for ancillary facilities. The settings in **Reference Public Notice, Article 60** to **Article 74** shall be applied with modification as necessary to the performance criteria based on the type of an ancillary facility.

(4) Others

① **Extension of slip ways**

In the performance verification of boat lift yards having slip ways, the extension of a slip way shall be appropriately set considering the dimensions of the design ships and the envisaged conditions of the use of the facility such that it does not hinder the use by design ships.

② **Area of the space on the back of a slip way**

In the performance verification of boat lift yards having slip ways, the area of the space on the back of the slip way shall be appropriately set considering the dimensions of the design ships and the envisaged conditions of the use of the facility such that it does not hinder the use by design ships.

③ **Slope angle of the slip way**

In the performance verification of boat lift yards having slip ways, the slope angle of the slip way shall be appropriately set considering the dimensions and shape of the design ships, the ground conditions, the tidal range, and the envisaged conditions of the use of the facility in order to enable smooth retrieving and launching of ships.

④ **Area of the anchorage at the front**

In the performance verification of boat lift yards, the area of the anchorage at the front shall be appropriately set considering the dimensions of the design ships and the envisaged conditions of the use of the facility so that design ships can be safely retrieved and launched and such that it does not hinder the sailing of other ships.

8.4.2 Height of Each Part

(1) It is preferable that the crown height of the front wall of the slip way section be located at a level lower than the mean monthly lowest water level (L.W.L.) by the draft of the design ships. This requirement indicates that it is necessary to lift ships even at the low water of neaps. The draft of the ship should be the light draft for the case of repair, refuge, and wintertime storage and should be the full-load draft for the case of lifting small fishing boats filled with catches. For boat lift yards that are to be constructed in the areas where tidal ranges are small or for the boat lift yards that are to be used even at the low-water springs during high waves, it is possible to lower the crown height of the front wall further.

(2) The crown height of the ship storage yard can be determined by applying **Part III, Chapter 5, 2.1.1 Dimensions of Quaywalls**. However, when the ship storage area is located adjacent to a quaywall, the crown height of the ship storage area can be set equal to the crown height of the quaywall to facilitate ease of use. In cases where waves are high in the water area in front of the boat lift yard, consideration of the wave runup height is preferable.

(3) It is preferable not to change the gradient of the slip ways considering the convenience of retrieving and launching of ships.

(4) If providing a point at which the gradient changes on the slip ways is unavoidable, due to the deep depth of water or constraint of available ground area, it is preferable that the position of the point of gradient change be set considering the heights of the following:

① **When the slip way consists of two different surfaces**

Near M.S.L. - H.W.L.

② **When the slip way consists of three different surfaces**

First point : Near L.W.L.

Second point: Near H.W.L.

(5) When waves are high in the water area in front of the boat lift yard, the crown height of the ship storage yard shall preferably be determined considering the wave runup height. In addition, in determination of the crown height of the ship storage yard, influence of abnormal tide levels and ground subsidence shall be appropriately considered.

(6) The submerged section of a slip way is usually covered with concrete blocks as the structure in many cases. In such a case, the height of the boundary between the cast-in-place concrete and concrete block section can be near M.S.L.

(7) For the relationship between the wave height at the front and wave runup height, **Part II, Chapter 2, 4 Waves** can be referred to.

8.4.3 Extension of Boat Lift Yards and Areas on the Back

(1) The extension of the slip way of a boat lift yard shall be calculated from the use of the yard and shall desirably be determined considering the facility layout of the entire port.

(2) The area of space on the back is to place retrieved ships, and usually, it is a flat space area. However, when the crown height of the ship storage yard is high, a part at the top of the slope is sometimes included. As the length of this section of the slip way, it is desirable to add approximately 5 m to the total length of design ships.

(3) Retrieved ships shall desirably be arranged at intervals of approximately 2 m in the longitudinal direction and approximately 1 m in the horizontal direction.

8.4.4 Front Water Depth

(1) The depth of water in front of the slip way may be determined referring to the sum of the draft of the design ship and a margin of 0.5 m.

(2) The required water depth at the front varies depending on the crown height of the wall of the front wall portion and ship retrieving and launching procedures. Generally, when wire ropes are used to launch a ship slowly, the ship does not draw deeper than the maximum draft. When a ship is slid on the slope to launch it, it may draw deeper than the maximum draft, so attention is required.

8.4.5 Gradient of Slip way

(1) The gradient of the slip way shall be determined appropriately in consideration of the shape of the design ships, the characteristics of foundation, and the tidal range, so that the lifting and launching of ships can be performed smoothly.

(2) When the slip way is to be utilized by small ships, it is preferable to have a slope with a single gradient. Single-gradient slopes are frequently used in slip ways for human power-based ship lifting in shallow waters. For this type of slip way, a slope inclination of 1:6 to 1:12 may be used as a reference.

(3) When the water in front of the slip way is deep or the area of the construction site is limited, the slip way may be built with two or more gradients. When this is the case, a two-gradient slip way may be adopted when the crown height of the front wall is about -2.0 m, and a three-gradient slip way may be adopted when the crown height of the front wall is lower than -2.0 m. The following values may be used as reference gradients:

① **When the slip way consists of two different surfaces:**

Front slope: 1:6 to 1:8

Rear slope: 1:8 to 1:12

② **When the slip way consists of three different surfaces:**

Front slope: an inclination steeper than 1:6

Central slope: 1:6 to 1:8

Rear slope: 1:8 to 1:12

(4) The submerged section can be a steep slope. However, seaweed gets on the low-water section and that makes the slope slippery when a ship is retrieved, so a gentle slope is desirable. If a steep slope is unavoidably used, footholds or other similar materials shall desirably be installed on the sides of the slip way to prevent slippage.

(5) At a section at which the gradient changes, larger force is required to pull up a ship, so the gradient shall not desirably be changed much. If the gradient is unavoidably changed largely, the step shall be smoothened using a curve with the large radius of curvature.

(6) For docking and undocking of ships with the total tonnage of 30 tons or more, rails are often used. In such a case, the slip way often consists of two different surfaces or three different surfaces. The gradient of the rear slope is gentler than 1:12 in some cases.

8.4.6 Area of Front Basin

(1) The basin in front of a boat lift yard shall have an appropriate area that allows for efficient operation of ship retrieving and launching without damage to the ships and safe and efficient navigation of nearby ships.

(2) When the ship is launched to the sea by sliding over the slip way, the ship runs over a certain distance after touching the water with the speed gained during the launch. This distance is more than about 5 times of the ship's length overall, although it varies depending on the gradient of slope, slip way friction, and launching distance. However, because the ship attains its maneuverability after moving a distance about 4 to 6 times of its length, it is sufficient to secure a distance about 5 times of the ship's length overall from the waterfront line of the slip way to the other end of the basin. When strong tidal currents exist, it is preferable to add an appropriate margin.

(3) When the ship is launched to the sea gently by wire ropes, a distance of about 3 times of the ship's length overall will suffice to secure the required width of water area.

8.5 Walls and Pavements of Front Wall Portions

8.5.1 Walls of Front Wall Portions

(1) The structure of walls of the front wall portions of boat lift yards shall be appropriately set based on the main dimensions of ships that use the yards, the crown height of the walls of the front wall portions, the methods to pull up such ships, and other factors.

(2) For the performance verification of the walls of the front wall portions of boat lift yards, performance verification for similar structures can be referred to, based on the structure of the walls.

(3) There are several types of wall structures for front wall portions such as blocks, cast-in-place concrete, and sheet piles.

(4) When the crown height of the wall of a front wall portion is high, the weight of a ship may concentratedly apply to the wall of the front wall portion, so particular attention is required for the bearing power of the foundation.

(5) If the foundation is scoured by incoming waves from the front wall portion or the return flow of waves that went up the slope, which may break not only the wall of the front wall portion but also the slope. Therefore, at a place with great waves, sufficient foot protection and covering are desired for the wall of the front wall portion.

8.5.2 Pavements

(1) Most pavements are cement concrete. The thickness of concrete slabs is 20 to 35 cm for cast-in-place concrete, and the intervals between joints are approximately 5 to 10 m. For precast concrete blocks, the size is 2 × 2 m and the thickness is approximately 30 cm in many cases. However, when the wave height is high or in recovery from disaster, **Fig. 8.1.2** can be used to determine the thickness of blocks.[2)] The roadbeds shall desirably be sufficiently compacted to prevent differential settlement. Usually, the thickness is approximately 30 cm.

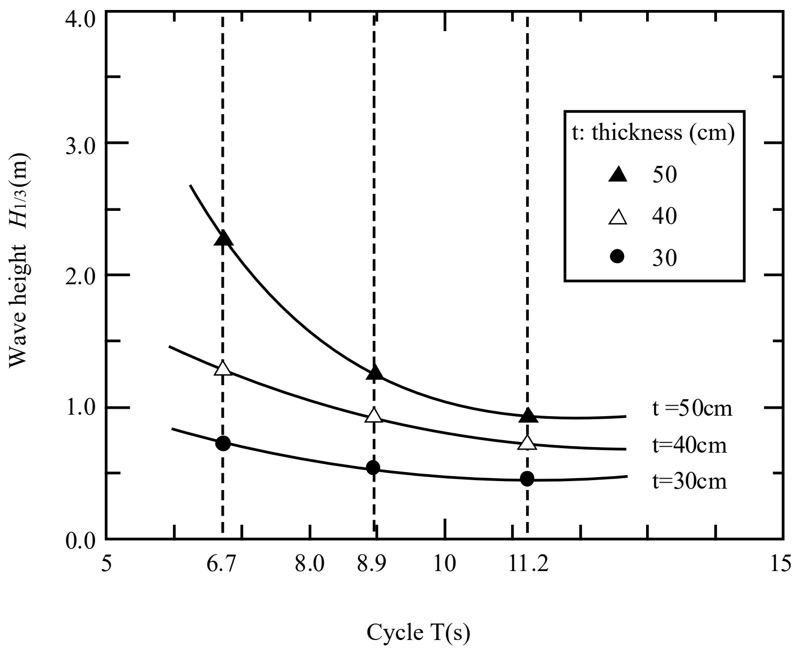


Fig. 8.1.2 Required Thickness of Pavements based on the Cycle and Wave Height t[2)]

(2) The raw materials of the foundation may be drawn out from the joints of the pavement, which may cause differential settlement and may break the pavement. Therefore, pavements shall be constructed with less joints as the structure. The materials are often drawn out at a slope where waves go up, in particular, so cast-in-place pavements shall desirably be constructed as much as possible.

(3) Cast-in-place pavements are difficult to construct under water, so precast concrete blocks are laid out. Such a structure shall desirably prevent sand from flowing out by using half lap joints as joints and by putting asphalt in between the joints. In addition, sand invasion prevention plates shall be used for joints between cast-in-place pavements and precast concrete blocks.

(4) The joints of pavements that may be easily broken by waves are connected with tie bars, or intermediate retaining walls are installed and the edges are cut in some cases. In addition, intermediate retaining walls are often installed at the boundaries between the concrete block section and concrete section. At the ends of pavements, retaining walls shall desirably be installed because if the site on the back is scoured, the foundation of the pavement may flow out.

(5) A point at which the gradient changes easily breaks due to concentration of the weight of a ship or other factors, so intermediate retaining walls, piles, and other similar materials are sometimes used for reinforcement.

(6) If there is a risk of a slope settling because the ground is soft, attention shall be paid to prevent differential settlement through rolling compaction of the foundation, soil improvement, and other measures.

(7) In performance verification of pavements, the necessary stability shall be secured against actions from ships and other elements.

[References]

1) Fisheries Agency: Reference for the Design of Fisheries Facilities, 2015 (in Japanese), http://www.jfa.maff.go.jp/j/gyoko_gyozyo/g_thema/sub52.html

2) Kimura., K.: Design method of plastering blocks for slip way, Journal of Public Works Research Institute (PWRI), Hokkaido Regional Development Bureau, No. 369、1984

9 Ancillary Equipment of Mooring Facilities

9.1 Mooring Posts, Bollards and Mooring Rings

[Public Notice] (Performance Criteria of Mooring Posts, Bollards and Mooring Rings)

Article 60

The performance criteria of mooring posts, bollards and mooring rings shall be as prescribed respectively in the following items:

(1) The mooring posts, bollards and mooring rings shall be located appropriately so as to enable the safe and smooth mooring of ships and cargo handling operations in consideration of the positions of the mooring ropes for the ships using the mooring facilities.

(2) The risk of impairing the integrity of the members of mooring posts, bollards and mooring rings and losing their structural stability shall be equal to or less than the threshold level under the variable situation in which the dominating action is the traction by ships.

[Interpretation]

11. Mooring Facilities

(13) Performance Criteria of Mooring Posts, Bollards and Mooring Rings (Article 33 of the Ministerial Ordinance, and the interpretation related to Article 60 of the Public Notice**)**

Serviceability shall be the performance requirement of mooring posts, bollards and mooring rings under the variable situation in which the dominating action is traction by ships. The performance verification items and standard indexes to determine the limit values for such actions shall be as shown in **Attached Table 11-30**. For the performance verification of the members of mooring posts, bollards and mooring rings, the standard indexes for determining the limit values for the soundness shall be appropriately set according to the kind of materials.

Attached Table 11-30 Performance Verification Items and Standard Indexes for Determination of Limit Values of Mooring Posts, Bollards and Mooring Rings in Each Design State (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
33	1	2	60	–	2	Serviceability	Variable	Traction by ships	–	Soundness of members of mooring posts, bollards and mooring rings	–
								Traction by ships	Self-weight	Stability of structures of mooring posts, bollards and mooring rings (sliding of superstructures)	Action-to-resistance ratio for sliding
										Stability of structures of mooring posts, bollards and mooring rings (overturning of superstructures)	Action-to-resistance ratio for overturning

9.1.1 Fundamentals of Performance Verification

(1) Mooring posts, bollards and mooring rings shall have structures that are safe against tractive forces by ships acting on them. They shall also have such sizes and shapes that they will not interfere with operations for mooring and unmooring ships.[1)]

(2) For the performance verification of mooring posts, bollards and mooring rings, **References2)** and **3)** can be used.

9.1.2 Layout of Mooring Posts, Bollards and Mooring Rings

(1) Mooring posts, bollards and mooring rings shall be located appropriately so as to enable the safe and smooth mooring of ships and cargo handling operations in consideration of the positions of the mooring ropes for the ships using the mooring facilities concerned.

(2) In general, bollards are installed close to the berth face line for mooring ships under ordinary conditions or for berthing and unberthing ships, whereas mooring posts are installed around both ends of the berth and as far away from the berth face line as possible for mooring ships in storm conditions.

(3) For positions and names of mooring ropes for a moored ship, **Part III, Chapter 5**, **2.1.1 Dimensions of Wharves** shall be referred to.

(4) It is common for posts to be installed close to the berth face line if they are intended to be used for mooring ships under ordinary conditions or for berthing and unberthing of ships. Otherwise, mooring ropes will traverse an apron and interfere with operations for loading and unloading ships. In principle, bollards shall be selected for use as posts in such positions, because mooring ropes may be stretched at a large angle of elevation. However, for mooring facilities to be used by small ships, mooring posts are often selected for use as posts installed close to the berth face line, because the crowns of small-ship mooring facilities are at almost the same level as decks of ships and thus mooring ropes are unlikely to be stretched at an extremely large angle of elevation. For the distance intervals between bollards and the minimum number of bollards installed per berth, the values given in **Table 9.1.1**. can be used as a reference[4)] When the gross tonnages of design ships are 200,000 tons or more, the distance intervals between bollards and the number of bollards installed may be determined by reference to the placement of bollards for ships with capacities of 150,000 tons or more and less than 200,000 tons.

Table 9.1.1 Placement of Bollards

Gross tonnage of design ship (ton)		Maximum interval between bollards (m)	Minimum number of bollards installed per berth (unit)
	Less than 2,000	10-15	4
2,000 or more	and less than 5,000	20	6
5,000 or more	and less than 20,000	25	6
20,000 or more	and less than 50,000	35	8
50,000 or more	and less than 100,000	45	8
100,000 or more	and less than 150,000	45	10
150,000 or more	and less than 200,000	45	12

(5) At small-ship mooring facilities where mooring ropes are unlikely to be stretched at a large angle of elevation, there are cases where mooring posts are installed at intervals of 10 to 20 m, instead of installing bollards. It is common to moor a small ship by stretching mooring ropes from the bow and stern of the ship to a quaywall, so mooring rings and other mooring devices that have strength equivalent to that of bollards may be installed at intervals of 5 to 10 m, in place of bollards. Such mooring devices can be installed on either the top or the side of mooring facilities. When they are installed on the side, they shall be positioned at an appropriate height with due consideration of the tide level.

(6) When posts are installed only in positions close to the berth face line at mooring facilities where ships are moored even in a storm, they cannot effectively work against the forces acting on a ship from the side. Therefore, posts shall also be installed away from the berth face line so that mooring ropes can be stretched as perpendicularly to the center line of a ship as possible and will not interfere with traffic for operations for loading and unloading ships. Mooring posts can be selected for use as posts in such positions, because mooring ropes hung on them are unlikely

to be stretched at a large angle of elevation. Mooring posts are positioned according to the conditions of use by ships. In general, they are installed in such positions as to allow mooring ropes to be stretched as perpendicularly to the center line of a ship as possible and, thus, effectively work against the forces acting on the body of a ship from the side. It is common to install two mooring posts on one berth. Bow ropes and stern ropes of a ship are stretched so that each rope makes a small angle with the center line of the ship to control the movement of the ship in the direction of its center line. It is preferable to install bollards so that the angle is kept larger than 25° to 30°. **Fig. 9.1.1** shows typical examples of arrangements of mooring posts. There are some cases where it was decided not to install mooring posts at a port where strong wind blows only in a certain direction and never come from the berth side (the land side) or at a port where no ship is moored in strong wind.

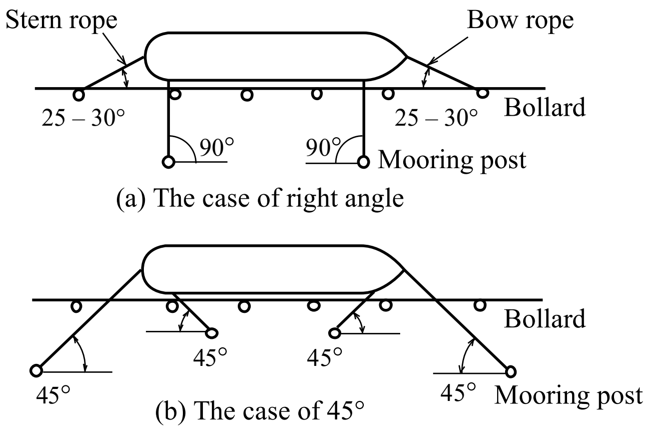


Fig. 9.1.1 Angles of Mooring Ropes against Quaywall Face Line

(7) There are cases where the mooring ropes stretched from two adjacently moored ships are hung on one mooring post or bollard installed at the junction of two berths. Since the ropes are stretched from different directions and their resultant force is not much larger than the tractive force from either of the ships, there is no need to install a larger-sized mooring post or bollard at the junction of two berths. However, to ensure safe release of mooring ropes for unberthing ships, it is preferable to install two bollards. At large mooring facilities, there are cases where bow ropes and stern ropes are stretched from both sides of a ship and, thus, there are four or more ropes from each of the bow and the stern. In view of this, it is preferable to install pairs of bollards at points to hang these ropes.

(8) From the aspect of safety in mooring and unmooring ships, bollards should be installed as close to the berth face line as possible and kept at a certain distance from curbing.[1)] It should be noted that, when bollards are positioned on the land side of curbing, mooring ropes hung on the bollards are likely to interfere with the curbing and thereby get damaged and/or bounce up. It is preferable that each mooring post or bollard be surrounded by a flat area with no obstacles or differences in level so that mooring ropes will not get damaged when rubbing against the mooring post or bollard and can be hung on and released from it smoothly.

9.1.3 Actions

(1) The tractive forces by design ships shall be appropriately calculated considering the berthing and mooring conditions of ships. For setting the tractive forces by design ships, **Part II, Chapter 8, 2.4 Actions Caused by Traction of Ships** shall be referred to.

(2) The verification of stability of superstructures against sliding and overturning shall be performed in terms of tractive forces from the most dangerous traction angles. The traction angles of the most dangerous tractive forces can be calculated using **equations (9.1.1)** and **(9.1.2)**. The envisaged ranges of traction angles depending on conditions such as the dimensions of design ships and tide levels need to be given due consideration. **Fig. 9.1.2** illustrates actions caused by the tractive force in sliding or overturning of a superstructure. The subscript *k* indicates the characteristic value in the following equations.

① The case of sliding (Refer to **Fig. 9.1.2 (a)**.)

$$\theta_k = \tan^{-1}\left(\frac{f}{m}\right) \quad \textbf{(9.1.1)}$$

where

θ : traction angle (rad)

f : friction coefficient

m : adjustment factor; m = 1.0 shall be used.

② The case of overturning (Refer to **Fig. 9.1.2 (b)**.)

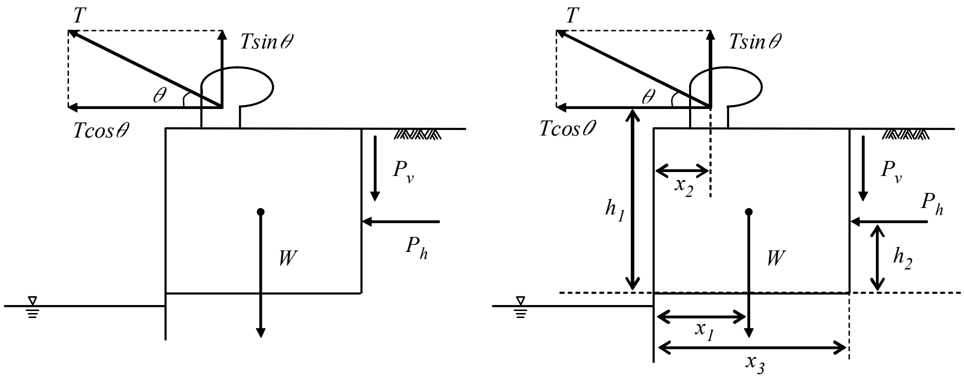
$$\theta_k = \tan^{-1}\left(\frac{x_2}{h_1}\right) \quad \textbf{(9.1.2)}$$

where

θ : traction angle (rad)

x_2 : distance from face line of quaywall to tractive force acting point (m)

h_1 : distance from bottom of superstructure to tractive force acting point (m)



(a) Sliding (b) Overturing

Fig. 9.1.2 Actions Caused by Tractive Forces

9.1.4 Performance Verification

(1) The performance of superstructures on which mooring posts, bollards and mooring rings are installed shall be verified in terms of the stability against sliding and overturning. When a superstructure is constructed behind a quaywall, not on the quaywall face line, the stability of the superstructure against sliding and overturning shall be verified by appropriately considering the forces of active earth pressure and passive earth pressure as components of the resultant vertical earth pressure and the resultant horizontal earth pressure.

① **Verification of stability against sliding**

The following equation can be used for verifying the stability against sliding of the superstructures on which mooring posts, bollards and mooring rings are installed.

$$f(W + P_v - T\sin\theta) \geq m(T\cos\theta + P_h) \quad \textbf{(9.1.3)}$$

where

f : friction coefficient

W : weight of superstructure (kN/m)

θ : traction angle (rad)

T : tractive force (kN/m)

P_v : resultant vertical earth pressure acting on superstructure (kN/m)

P_h : resultant horizontal earth pressure acting on superstructure (kN/m)

m : adjustment factor; m = 1.0 shall be used

② **Verification of stability against overturning**

The following equation can be used for verifying the stability against overturning of the superstructures on which mooring posts, bollards and mooring rings are installed.

$$x_1 W + x_3 P_v \geq m(h_1 T \cos\theta + x_2 T \sin\theta + h_2 P_h) \quad \textbf{(9.1.4)}$$

where

W : weight of superstructure (kN/m)

θ : traction angle (rad)

T : tractive force (kN/m)

P_v : resultant vertical earth pressure acting on superstructure (kN/m)

P_h : resultant horizontal earth pressure acting on superstructure (kN/m)

x_1 : distance from face line of quaywall to superstructure weight acting point (m)

x_2 : distance from face line of quaywall to tractive force acting point (m)

x_3 : distance from face line of quaywall to acting point of resultant vertical earth pressure (m)

h_1 : distance from bottom of superstructure to tractive force acting point (m)

h_2 : distance from bottom of superstructure to acting point of resultant horizontal earth pressure (m)

m : adjustment factor; m = 1.1 shall be used.

(2) When a superstructure equipped with mooring posts, bollards and mooring rings is constructed on a sheet pile quaywall, a mooring dolphin or other mooring facilities, it is connected to the heads of steel sheet piles, steel pipe piles or the like. Therefore, the verification of its stability shall be performed by using an appropriate method with due consideration of structural characteristics of the mooring facilities.

9.2 Fender Systems

[Public Notice] (Performance Criteria of Fender Systems)

Article 61

The performance criteria of fender systems shall be as prescribed respectively in the following items:

(1) The fender systems shall be located appropriately and provided with the necessary dimensions so as to enable the safe and smooth berthing and mooring of ships in consideration of the environmental conditions to which the systems are subjected, the berthing and mooring conditions of ships, and the structural type of mooring facilities.

(2) The risk that the berthing energy of ships may exceed the absorbed energy of the fender system under the variable situation, in which the dominating action is ship berthing, shall be equal to or less than the threshold level.

[Interpretation]

11. Mooring Facilities

(14) Performance Criteria of Fender Systems (Article 33 of the Ministerial Ordinance and the interpretation related to Article 61 of the Public Notice)

Serviceability shall be the performance requirement of fender systems under the variable situation in which the dominating action is ship berthing. The performance verification items and standard indexes to determine the limit values for such actions shall be as shown in **Attached Table 11-31**.

Attached Table 11-31 Performance Verification Items and Standard Indexes for Determination of Limit Values of Fender Systems in Each Design State

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index for Determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
33	1	2	61	-	2	Serviceability	Variable	Ship berthing	–	Berthing energy against fender system	Absorbed energy of fender system

9.2.1 Fundamentals of Performance Verification

(1) To verify the performance of a fender system, its installation position and dimensions shall be appropriately determined by considering the environmental conditions to which the system is subjected, the berthing and mooring conditions of ships, and the structural type of the mooring facility to ensure that the ships are berthed and moored safely and smoothly.

(2) When a ship is berthed to a mooring facility or when a moored ship motions owing to the actions of wind and waves, a berthing force and impact forces are generated between the ship and the mooring facility. To prevent damage to the ship’s hull and the mooring facility due to the generated forces, fender systems shall be installed on the mooring facility in principle. However, fender systems are not always required in cases where small ships, certain types of ferries, and other ships provided with fender equipment, such as ship fenders or tires, are maneuvered very carefully during berthing by considering the fender equipment’s energy absorption capacity; thus, the berthing force is relatively small.

(3) Rubber fenders and pneumatic fenders are commonly selected for use in fender systems. Other types of fenders, such as foam type, hydraulic type, gravity type, pile type, and timber type, are also used.[5)]

(4) The procedure for performance verification of rubber fenders, pneumatic fenders, and pile type fenders is shown in **Fig. 9.2.1** as an example.

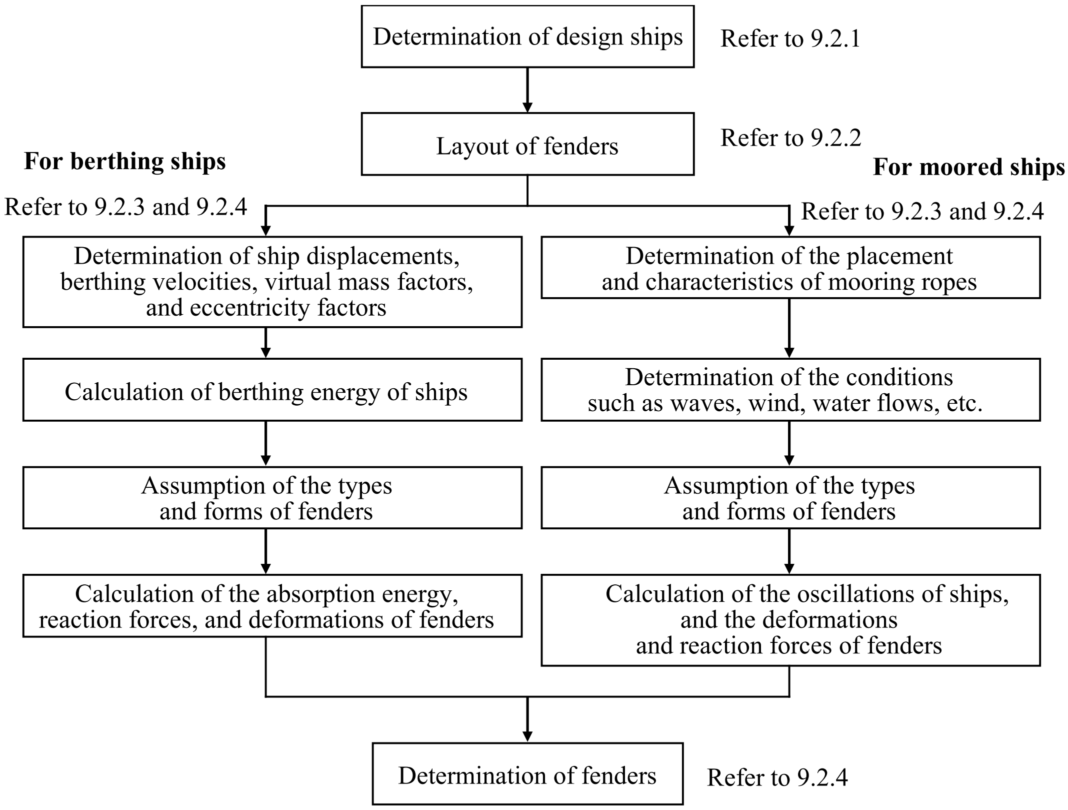


Fig. 9.2.1 Example of Performance Verification Procedure for Fenders

(5) The performance of fenders significantly affects the construction costs of mooring facilities, the maintenance costs after construction, and the efficiency of loading and unloading ships. Therefore, it is preferable to consider not only the construction costs of the fenders but also the comprehensive costs of all the aforementioned factors when selecting fenders. In general, the structures of mooring facilities, such as piled piers and dolphins, are significantly affected by the reaction forces of fenders; thus, the total construction costs of such mooring facilities may be reduced by selecting high-performance fenders even if they are expensive. By contrast, the structures of gravity-type and sheet-pile-type mooring facilities are not affected by the fender reaction forces caused by ships; thus, the performance of fenders does not affect the construction costs of those mooring facilities. However, by considering the maintenance costs after the completion of construction, the selection of easy-to-maintain fenders may result in cost reduction in the long run even if their initial cost is high. There are also cases in which high-performance fenders should be selected to allow the berthing of ships even under relatively severe oceanographic and meteorological conditions and to reduce the motions of the moored ships, thereby improving the efficiency of loading and unloading ships.

9.2.2 Layout of Fenders[5) 6)]

(1) Fenders shall be appropriately placed so that the ships have no direct contact with the mooring facilities before the fenders absorb a certain amount of berthing energy.

(2) Fenders are normally placed at 5 to 20-m intervals. When a ship berths, a point near the bow or stern initially contacts the mooring facility. Since the ship has a curved surface on the side facing the mooring facility, the fenders placed at excessively long intervals cause direct contact between a part of the ship's hull and a part of the mooring facility, where no fender is placed, before the fenders absorb a sufficient amount of berthing energy. Intervals of approximately 5 m normally cause no problem. However, when the intervals are 10 m or more and when a part of the ship's hull may directly contact a part of the mooring facility where no fender is placed, it is preferable to construct concrete superstructures so that the parts where the fenders are placed project from other parts by 0.2 to 0.5 m. Fenders should be placed at intervals of approximately 1/5 to 1/6 of the length of the parallel side of design ships.[7)]

(3) In case of large mooring facilities to be berthed by small ships, where the fenders for large ships are placed at long intervals and the fenders for small ships are placed in between them, the front surfaces of the fenders for small ships shall be set back from those for large ships to some extent. If the front surfaces of the fenders for small ships are not adequately set back, large ships may contact the small ship fenders that have small energy absorption capacity while being berthed, causing a significant increase in the reaction forces of the small ship fenders.

(4) In majority of the cases, timber fenders are placed continuously on the front surface of a mooring facility. There are also cases in which timber fenders are concentrated at intervals of 8 to 13 m.

(5) Particular attention shall be paid to the layout of the fenders for piled piers and other mooring facilities, where the dominating action is the berthing forces of ships.

(6) The installation heights of fenders shall be carefully determined by considering the design ships, which may include cement tankers and other ships with very low gunwale, as well as ferries and other ships with very high gunwale.

(7) When a small ship is berthed at a mooring facility located in a place, where a large height difference can be observed between high and low tides or where there are large waves, the side of the ship may come into direct contact with the mooring facility with no cushioning by fenders; further, the gunwale of the ship may get caught by protruding fenders. Thus, it is necessary to carefully determine the installation heights of the fenders. To prevent these problems, the fenders may be placed horizontally in two lines, or vertical fenders may be placed.

(8) For mooring facilities used by container ships and car carriers, especially ships with large flare, it is preferable to take measures to prevent the ship's side from coming into direct contact with mooring posts, bollards, container cranes, or other cargo handling equipment.

(9) From the aspect of safety while mooring and unmooring ships, mooring ropes may interfere with the top or bottom of a fender, especially a fender with a fender board.[1)] To prevent such interference, it is necessary to take measures for the top of a fender board; for example, selecting the structure that has no hangers or other protruding parts or providing a fender board with chains to prevent it from catching mooring ropes. It is also necessary to take measures for the bottom of a fender board; for example, providing it with steel members that prevent mooring ropes from getting caught underneath the fender board or adjusting the shape and/or installation height of the fender board so that its bottom will not be exposed above the sea surface at low tide. It is preferable to determine the positions of the fenders on the face line of a quaywall relative to the mooring posts or bollards by considering the actual operations for mooring and unmooring ships.

9.2.3 Actions

(1) For calculating the berthing energy of ships, **Part II**, **Chapter 8**, **2.2 Actions Caused by Ship Berthing** shall be referred to.

(2) To calculate the berthing force, it is common to initially draw a curve of load versus energy absorbed by a mooring facility and a curve of load versus energy absorbed by the whole part of one fender, and then to obtain a curve of load versus absorbed energy that shows the sum of the absorbed energy E_{f1} caused by the fender deformation and the absorbed energy E_{f2} caused by the mooring facility deformation, as shown in **Fig. 9.2.2**. The berthing force P for a given berthing energy E_f can be determined from the obtained curve. For gravity-type mooring facilities or other rigid mooring facilities, it shall be assumed that no energy is absorbed by the deformation of the main body of the mooring facility in general.

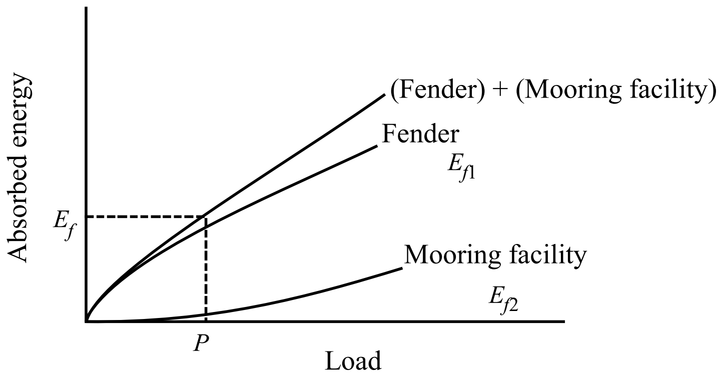


Fig. 9.2.2 Relation between Load and Absorbed Energy

(3) At mooring facilities exposed to wave actions, ships oscillate in both the horizontal and vertical directions owing to waves. These motions may cause excessive shear deformation in fenders in addition to the normal compressive deformation, and there were some cases in which fenders broke because of these deformations. There were also some cases in which fenders broke because of rough berthing of small ships at large mooring facilities available to both large and small ships. Since the coefficient of friction between a dry rubber material and an iron material is approximately 0.3 to 0.4, the shearing force, assumed as the friction force, is estimated to be approximately 30 to 40% of the reaction force of fenders at mooring facilities similar to those mentioned above. There was a case in which portable pneumatic fenders were selected for a port exposed to strong swells so that the fenders would not be damaged by the shearing force acting on them. It must be noted that some of the constant reaction type fenders exhibit the characteristic that the compression reaction force decreases when the shearing force acts on them.

(4) The fenders shall be equipped with a fender board or the like, as necessary, to reduce the load (surface pressure) per unit area to prevent the berthing force and other forces from acting on ships as a concentrated load. A synthetic resin plate or the like may be attached on the front surface of a fender board to reduce the shearing force acting on the fender.

9.2.4 Performance Verification

(1) An appropriate type of fenders shall be selected by considering the following items:

① Structural characteristics of the mooring facilities and the ships using them.

② For the mooring facilities exposed to wave actions, the motions of the moored ships and the ship berthing conditions such as the berthing angles.

③ Effects of the reaction forces of the fenders generated during ship berthing on the structures of the mooring facilities.

④ Variation ranges of the physical characteristic values of fenders due to manufacturing variation, dynamic characteristics, thermal characteristics, and other factors.

(2) Mooring facilities, such as the gravity-type mooring facilities, sheet-pile-type mooring facilities, and mooring facilities with relieving platforms, exhibit sufficient resistance against normal berthing forces. By contrast, piled piers, dolphins, detached piers, and other mooring facilities with a flexible structure, especially mooring facilities constructed on vertical piles, exhibit relatively small resistance against horizontal forces; therefore, the berthing forces must be lower than the tolerable load level. For the performance verification of the resistance of the piled piers, dolphins, and detached piers against berthing of ships, **Part III, Chapter 5, 5 Piled Piers of this Chapter** shall be referred to.

(3) The berthing energy is absorbed by the deformation of a ship's hull and the deformation of a mooring facility. However, the small energy absorbed by the deformation of the ship's hull is commonly disregarded.

① The deformation of a ship's hull can be classified as local deformation or whole deformation.

(a) Local deformation (Local deformation at the shell and ribs of the hull)

(b) Whole deformation (Strength of the entire hull against side bend)

② The deformation of a mooring facility can be classified as the deformation of the main body of the mooring facility or the deformation of one or more fenders.

(a) Deformation of the main body of the mooring facility (Deformation of a piled pier, dolphin, or other mooring facility due to ship berthing)

(b) Deformation of one or more fenders (Deformation of fenders due to ship berthing)

(4) The energy absorption by the deformation of a mooring facility can be given as follows:

① It shall be assumed that there is no energy absorption by the deformation of the main bodies of such rigid mooring facilities as gravity-type mooring facilities, sheet-pile-type mooring facilities, mooring facilities with relieving platforms, and cellular-bulkhead mooring facilities in general.

② Piled piers, dolphins, detached piers, and similar mooring facilities are classified into two types: with a rigid structure and with a flexible structure. There is no energy absorption by the deformation of the former type facilities. However, energy absorption can be observed by the deformation of the latter type facilities because of their flexibility, and the energy absorption can be generally given by **equation (9.2.1)**.

$$E_1 = \int_0^{Y_1} g(y_1)dy_1 \qquad \textbf{(9.2.1)}$$

where

E_1 : energy absorbed by the deformation of the main body of the mooring facility (kJ)

Y_1 : maximum displacement of the main body of the mooring facility (m)

y_1 : displacement of the main body of the mooring facility (m)

$g(y_1)$: characteristics of the reaction force caused by the deformation of the main body of the mooring facility (kN)

Mooring facilities having a flexible structure are normally manufactured using steel materials. Since their performance required for the actions caused by the berthing forces of ships is serviceability and the responses are within an elastic limit, there is a linear relation between the deflection and reaction forces of such mooring facilities. When a mooring facility and its fenders completely absorb the berthing energy of a ship, the energy absorbed by the mooring facility can be expressed by **equation (9.2.2)**, where C denotes the spring constant of the mooring facility.

$$E_1 = \frac{1}{2}CY_1^2 \qquad \textbf{(9.2.2)}$$

The same shall apply to the energy absorbed by the pile-type fenders.

③ The single pile structure (SPS) is a type of structure expected to absorb the berthing energy through the deformation of the piles made from high strength steel. In the performance verification of the berthing dolphins that use SPS, it is necessary to evaluate the amount of energy absorption by considering the residual deformation of the piles due to repeated berthing. As shown in **Fig. 9.2.3**, the amount of energy that can be absorbed by piles during ship berthing can be calculated from the displacement obtained by subtracting the residual displacement from the loading point displacement.[8)]

The loading point displacement with the considered residual displacement can be calculated from **equation (9.2.3)**.

$$y_{top} = A_1 y_0 + A_2 i_0 h + \frac{Ph^3}{3EI} \qquad \textbf{(9.2.3)}$$

where

y_{top} : loading point displacement (m)

y_0 : pile displacement at sea bottom at the time of initial loading (m)

i_0 : pile deflection angle at sea bottom at the time of initial loading (rad)

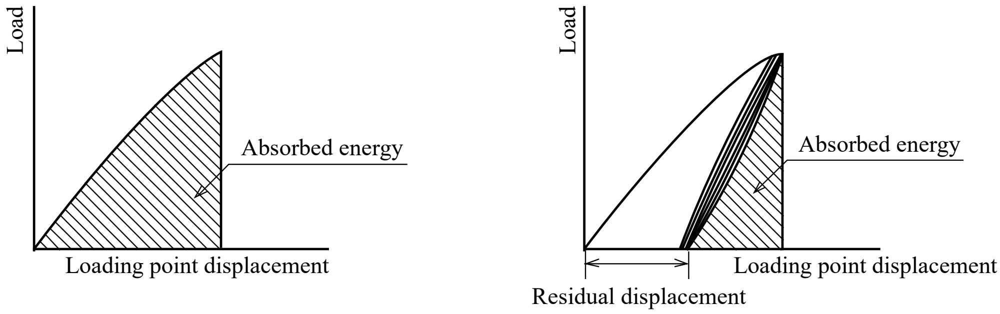
P : horizontal load (kN)

h : height of loading point (m)

EI : flexural rigidity of the pile (kNm^2)

A_1, A_2 : coefficients of influence of repeated loading

The time of initial loading refers to the situation in which the pile is exposed to the largest ever load.



(a) Absorbed energy at the time of initial loading

(b) Absorbed energy at the time of ship berthing

Fig. 9.2.3 Energy Absorbed by Deformation of Piles

Table 9.2.1 presents the values of the coefficients of influence of repeated loading that were obtained from the results of the in-situ full-scale loading tests[9] and the model tests.[10]

Table 9.2.1 Values of the Coefficients of Influence of Repeated Loading [8]

	For obtaining the maximum displacement	For obtaining the energy absorbed by the deformation of piles	For obtaining the residual displacement
A_1	1.4	0.4	0.8
A_2	1.2	0.6	0.5

(5) For mooring facilities with a rigid structure, where there is no energy absorption by the deformation of their main bodies, the energy absorbed by a fender can be calculated using the following equation:

$$E_s = \phi E_{cat} \geq E_f \tag{9.2.4}$$

where

E_s : energy absorbed by the fender (kJ)

ϕ : manufacturing error (tolerance) of the fender

E_{cat} : specified value of the energy absorbed by the fender (kJ)

E_f : berthing energy of the ship (kJ)

The characteristic value of the berthing energy of a ship can be expressed by **equation (2.2.1)** given in **Part II, Chapter 8, 2.2 Actions Caused by Ship Berthing**.

(6) There are various types of rubber fenders, including V-shaped, circular hollow, and rectangular hollow types. They differ from each other in terms of the relation between the reaction force and deformation as well as the energy absorption rate. Fender manufacturers' catalogs shall be referred to regarding the graphs of the amount of energy absorption versus the deformation and those of the reaction force versus the deformation for each type of fender.

Constant reaction type fenders, such as V-shaped fenders, are characterized by the low reaction forces and high energy absorption rates. However, it must be kept in mind that the total reaction force of such fenders may become large when a ship simultaneously comes in contact with two to three fenders. This is because the reaction force of each fender increases almost to the maximum level when it contains absorbed energy equivalent to one-third of its design capacity.

(7) The factors that cause variations in the characteristics of rubber fenders include the manufacturing variation, deterioration over time, dynamic characteristics (i.e., velocity-dependent characteristics), creep characteristics, repetition characteristics (i.e., compression frequency-dependent characteristics), oblique compression characteristics, and thermal characteristics. For fenders used for mooring floating structures, these factors are important in evaluating their safety as mooring equipment. For fenders used for mooring ships, it is also appropriate to verify the performance of the fenders by considering the factors, including the manufacturing variation, dynamic characteristics (i.e., velocity-dependent characteristics), oblique compression characteristics, and thermal characteristics. For example, when the manufacturing variation (tolerance) of a fender is ±10%, it is preferable to decrease the characteristic (performance) values presented in its catalog by 10% for calculating the amount of energy absorption and to increase the values by 10% for evaluating the reaction force of the fender that acts on the mooring facility. With regard to the dynamic characteristics, it is preferable to confirm that the reaction force of a fender at the time of ship berthing will not exceed the standard capacity shown in the catalog issued by its manufacturer by considering the berthing velocity of the ships. It should also be borne in mind that the fender reaction force becomes higher in a low-temperature environment than that in the standard temperature environment.

(8) A working group of the World Association for Waterborne Transport Infrastructure (PIANC) has recommended to correct the amount of energy absorption and the reaction force in the standard environment using the velocity correction factor and the temperature correction factor when selecting a fender by considering the fact that its characteristics will vary depending on the ship berthing velocity, the ambient temperature, and other conditions of the actual environment in which the fender will be used. The working group has also published guidelines[11) 12)] for selecting a fender using these correction factors. The actual values of the velocity correction factor and the temperature correction factor should be checked with the fender manufacturers as they vary depending on the ship berthing velocity, the ambient temperature, and the kind of rubber used for the fender. It should also be borne in mind that the reaction force acting on a mooring facility during the berthing of a small ship at a high berthing velocity may be larger than that during the berthing of a large ship at a low berthing velocity.

(9) The berthing force of a ship may cause permanent deformation of the shell of the ship; hence, it is necessary to select a fender carefully.[13) 14)] It is preferable to attach fender boards on the front surfaces of fenders as necessary to reduce the loads on the ships. Since the damage to the shell of a ship is affected by not only the magnitude of berthing force but also the structural strength of the shell, it is preferable to increase the contact area of each fender so that it contacts two ribs of the ship at the same time. The guidelines for designing fenders[11) 12)] recommend that the maximum allowable surface pressure for each type of ship should be approximately 200 to 400 kN/m^2. For the effects of the reaction force of fenders on the shell structure, **References 13) 15)** and **16)** can be used.

(10) Fenders must also be safe against the shearing force generated by the friction that occurs in the direction of the face line of a mooring facility due to oblique berthing of the ships. This force can be normally calculated using the equation suggested by Vasco Costa[17)]. When a ship is berthing to a mooring facility at an angle of 6 to 14° with the face line of the mooring facility, this force becomes 10 to 25% of the berthing force of the ship.

(11) According to the simulation results of the motions of moored ships[7) 18)], the deformation of fenders owing to the motions of a moored ship can be larger than the deformation owing to the berthing force of the ship when the period of waves acting on the ship is long due to swells, when waves act on the side of the ship's hull perpendicularly, or when the value of E/δ_a, which is the ratio of the amount of energy absorption by a fender E to the allowable deformation δ_a, is large. Therefore, it is advisable to select a fender that has a small value of E/δ_a, i.e., a fender that has the largest value of allowable deformation δ_a among the fenders which are equivalent with respect to the amount of energy absorption E.

9.3 Skirt Guards

9.3.1 General

(1) The facility shall be equipped with appropriate skirt guards when there is a risk of a small ship getting into an empty space beneath the slab of a piled pier, dolphin, or other mooring facility.

(2) The majority of skirt guards have precast slabs or section steel members, which are placed in the form of a wall, comb, or grid in appropriate positions by considering the tidal range and other conditions.

9.4 Lighting Facilities

[Public Notice] (Performance Criteria of Lighting Facilities)

Article 62

The performance criteria for lighting facilities shall be such that appropriate lighting facilities are installed so as to enable the safe and smooth use of mooring facilities where cargo handling operations, berthing and unberthing of ships, and entry and exit of people occur in consideration of the usage conditions of the mooring facilities.

9.4.1 General

(1) Appropriate lighting facilities should be provided at mooring facilities and related facilities where cargo handling works such as loading, unloading and transfer, berthing/unberthing of ships, and use by passengers and others are performed at night in consideration of the use conditions of the concerned mooring facilities.

(2) The description here may be applied to the installation, improvement, and maintenance of lighting facilities at wharves where cargo handling, berthing and unberthing, passenger use, etc., are performed at night. The lighting facilities for other facilities shall comply with the descriptions here and the standards separately provided for respective facilities.

(3) Many lighting facilities are designed these days to highlight the night views of structures, parks, watersides, etc., in urban fringes and tourist sites to meet social needs for lighting and other facilities in port facilities. In these cases, not only illumination but also light colors and color-rendering property are needed to give people pleasure, familiarity, and peace of mind. On the contrary, given that lighting facilities have come into wide use, it is essential to consider the adverse effects of lighting on the surroundings and on energy savings. The performance verification of lighting facilities should fully take into account these demands. Properly examine lighting functions and individually take necessary measures suited to individual facilities in coastal areas where people interact, such as amenity-oriented revetments, marinas, parks, and promenades.

9.4.2 Performance Verification Items for Lighting Facilities

(1) In designing lighting facilities, the locations of lamp fittings shall be determined by appropriately selecting lighting methods, light sources, and lighting apparatuses in consideration of the following items according to the installation locations of the lighting facilities. Furthermore, lighting facilities that have possible influences on sea surfaces should be designed in a way that prevents its interference with ship navigation at sea.

① Standard intensity of illumination
② Distribution of illumination
③ Glare
④ Color and color-rendering property
⑤ Obstacle light and energy saving

9.4.3 Standard Intensity of Illumination

(1) General

① The standard intensity of illumination is an average horizontal-plane illumination and is defined as the minimum value for safely and effectively using the facilities concerned. The objective generally used in designing lighting facilities is illumination. Horizontal illumination is the illumination of a floor surface or a ground surface. The average horizontal illumination is the average value of that illumination.

② The illumination of lighting facilities should be properly determined on the basis of varieties and systems of work to enable the facilities concerned to be used safely and smoothly.

③ The International Commission on Illumination (CIE) has been examining the criteria of illumination and has published **GUIDE FOR LIGHTING EXTERIOR WORK AREAS.** The guide includes the recommendations for the regulation values of maintenance illumination and the uniformity ratios of illumination, glare, and color-rendering property.

④ The performance verification of lighting facilities can refer to the standard intensity of illumination described here because it has been determined by taking into consideration the following laws and regulations, actual situations of lighting facilities at domestic and international ports, and other reference materials. However, given that the standard intensity of illumination described here is the minimum value, it can be increased as needed.

(a) **Ordinance on Industrial Safety and Health** (Ordinance of the Ministry of Labor No. 32, September 30, 1972)

(b) **Order for Enforcement of the Parking Place Act** (Cabinet Order No. 340, December 13, 1957)

(c) **General Rules of Recommended Lighting Levels** (JIS Z 9110: 2010)

(d) **Lighting for Roads** (JIS Z 9111)

(e) **Lighting of Tunnels for Motorized Traffic** (JIS Z 9116)

(f) **Standards and Commentaries for the Installation of Road Lighting Facilities**[19)]

(g) **Lighting of Outdoor Work Places**[20)]

(h) **Lighting of Indoor Work Places** (JIS Z 9125: 2007)

(i) **Lighting of Outdoor Work Places** (JIS Z 9126: 2010)

(j) **Standard and Design Guide for Lighting of Indoor Work Places** (JCIE-002 2009)

(k) **Maintenance Factors and Maintenance Planning in Lighting Design, Third Edition** (JIEG-001 (2005))

(l) **Maintenance Factors and Maintenance Planning in Lighting Design, Third Edition, Enlarged Edition for LED Lighting** (JIEG-001 (2013))

(2) Standard intensity of illumination for outdoor lighting

① The values shown in **Table 9.4.1** may be used for the standard intensity of illumination of each type of outdoor facility.

(a) Mooring facilities for passengers, vehicles, and pleasure boats and general cargo and container berths

The lighting for aprons on these facilities is required to show relatively advanced visual information, such as the clothes and facial expressions of passengers and the types, shapes, and colors of vehicles or cargoes, and meet a certain level of comfort. Thus, the standard intensity of illumination is set at 50 lx with reference to the **General Rules of Recommended Lighting Levels (JIS Z 9110)** and actual measurements at existing lighting facilities (**Table 9.4.1**).

Table 9.4.1 Standard Intensity of Illumination for Outdoor Lighting

Facility			Standard intensity of illumination (lx)
Wharf	Apron	Mooring facilities for passengers, vehicles, and pleasure boats and general cargo and container berths	50
		Slipways for pleasure boats and aprons for handling dangerous goods using pipelines	30
		Aprons for simple work using pipelines and belt conveyors	20
	Yard	Container yards, general cargo storage, handling yards, and cargo transfer yards	20
	Path	Passenger gates and vehicle gates	75
		Passenger paths and vehicle paths	50
		Other paths	20
	Security	All facilities	1–5

Facility			Standard intensity of illumination (lx)
Road and Park	Road	Main roads	20
		Other roads	10
	Parking lots	For ferries	20
		Others	10
	Park and green space	Garden paths	3

(b) The slipways for pleasure boats and aprons for handling dangerous goods using pipelines

The standard intensity of illumination of the slipways for pleasure boats is set at 30 lx because these slipways are not required to show detailed visual information, such as shapes and colors. The standard intensity of illumination of the aprons for handling dangerous goods is also set at 30 lx in consideration of their safety.

(c) Aprons for simple work

The standard intensity of illumination of 20 lx is set for aprons where simple cargo handling work is executed using pipelines or belt conveyors.

③ **Container yards**

(a) In container yards, containers are transferred and stacked by straddle carriers and transfer cranes. In cargo sorting area, cargoes are transferred and loaded and unloaded from trucks by forklifts. The work procedures in container and cargo sorting areas are similar to those in inland truck yards. Thus, the standard intensity of illumination in the container and cargo handling yards is set at 20 lx with reference to the **General Rules of Recommended Righting Revels (JIS Z 9110)** and actual measurements at existing lighting facilities (**Table 9.4.1**).

(b) The standard intensity of illumination is not necessarily applied to all areas of the container yards. Container yards can be divided into zones in accordance with lighting objects, and different values of standard intensity of illumination can be set depending on the importance of work executed in respective zones, e.g., chassis travel paths and container transship points. Furthermore, in storage yards where containers are stacked, the standard intensity of illumination is not necessarily applied to the places between the stacked containers.[21) and 22)]

④ **Paths**

(a) Gates and paths for passengers and vehicles

Lighting facilities are required for movable bridges used by passengers and vehicles when they board passenger ships or ferries. Although vehicles have head lamps, the glare of head lamps may cause traffic controllers to make erroneous guidance, thereby leading to accidents such as falls into the sea. Thus, the standard intensity of illumination for paths and gates is set at 50 and 75 lx, respectively with reference to the **General rules of recommended lighting levels (JIS Z 9110)** and actual measurements at existing lighting facilities (**Table 9.4.1**).

(b) Other paths

The standard intensity of illumination for other paths, including pedestrian paths, for workers is set at 20 lx because they are at less risk of accidents than the paths for passengers and vehicles.

⑤ **Lighting for security control**

Facilities that are not used at night also require lighting for crime prevention and security control. Thus, the standard intensity of illumination for the security purpose is set at 1 to 5 lx.

⑥ **Roads**

The standard intensity of illumination for roads on and around wharves should be determined with consideration to the followings:

(a) Work conditions on wharves (necessity and frequency of night work)

(b) Traffic conditions (traffic volume, traveling speeds, and types of vehicles)

(c) Road conditions (types of alignments, structures, and pavement)

(d) Topographic conditions

(e) Economic effects.

Thus, the standard intensity of illumination for main roads and other roads is set at 20 and 10 lx, respectively, so that the above conditions can be fulfilled to the greatest possible extent with reference to the **General rules of recommended lighting levels (JIS Z 9110), Lighting for Roads (JIS Z 9111), Standards and Commentaries for the Installation of Road Lighting Facilities**, and actual measurements at existing lighting facilities. For lighting in tunnels, refer to the **Lighting of Tunnels for Motorized Traffic (JIS Z 9116)** and the **Standards and Commentaries for the Installation of Road Lighting Facilities.**

⑦ **Parking lots**

Even at slow speeds, vehicles need to be carefully maneuvered in parking lots. In particular, vehicles need lighting to accurately identify the locations of other parked vehicle. From the viewpoint of crime prevention, a certain level of intensity of illumination is required in parking lots so that vehicles and persons can be identified.

Thus, the standard intensity of illumination for the parking lots in ferry wharves and other parking lots is set at 20 and 10 lx, respectively, with reference to the **Order for Enforcement of the Parking Place Act**, the **General rules of recommended lighting levels** and actual measurements at existing lighting facilities. **(JIS Z 9110: 2010)**.

⑧ **Parks and green spaces**

The lighting at parks as places of relief needs to provide the following types of brightness:

(a) Brightness to ensure safe routes where people walk

(b) Brightness of atmosphere enabling people to feel psychologically safe

(c) Local brightness to highlight the beauty of trees and objects inside parks

(d) Brightness to crime prevention

Thus, the standard intensity of illumination and lighting methods shall be appropriately determined by taking into consideration the locations and areas, use purposes and situations, and facilities and objects requiring lighting. The standard intensity of illumination for paths is set at 3 lx with reference to the **General rules of recommended lighting levels (JIS Z 9110)**, and actual measurements at existing lighting facilities (**Table 9.4.1**).

⑨ **Others**

Mooring facilities where ships berth or unberth at nighttime should be provided with lighting as needed to make the normal lines of berths or the locations of corner sections easily identifiable.

(3) Standard Intensity of Illumination for Indoor Lighting

① The values shown in **Table 9.4.2** can be used for the standard intensity of illumination of each type of indoor facility.

Table 9.4.2 Standard Intensity of Illumination of Indoor Lighting

Facility		Standard intensity of illumination (lx)
Passenger terminal	Waiting lounges	300
	Passenger boarding paths and gates	100
Shed and Warehouse	Cargo handling spaces for fishing boat berths	200
	Container freight stations and dedicated vehicle sheds	100
	Rough work sheds and warehouses	70
	Other sheds and warehouses	50

② **Passenger terminals**

Considering that waiting lounges are places for relaxation, they shall provide people with a comfortable environment where they can feel at ease. Thus, the standard intensity of illumination for waiting lounges is set at 300 lx with reference to the **General rules of recommended lighting levels (JIS Z 9110)**, the **Standard for the Design and Construction of Electric Workpieces (Power Utilities)**, and actual measurements at existing lighting facilities (**Table 9.4.2**). The standard intensity of illumination for paths and gates is set at 100 lx with particular attention to ensuring the safety at these facilities with reference to the **General Rules of Recommended Lighting Levels (JIS Z 9110)**, the **Standard for the Design and Construction of Electric Workpieces (Power Utilities)**, and actual measurements at existing lighting facilities.

③ **Sheds and warehouses**

The standard intensity of illumination for the cargo handling spaces of fishery berths is set at 200 lx to facilitate the accurate judgment of fish freshness. For areas such as container freight stations where complex cargo handling operations are executed and facilities such as dedicated vehicle sheds that are important to security, the standard intensity of illumination is set at 100 lx.

Furthermore, for areas in sheds and warehouses where cargo sorting work or other work requiring safety is executed, the standard intensity of illumination is set at 70 lx with reference to the **Ordinance on Industrial Safety and Health** and the **General Rules of Recommended Lighting Levels (JIS Z 9110)**.

For other sheds and warehouses, the standard intensity of illumination is set at 50 lx with reference to the **General Rules of Recommended Lighting Levels (JIS Z 9110)** and actual measurements at existing lighting facilities. The standard intensity of illumination for the administration offices attached to passenger terminals, sheds, and warehouses shall be appropriately set with reference to the **General Rules of Recommended Lighting Levels (JIS Z 9110)**.

(4) Methods for calculating intensity of illumination

① **Methods for calculating intensity of illumination**

The methods for calculating the intensity of illumination include the flux method and the point-by-point method. Given that the flux method can be expressed by a relatively simple equation, it has been used for calculating the required number of lighting apparatuses in general. The point-by-point method is capable of accurately calculating the intensity of illumination required for specific points; therefore, it has been used generally for examining the evenness of intensity of illumination in a manner that obtains the intensity of illumination at individual points in areas for which the required intensity of illumination is calculated by the flux method.

(a) Calculation of intensity of illumination by the flux method

The average intensity of illumination in an area to be lit can be calculated by **equation (9.4.1)**.

Average intensity of illumination
$$E = \frac{NFUM}{A} \tag{9.4.1}$$

where

E : average intensity of illumination (lx);

N : number of lighting apparatuses (units);

F : total light flux per light source (lm);

U : utilization factor;

M : maintenance factor;

A : area of a plane of illumination (m^2).

(b) Calculation of intensity of illumination by the point-by-point method[23)]

In lighting design, the point-by-point method can be generally used to obtain the evenness of the intensity of illumination (**equation (9.4.4)**).

In the point-by-point method, a plane of illumination is first divided into pieces of rectangles in a grid pattern, and the intensity of illumination at the centers of respective rectangles is then calculated. The direct horizontal illumination E_h of light from a light source L at point P on a plane is expressed by **equation (9.4.2)** (refer to **Fig. 9.4.1**).

$$E_h = \frac{I_\theta \cos\theta}{l^2} \tag{9.4.2}$$

where

E_h : direct horizontal illumination at point P (lx);

I_θ : brightness of light with an incident angle of θ (cd);

l : distance from a light source to point P (m);

θ : incident angle (°).

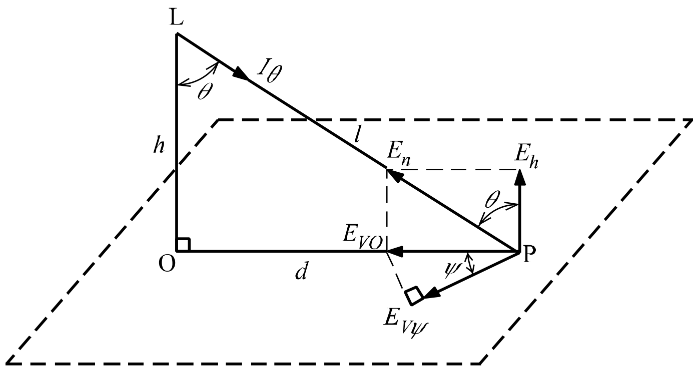


Fig. 9.4.1 Horizontal Illumination at Point P

The shape of each divided piece should be as close to a square as possible, and the finesses ratio of each piece is in the range of 0.5 to 2.0. The maximum length (p) of a side of each divided piece is the smallest length in equation **(9.4.3)**.

$$p \le 0.2 \times 5^{\log d}, \quad p = 10 \tag{9.4.3}$$

where

d : width of a calculation range (m);

p : maximum value of a side of each divided piece (m).

The evenness (E_{av}) is the minimum average intensity of illumination, and the average intensity of illumination can be calculated by **equation (9.4.4)** (refer to **Fig. 9.4.2**).

$$E_{av} = \frac{E_1 + E_2 + \cdots + E_n}{n} \tag{9.4.4}$$

where

n : number of calculation points;

E_n : intensity of illumination at the center of nth divided piece.

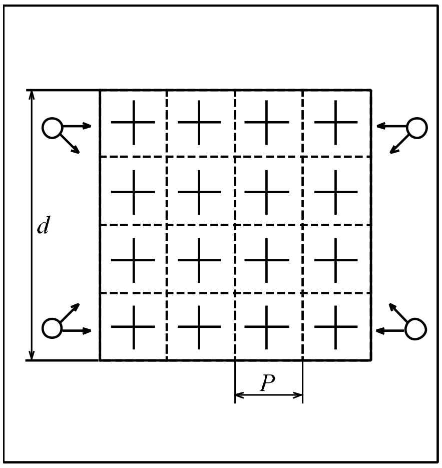


Fig. 9.4.2 Division of the Plane of Illumination in a Grid Pattern

② **Light ratio**

The light ratio is the ratio of light flux reaching the plane to be lighted to the whole flux of the light source. When calculating the intensity of illumination, it is preferable to take into consideration the fact that the utilization factors of indoor lighting varys depending on the efficiency of lighting apparatuses, areas of planes of illumination, situations inside rooms, and reflection ratios of respective sections inside rooms.

The light ratio of outdoor lighting can be calculated from the efficiency of lighting apparatuses and the areas of planes of illumination. The light ratio that have been practically used are in the range of 0.2 to 0.5 and are generally set at 0.4.

③ **Maintenance factors**

The maintenance factor is a value obtained by dividing the intensity of illumination of a lighting apparatus after a lapse of a certain period by the initial intensity of illumination. The total light flux of a light source inside a lighting apparatus is largest in an early stage and is gradually reduced as lighting time advances. The maintenance factors are also reduced when lighting apparatuses become dirty. The maintenance factors used for performance verification are determined on the basis of the assumption that the replacement of lamps and cleaning of lighting apparatuses are appropriately implemented such that maintenance factors are ensured. The maintenance factor (M) can be calculated by equation **(9.4.5)**.

$$M = M_a M_f M_d M_w \tag{9.4.5}$$

where

M_a : lumen maintenance factor of a light source;

M_f : residual factor of a light source;

M_d : partial maintenance factor due to dirt on a light source and a lighting apparatus;

M_w : partial maintenance factor due to an interior surface.

The lumen maintenance factors M_a and the residual factors M_f of light sources are determined by the characteristics of light sources; therefore, they are collectively called light source design lumen maintenance factors M_l. By contrast, partial maintenance factors due to dirt on light sources and lighting apparatuses are collectively called lighting apparatus design lumen maintenance factors M_d. The dirt and color degradation on interior surfaces cause the reductions in reflection ratios, thereby reducing the intensity illumination of interreflection components. However, the partial maintenance factors M_w are generally neglected in the calculation of maintenance factors because the contribution ratios of M_w to the reduction in the intensity of illumination are smaller than those of light sources and lighting apparatuses. Thus, the maintenance factors (M) can be calculated by equation **(9.4.6)**. For the lighting apparatus design lumen maintenance factors of high-intensity electric-discharge (HID) and LED lamps, refer to literature 23).

$$M = M_{\ell} M_d \tag{9.4.6}$$

where

M_l : a light source design lumen maintenance factor;

M_d : a lighting apparatus design lumen maintenance factor.

9.4.4 Performance Verification of Illumination Distribution

(1) Unfavorable illumination distribution on the planes of illumination not only causes passengers and workers to feel uncomfortable but also creates dark places that are difficult to see, thereby causing reduced work efficiency and accidents. Lighting design should be performed with consideration to the following items.

① Appropriate adjustment of the ratio between installation intervals and heights of lighting apparatuses to achieve favorable illumination distribution

② Auxiliary lighting apparatuses in cases of shady plants or cargoes

③ Adherence to the guiding average value for horizontal illumination and the recommended value for evenness proposed in **GUIDE FOR LIGHTING EXTERIOR WORKS AREAS** by CIE. In this guideline, evenness is defined as the ratio of the minimum illumination to the average illumination.

9.4.5 Performance Verification for Glare

(1) Glare is excessive brightness or excessive irregularity in brightness and causes people to have uncomfortable feelings and reductions in eyesight. Glare with respect to lighting facilities can be categorized into glare affecting ships and glare affecting passengers and workers.

① **Glare affecting ships**

Glare may prevent crews or pilots from identifying beacons and other ships at anchor and trigger erroneous ship handling, thus leading to accidental contact or collisions with other ships or berths. Therefore, the distribution of light and the installation locations of lighting facilities shall be carefully examined to ensure safety in ship navigation.

② **Glare affecting passengers and workers**

Glare may prevent passengers and workers from identifying cargoes and shipping tags and obstacles and may cause reduced work efficiency and fatigue. Therefore, lighting facilities shall be installed appropriately in full consideration of the interaction between the heights of the visual lines of passengers and workers and lighting apparatuses to prevent light from getting into people’s eyes directly. In **GUIDE FOR LIGHTING EXTERIOR WORKS AREA**S, the CIE recommends to observe the values set for controlling glare to prevent glare from hindering visual work and traffic safety.

9.4.6 Performance Verification of Light Colors and Color-Rendering Properties

(1) The color property of light sources can be expressed by light colors and color-rendering property.

(2) Light colors

One of the ways to numerically rate the reddishness and bluishness of light is to use color temperature. Light becomes bluish and reddish when the color temperature is higher than 5,000 K and lower than 3,300 K, respectively. Generally, metal halide lamps have the highest color temperature, followed by white mercury and fluorescent lamps with intermediary color temperature, orangish white incandescent and high-pressure sodium lamps, and orangish-yellow low-pressure sodium lamp with the lowest color temperature.

(3) Color-rendering property

The color-rendering property is the effect of light from certain light sources on the changes in colors of objects in comparison with the colors of objects lit by light from a standard light source. Generally, electric, fluorescent, and metal halide lamps have favorable color-rendering properties, followed by fluorescent mercury and high-pressure sodium lamps. Clear mercury lamps have color reproductive properties that are sufficiently favorable for the greenish colors of leaves but are unfavorable for the colors of other objects. Considering that low-pressure sodium lamps are single spectrum light sources, they cannot be used for discerning colors.

(4) Color temperature and thermal sensation

The color temperature (K) is used to numerically express light colors. The degree of color temperature affects the thermal sensation, i.e., light colors come closer to red and bluish white because the color temperature is reduced and increased, respectively. **Table 9.4.3** shows the relationship between color temperature and thermal sensation.

Table 9.4.3. Relationship between Color Temperature and Thermal Sensation

Color temperature (K)	Thermal sensation
3,300 or less	Warm
3,300 to 5,300	Neutral
5,300 or higher	Cool

(5) Color-rendering property and average color-rendering indexes

Average color-rendering indexes (Ra) are used as typical indexes to represent the degree of color-rendering property. Ra is an average of rendering indexes with respect to eight prescribed test colors. The CIE shows the applicability of classified Ra to the different types of outdoor workplaces. **Table 9.4.4** shows the relationships of classes, ranges of Ra in respective classes, types of lamps, and applicable workplaces.

Table 9.4.4. Classification of Color-Rendering Property for Outdoor Lighting

Color-rendering property class	Average color-rendering index (Ra)	Type of lamp	Applicability
1	80 ≤ Ra Very good	Incandescent lamp	Applicable to workplaces requiring color differentiation
2	60≤ Ra < 80 Good	Fluorescent lamp, metal halide lamp, high-pressure sodium lamp with improved color-rendering property, LED lamp	
3	40 ≤ Ra < 60 Satisfactory	Mercury lamp	Applicable to workplaces of general work
4	20 ≤ Ra <40 Allowable	High-pressure sodium lamp	
5	Ra < 20	Low-pressure sodium lamp	Not applicable to workplaces where color differentiation is important

9.4.7 Performance Verification for Obstacle Light and Energy Saving

(1) Light leaking from outdoor lighting facilities affects the surrounding environments in terms of disturbance in astronomical observation, increased burden on the ecosystem, and glare interfering visual identification and causes loss of energy by lighting unnecessary objects. Considering that leaked light may cause social problems, it is preferable to give due consideration to the prevention of leaked light in lighting design.

9.4.8 Selection of Light Sources

(1) Light source for wharf lighting is preferably selected by considering the following requirements:

① The light source should have high efficiency and a long service life.

② The light source should be stable against the variations of ambient temperature.

③ The light source shall provide a good light color and good color-rendering performance.

④ The time of stabilization of the light after turning-on shall be short.

(2) Any light source other than a light bulb shall be used together with an appropriate stabilizer.

(3) Types of light sources

Fig. 9.4.3 shows the classification of light sources. **Table 9.4.5** shows the summary of the characteristics of the respective types of lamps. The light sources generally used on wharves are those classified as HID lamps with the following characteristics:

① **High-pressure sodium lamps**

High-pressure sodium lamps have lower efficiency than low-pressure sodium lamps but have a long service life, favorable start-up performance, and improved color-rendering property. They also have an orangish color and require pulse voltage to start lighting. There are two types of high-pressure sodium lamps: one is a starter stabilized type with a pulse generator (starter) integrated with a lamp; and the other is a dedicated stabilizer type with the pulse generator stored in a stabilizer. The former type has approximately 10% higher efficiency than the latter type.

② **Fluorescent mercury lamps**

Fluorescent mercury lamps have lower efficiency than high-pressure sodium lamps but have a good color-rendering property. These lamps have a white light color that is similar to fluorescent lamps and can be used for yard lighting.

③ **Metal halide lamps**

Metal halide lamps have a slightly shorter service life than high-pressure sodium lamps but can be used for yard lighting similar to high-pressure sodium lamps because they have a good color-rendering property. They have a whitish light color.

④ **LED lamps**

LED lamps are light sources with a whitish light color and lower color-rendering property than halogen lamps and incandescent lamps but have a favorable service life, efficiency, and economic performance. Although LED lamps have a long service life, their accessories need to be replaced after approximately 8 years.

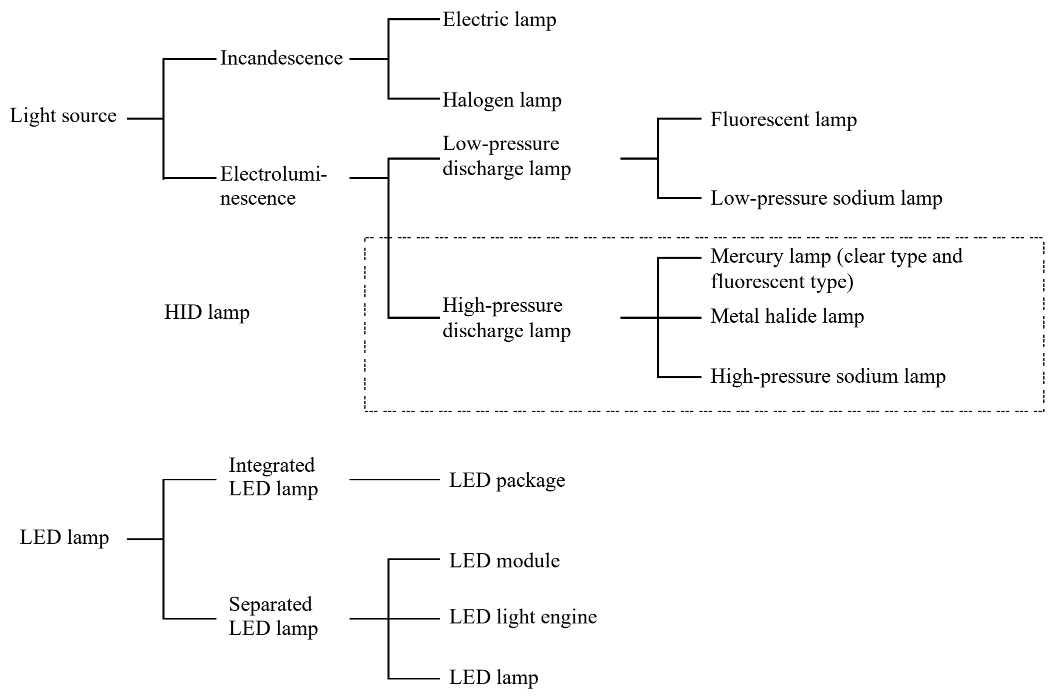


Fig. 9.4.3 Types of Light Sources

Table 9.4.5 Characteristics of Lamps

Type of lamp \ Characteristics	Lamp efficiency (lm/w)	Light color (K)	Color-rendering property (Ra)	Service live (hours)	Ambient temperature dependency	Start-up performance	Restart performance	Modulation
Incandescent	Low 15 to 20	Orangish white 2,800	Good 100	Short 1,000 to 2,000	Stable	Instantaneous	Instantaneous	Easy
Halogen	Low 17 to 22	Orangish white 3,000 to 3,200	Good 100	Short 1,000 to 2,000	Stable	Instantaneous	Instantaneous	Easy
Fluorescent (white)	Medium 80 to 100	White 3,000 to 4,000	Slightly good 50 to 95	Long 6,000 to 12,000	Affected	Fast 2 to 3 seconds	Fast 2 to 3 seconds	Possible
Low-pressure sodium	Highest 100 to 180	Orangish yellow 1,700	Bad –	Normal 9,000	Stable	20 minutes	Slightly fast 10 seconds	Difficult
Mercury	Slightly low 40 to 60	White (bluish) 3,500 to 4,000	Normal 40 to 50	Long 9,000 to 12,000	Stable	8 minutes at normal temperature	Slightly slow Less than 10 minutes	Possible up to 50%
Metal halide	Medium 70 to 80	White 4,000 to 6,500	Good 70 to 90	Normal 6,000 to 9,000	Slightly affected	5 minutes at normal temperature	Slightly slow Less than 10 minutes	Difficult
High-pressure sodium	Medium 60 to 120	Orangish white 2,100	Normal 25 to 80	Long 9,000 to 12,000	Stable	5 to 10 minutes	Slightly fast 1 to 5 minutes	Possible up to 50%
White LED (equivalent to electric lamp)	Slightly high 60 to 150	Orangish white 2,800	Slightly good 70 to 80	Long up to 40,000	Stable	Instantaneous	Instantaneous	Easy

(4) Efficiency

It is preferable to enhance not only the efficiency of lamps but also the comprehensive efficiency of lighting facilities, including stabilizer and accessories.

(5) Ambient temperature

① **Influences on efficiency**

Most light sources are not affected by ambient temperature, except fluorescent lamps. The efficiency of fluorescent lamps is reduced not only with the increase in ambient temperature but also with the decrease in ambient temperature. Therefore, it is preferable to give due consideration to the fact that the degrees of influences of ambient temperature on the efficiency of fluorescent lamps vary depending on the structures of lighting apparatuses.

② **Influence on start-up and restart time**

Electric, high-pressure sodium, and low-pressure sodium lamps are not affected by ambient temperature, but fluorescent and metal halide lamps degrade their start-up performance when ambient temperature is reduced. In such cases, lamps and stabilizers shall be provided with special measures to prevent the degradation of start-up performance.

9.4.9 Selection of Apparatuses

(1) Outdoor lighting

① It is preferable to select apparatuses for outdoor lighting in consideration of the following requirements:

(a) Lighting apparatuses should have rainproof structures. Furthermore, they shall have explosion-proof structures for cases wherein large amounts of flammable dangerous goods are handled in the proximity of lighting apparatuses. In particular, outdoor lighting facilities using LED apparatuses should be provided with measures against low temperature and rainwater to prevent apparatuses from dew condensation.

(b) The materials used for lamp bodies, reflector surfaces, and illumination covers should have good quality, have high durability, and good resistance against deterioration and corrosion.

(c) Sockets should be compatible with respective light sources.

(d) Stabilizers and internal wiring should be capable of withstanding the expected increase in the temperature of apparatuses.

(e) Outdoor lighting apparatuses should have high efficiency.

(f) Luminous intensity distribution should be controlled appropriately in consideration of the use purposes of respective apparatuses.

② **Types of apparatuses**

Considering that wharves have a wide variety of outdoor facilities such as aprons, yards, roads, parking lots, parks, green areas, and squares, it is preferable to select lighting apparatuses that are appropriate for outdoor facilities. The main apparatuses of outdoor lighting facilities are as follows:

(a) Projectors

Projectors are apparatuses that have axisymmetric luminous intensity distribution to focus light on relatively narrow projection angles and are suitable for lighting wide areas. These apparatuses are installed on poles, steel towers, roofs, and walls of buildings. Some square projectors have asymmetric intensity distributions.

(b) Lighting apparatuses for roads

These lighting apparatuses are normally installed on poles with heights of 8 to 12 m.

(c) Others

There are other types of lighting apparatuses installed directly on or suspended from eaves of sheds.

③ **Structures of apparatuses**

Apparatuses shall have structures that facilitate the replacement, maintenance, and inspections of lamps; have opening and closing sections that are capable of being fastened by simple and reliable means; and are free from danger.

Sockets should have structures that can keep lamps from falling out or loosing connections even when apparatuses are subjected to vibrations. Furthermore, waterproof seals and internal wiring shall have resistance to temperature increase owing to heat radiation from apparatuses. It is also preferable that the materials and finishing of apparatuses, including poles and steel towers to which apparatuses need to be attached, have resistance to chloride-induced corrosion. Apparatuses installed in areas such as wharves that require particular attention to fires shall have explosion-proof structures that complying with the **Electrical Apparatus for Explosive Atmospheres in General Industry (JIS C 0903)**. Apparatuses with a risk of dazing passengers, workers, crews, or pilots with glare should have louvers or hoods to prevent glare.

④ **Waterproof structures of apparatuses**

Considering that apparatuses for outdoor lighting facilities are exposed to wind and rain, they should have waterproof structures, as stipulated in the **Test to Prove Protection against Ingress of Water and Degree of Protection (JIS C 0920)**.

⑤ **Outdoor lighting methods**

(a) Lighting method using high poles

This is a general lighting method for roads using 8 to 12 m poles with lighting apparatuses attached to them. This method requires a large number of poles when used for lighting wide areas such as parking lots and may hinder cargo handling work. Therefore, this method is suitable for small-scale parking lots, ferry boarding facilities, and places where no cargo handling work is executed.

(b) Method for lighting from high places

This is a method that uses structures with heights of 15 to 40 m and sizes larger than lighting poles so that wide areas can be illuminated with a small number of structures. This method is suitably used for yards, large-scale parking lots and other large areas with wide lighting ranges.

(c) Method for lighting from sheds

This is a method that uses buildings such as sheds if they exist near the places that require lighting in a manner that installs lighting apparatuses on roofs or side walls of buildings.

(d) Catenary lighting method

This is a method that uses overhead wires installed between poles or buildings at wide intervals (60 to 90 m) with lighting apparatuses suspended from the wires. This method requires a smaller number of poles than lighting method using high poles. Furthermore, this method enables lighting apparatuses to be moved closer to each other so that illumination distribution can be improved.

⑥ **Fig. 9.4.4** shows an example of the lighting method using high poles.

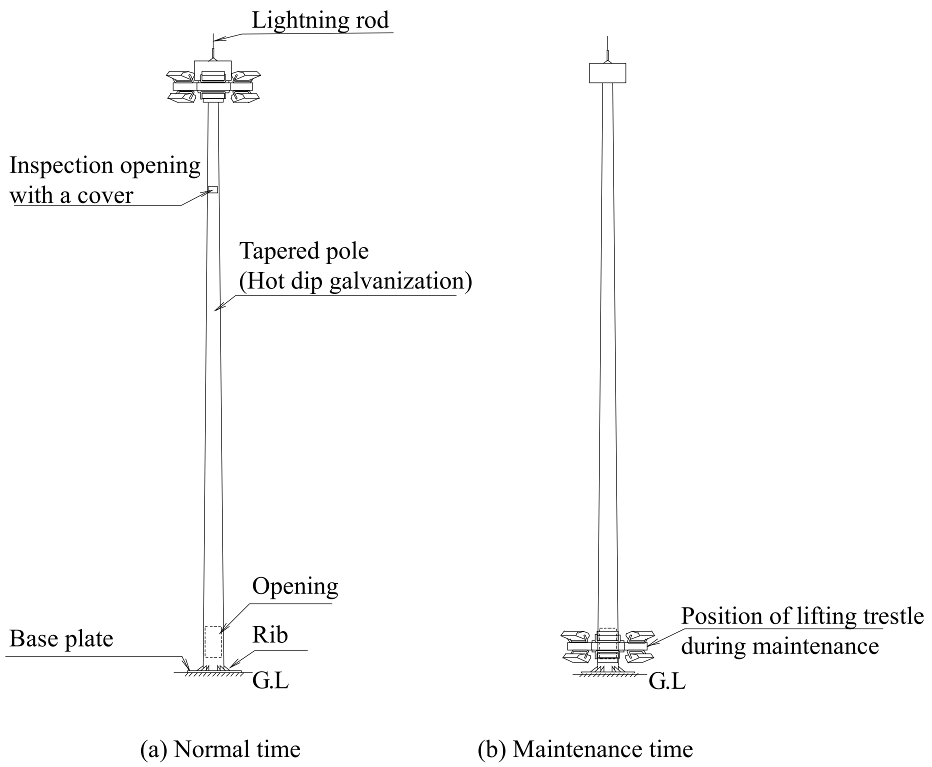


Fig. 9.4.4 Example of Lighting Method Using High Poles
(Lifting Type Mounted with Running Block and Balance Weight)

(2) Indoor lighting

① The selection of the apparatuses for indoor lighting should be made in consideration of the following requirements:

(a) Luminous intensity distribution should be controlled appropriately in consideration of the use purpose of respective apparatuses.

(b) Sockets should be compatible with respective light sources.

(c) Stabilizers and the internal wiring should be capable of withstanding the expected increase in the temperature of the equipment.

(d) Lighting apparatuses should have high efficiency.

② **Types of apparatuses**

General indoor lighting apparatus can be classified into ceiling mounted, ceiling embedded, and hanging types. Given that each type has axisymmetric or similar luminous intensity distributions, it is preferable to select appropriate apparatuses on the basis of their use purposes.

③ **Structures of apparatuses**

Apparatuses shall have structures that facilitate the replacement, maintenance, and inspections of lamps and are free from danger. Furthermore, stabilizers and internal wiring should be capable of withstanding the expected increase in the temperature of the apparatuses.

When selecting apparatuses for places where people rest, such as waiting lounges, it is also important to select apparatuses that are harmonized with buildings. Furthermore, apparatuses with resistance to corrosion are preferably selected for the types of sheds where apparatuses are subjected to corrosive gases.

④ **Indoor lighting methods**

(a) Method for lighting from ceilings

This method can be classified into the following three categories:

1) Ceiling mounted method

2) Ceiling embedded method

3) Hanging method

Among the three methods, the ceiling mounted method is the most economical and easiest method. Although the ceiling embedded method is economically disadvantageous, it is suitable for waiting lounges and paths where aesthetic value is important. The hanging method is suitable for sheds and warehouses with high ceilings.

(b) Floodlighting method

This method creates shadows behind cargoes, thus hindering cargo handling operations; therefore, is not suitable for these facilities.

9.4.10 Maintenance

(1) Inspections

① Inspections shall be periodically performed on the following:

(a) Lighting states

(b) Dirt on and damage to apparatuses

(c) Flaking states of paint

② Illumination intensity measurement should be periodically performed in a manner that selects plural measurement points at the typical places of respective facilities.

③ **Inspection frequency**

Although the inspection frequency shall be set by taking into consideration the types of lighting apparatuses, installation locations, meteorological, and oceanographic conditions, it is preferable to implement inspections at the following intervals depending on the inspection items:

(a) Lighting states: 1 month

(b) Other inspection items: 1 year

④ **Illumination intensity measurement points**

The standard illumination intensity is determined by obtaining the minimum value of average illumination intensity, and it requires a complex operation to confirm the standard illumination intensity. Therefore, the changes in illumination intensity are generally confirmed in a manner that preliminarily sets plural measurement points and monitors illumination intensity at these points.

⑤ **Illumination intensity measurement frequency**

It is preferable that illumination intensity is measured at the following intervals on the basis of the importance of facilities and the lengths of lighting time:

(a) Important facilities with long lighting times: 6 months

(b) Other facilities: 1 month

⑥ **Decline in light flux**

Light sources undergo a decline in light flux as lighting time advances. Therefore, lamps need to be replaced when light flux becomes lower than the design light flux. For design lumen maintenance factors, a reference can be made to literature 23).

(2) Cleaning and repair

① The lighting function of lighting facilities should be maintained via the implementation of repair and cleaning when inspections identify damage to lighting facilities, unlit lamps, and dirt on apparatuses.

② Given that dirt on the interior surfaces of lighting apparatuses reduces the illumination intensity on road surfaces, these apparatuses should be cleaned once they are identified to be dirty as a result of visual inspection or the measurement of illumination intensity.

③ Inspections should be implemented with particular focus on chloride-induced corrosion, and flaking paint should be repaired promptly.

④ It is necessary to identify the causes for the insulation failures of wiring and the failures in the control function of wiring devices and to remove such causes by performing repair to prevent unlit lamps.

⑤ As important data for future maintenance work, recoding ledgers should be prepared for respective installed lighting facilities to keep the records of structural types of lighting apparatuses, support columns, foundations, wiring devices, and management numbers of support columns. It is also preferable to keep the records of cleaning and repairs in the ledgers.

9.5 Staircases and Ladders

9.5.1 General

The intervals of staircases and ladders shall be appropriately set in accordance with the sizes and use states of facilities.

In facilities that are expected to be used by passengers when they embark on or disembark from ferries or other passenger ships, staircases are preferably installed at easily accessible places at one or more locations for every berth for emergency situations. Generally, staircases are preferably positioned at the anterior or posterior ends of respective wharves so that cargo handling work is not disturbed. Unlike ladders, staircases are used by passengers when they embark on and disembark from small ships or by workers when they load or unload small cargoes. Therefore, staircases need to have structures that ensure the safety of people. Ladders should also have structures that not only ensure the safety of passengers during their embarkation and disembarkation but also enable people in the water to easily climb up.

9.5.2 Performance Verification

(1) Staircases preferably have a width of 0.75 m or more, a rise of 20 cm, and a tread of 30 cm with concrete surfaces that are roughly finished.

(2) Generally, staircases should have landings. Staircases installed in areas with large tidal levels should have landings at shorter intervals than normal cases. The length of landings can be 1.5 m.

(3) When installing ladders on mooring facilities, ladders should be installed at the transition positions between berths so that the mooring of ships will not be disturbed. Generally, installation spaces are prepared by cutting out the portions of concrete superstructures of mooring facilities in the shapes of vertical trenches with a width of 75 cm and a depth of 30 cm, and ladders are placed in the trenches with a distance of approximately 20 cm from the cut concrete surfaces. Each spoke should have a downward gradient across it to prevent people from slipping and also have a one-way drainage gradient with a throating.

(4) It is preferable that spokes have a width of 45 cm and a vertical interval of 30 cm, with the lowermost spoke positioned below the L.W.L. Banisters are preferably embedded in wharves and extended 30 cm above the tops of mooring facilities and 45 cm inside normal lines or special jigs to ensure the safety of people.

(5) Generally, the surcharges on ladders shall be 1 kN per 1 m in both the vertical and horizontal directions. Mounting brackets should have special resistance to corrosion due to chloride damage and structures facilitating the repair of damaged or corrosive ladders when needed.

(6) The aprons at the foot of staircases and ladders should be protected from cargo handling machines by curbings and the like. In some cases, mooring posts or rings for small ships are used in place of curbings.

(7) There are two types of accommodation ladders: metallic and rubber. The selection of accommodation ladders shall be made by taking into consideration durability and resistance to berthing force because these ladders are subjected to corrosion and contact with small ships.

9.6 Lifesaving Facilities

[Public Notice] (Performance Criteria of Lifesaving Facilities)

Article 63

The performance criteria of lifesaving equipment shall be such that appropriate lifesaving equipment is provided and readily available as necessary so as to secure the safety of human beings on the mooring facilities to serve for passenger ships with gross tonnage equal to or larger than 500 tons.

9.6.1 General

(1) Lifesaving facilities refer to life rings and small ships.

(2) The types, shapes, installation locations, and materials of lifesaving facilities should be appropriately set to ensure the safety of users in accordance with the use conditions and structural characteristics of mooring facilities.

9.7 Curbings

[Public Notice] (Performance Criteria of Curbing)

Article 64

The performance criteria of curbing shall be as prescribed respectively in the following items:

(1) The curbing shall be installed at appropriate locations and shall be provided with the dimensions necessary for ensuring the safe use of mooring facilities while not hindering ship mooring and cargo handling in consideration of the structural types and usage conditions of the mooring facilities.

(2) The risk of impairing the integrity of curbing shall be equal to or less than the threshold level under the variable situation, in which the dominating action is collision of vehicles.

[Interpretation]

11. Mooring Facilities

(15) Performance Criteria of Curbing (Article 33 of the Ministerial Ordinance on Criteria and the interpretation related to Article 64 of the Public Notice)

The required performance of curbing under the variable action situation in which the dominant action is the impact of vehicles should focus on serviceability. **Attached Table 11-32** shows the performance verification items and standard indexes to determine limit values with respect to the action.

In the performance verification of curbing, the standard indexes to determine limit values with respect to their soundness shall be appropriately set in accordance with materials and the like.

Attached Table 11-32 Performance Verification Items and Standard Indexes to Determine Limit Values under Respective Design Situations (excluding accidental situation) of Curbing

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
33	1	2	64	–	2	Serviceability	Variable	Impact of vehicle	–	Soundness of curbing	–

9.7.1 General

(1) The structures, shapes, layouts, and materials of curbing should be set properly in such a way that the safety of users is ensured and that cargo handling work is not hindered in consideration of the structural characteristics and use conditions of mooring facilities.

(2) The materials generally used for curbing are concrete (with steel plate cladding), concrete (with cast iron cladding), prestressed concrete, resin concrete, synthetic resin, and square steel pipes.

(3) It is preferable to take comprehensive hazard prevention measures in a manner that combines curbings with warning signs, road markings, and barricades as needed. Particular attention is required for hazard prevention at the edges and boundaries of berths where vehicles are at high risk of falls.

(4) The locations and dimensions of curbings should be appropriately set to ensure the safety of users, smooth ship mooring, and cargo handling in accordance with the structures and use conditions of mooring facilities.

9.7.2 Performance Verification

(1) The distance intervals between curbings need to be shorter than the wheel treads of the cargo handling equipment and vehicles. They may be set at approximately 30 cm in general to drain rainwater from the aprons. However, it is preferable to set the intervals of curbing installed at both side of mooring posts at 1.5–2.5 m. In cases wherein vehicles are not expected to pass because fences or other barriers are set up to prohibit the passage of vehicles, there is no need to install curbing.

(2) It is preferable that the heights of curbing are separately set for dangerous and general zones depending on the degree of dangers due to vehicle falls (refer to **Table 9.7.1** and **Fig. 9.7.1**).

Table 9.7.1 Heights of Curbing

Degree of Danger	Height of curbing	Example
Dangerous zone	25 to 30 cm	Berth edges and berth boundary
General zone	15 to 20 cm	Areas other than dangerous zones

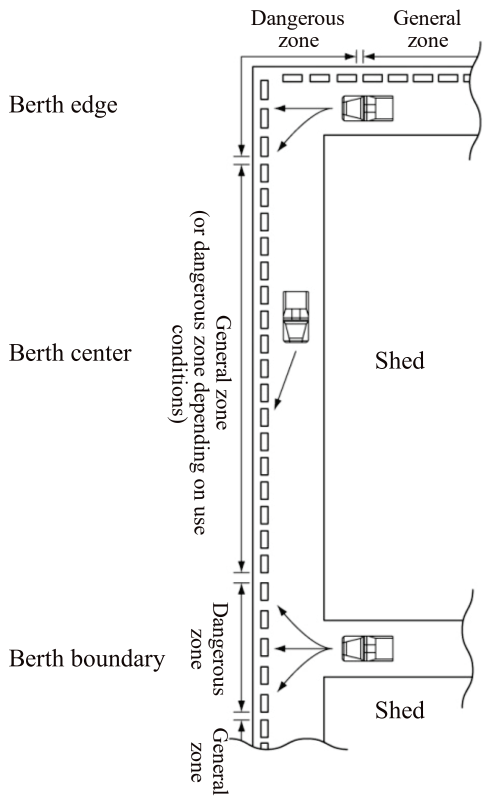


Fig. 9.7.1 Dangerous and General Zones on Passages

(3) Curbing is preferably installed at places approximately 10 cm landward from the face lines of wharves.

(4) The performance verification of curbing should be made by taking into consideration durability and visibility. Furthermore, refer to the **Curbing Design Manual**.[24)]

(5) Some curbing is installed with spaces between their bottom faces and road surfaces for the purpose of draining rainwater.

9.8 Vehicle Loading Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.8.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.9 Water Supply Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.9.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.10 Drainage Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.10.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.11 Fueling Facilities and Electric Power Supply Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.11.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.12 Passenger Boarding Facilities

(English translation of this section from Japanese version is currently being prepared.)

9.12.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.13 Fences, Doors, Ropes, etc.

(English translation of this section from Japanese version is currently being prepared.)

9.13.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.14 Monitoring Equipment

(English translation of this section from Japanese version is currently being prepared.)

9.14.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.15 Rest Rooms

(English translation of this section from Japanese version is currently being prepared.)

9.15.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.16 Signs

(English translation of this section from Japanese version is currently being prepared.)

9.16.1 Placement of Signs and Marks

(English translation of this section from Japanese version is currently being prepared.)

9.16.2 Forms and Installation Sites of Signs

(English translation of this section from Japanese version is currently being prepared.)

9.16.3 Installation Locations of Signs

(English translation of this section from Japanese version is currently being prepared.)

9.16.4 Structure of Signs

(English translation of this section from Japanese version is currently being prepared.)

9.16.5 Raw Materials

(English translation of this section from Japanese version is currently being prepared.)

9.16.6 Maintenance and Management

(English translation of this section from Japanese version is currently being prepared.)

9.16.7 Guard Fences

(English translation of this section from Japanese version is currently being prepared.)

9.16.8 Barricades

(English translation of this section from Japanese version is currently being prepared.)

9.17 Fire-fighting Equipment and Alarm Systems

(English translation of this section from Japanese version is currently being prepared.)

9.17.1 General

(English translation of this section from Japanese version is currently being prepared.)

9.18 Aprons

[Public Notice] (Performance Criteria for Aprons)

Article 73

The performance criteria for aprons shall be as prescribed respectively in the following items:

(1) Aprons shall be provided with the dimensions necessary for enabling the safe and smooth cargo handling operations.

(2) The surface of aprons shall be provided with the gradient necessary for draining rainwater and other surface water.

(3) Aprons shall be paved with appropriate materials in consideration of surcharge loads and the usage conditions of the mooring facilities.

(4) The risk of incurring damage to the pavement to the extent of affecting cargo handling operatons shall be equal to or less than the threshold level under the variable situation in which the dominating action is surcharge load.

[Interpretation]

11. Mooring Facilities

(17) Performance Criteria of Aprons (Article 33 of the Ministerial Ordinance and the interpretation related to Article 73 of the Public Notice)

① The required performance of aprons shall be usability. Here "usability" means the following.

a) Apron widths shall be properly set to ensure safe and smooth cargo handling.

b) Apron gradients shall be properly set to drain rainwater and other surface water.

c) Aprons shall be paved with proper materials taking account of the surcharges and the use conditions of mooring facilities.

② In addition to the above, the required performance of aprons, under the variable situation in which the dominating action is surcharge, shall be serviceability. The performance verification items and standard indexes to determine limit values with respect to the action shall be as shown in Attached Table 11-34. In the performance verification of apron pavement, the standard indexes to determine limit values with respect to the soundness shall be appropriately set in accordance with materials and the like.

Attached Table 11-34 Performance Verification Items and Standard Indexes to Determine Limit Values under Respective Design Situations (Excluding Accidental Situation) of Aprons

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
33	1	2	73	-	4	Serviceability	Variable	Surcharge	–	Soundness of pavement	–

9.18.1 General

The performance verification of aprons shall be carried out in terms of both dimensions and pavements.

9.18.2 Dimension of Aprons

(1) Apron widths

① The apron widths of ordinary mooring facilities may generally refer to the values shown in Table 9.18.1.

Table 9.18.1 Apron Widths

Berth water depth (m)	Apron width (m)
Less than 4.5	10
4.5 or more and less than 7.5	15
7.5 or more	20

② However, because required apron widths of container wharves and internal trade unit load terminals vary depending on the types of cranes, ships, and cargo handling methods, the widths shall be properly set, taking account of the traveling widths of cargo handling equipment and trailers as well as the actual situation of cargo handling operation.

③ The determination of apron widths of general cargo wharves shall normally take account of the spaces for cranes, temporary storage, cargo handling, and traffic paths. It is preferable that aprons have the widths of not less than 15 to 20 m when sheds are installed at the back and fork lifts are used, and not less than 10 to 15 m when roads and open storage yards are in the immediate vicinity and trucks are allowed to drive into the aprons for cargo handling operations directly to and from general cargo ships.

(2) Apron gradients

① Aprons are where cargo handling is performed and closely related to the conditions of cargo handling operation at the backyards, and hence transverse slopes need to be properly determined taking these conditions into consideration.

② Aprons normally have a down slope of 1 to 2% toward the sea. Shallow draft wharves have steep slopes. Aprons in snowy places often have relatively steep slopes for the easy removal of snow. In some cases, reverse slopes are used depending on the conditions of use of aprons and environmental consideration.

③ Since the settlement of backfilling may cause slopes to be reversed, construction should be carefully performed.

(3) Countermeasures against apron settlement

① Appropriate countermeasures shall be taken to prevent settlement due to sand washing-out or consolidation of lower landfill materials from hindering cargo handling operation and the vehicle traffic on aprons.

② In general, apron pavements are at risk for settlement due to the consolidation of the layers below the subgrade of the apron pavements. There are many other cases of settlement caused by the washing-out of landfill soil used as a part of the layers below the subgrade through the joints of quaywalls or compression of backfill materials. In many cases, settlement is considered to be the main reason for the failures of apron pavements.[28)] Therefore, it is preferable to consider preventive measures against the settlement of the layers below the subgrade such as sand washing-out prevention work and compaction of backfilling materials. There may be the cases where aprons are placed into service with temporary pavements until settlement subsides and full-scale pavements are implemented or aprons are provided with block pavements so as to facilitate future maintenance.

③ When aprons undergo settlement to such an extent that hinders cargo handling or vehicle traffic, repairs including leveling work shall be implemented. In the case of leveling work by constructing granular base courses on existing pavements or asphalt mixture layers on existing concrete pavements, proper drainage measures shall be taken against possible adverse impact of accumulated water due to the low permeability of existing pavements on leveling work.

④ Special attention is required when carrying out performance verification and executing paving work in the case where fluctuations of tide levels may cause the elevation of ground water levels higher than the subgrade of apron pavements.

9.18.3 Performance Verification

(1) General

① The types of apron pavements shall be properly selected in a comprehensive judgment taking account of the soil property below the subgrade, constructability, surrounding pavement conditions, cargo handling methods, economic efficiencies, and maintenance.

② The types of apron pavements include concrete, asphalt, semi-flexible, block, reinforced concrete, continuous reinforced concrete, prestressed concrete (PC), and interlocking block pavements. In addition, colored pavement is another type used when aesthetic property is of importance. The former two types of pavements have been often used for the aprons of mooring facilities, and the performance verification of these two types can be referred to **Part II, Chapter 5, 9.18.3 (4) Performance verification of concrete pavement** and **9.18.3 (5) Performance verification of asphalt pavement**.

③ Because there is no uniform way of selecting an optimal type of pavement, it shall be appropriately determined taking into consideration a variety of factors including the use conditions of aprons. When a concrete pavement is desirable but site conditions or other factors do not allow it to be constructed, the alternative choice is preferably a semi-flexible pavement or an asphalt pavement using special asphalt having dynamic stability equivalent to a semi-flexible mixture.

④ The characteristics of respective types of pavements are as follows.

(a) Concrete pavement

1) The advantages of concrete pavements

i. Concrete enables sufficient pavement structures to be constructed without being much affected by the bearing capacity of subgrade and its heterogeneous property and thereby reducing the thicknesses of base courses.

ii. Concrete pavements are extremely resistant to concentrated loads with large contact pressure and, therefore, advantageous to the use of truck cranes with outriggers.

iii. Concrete slabs have high durability and thereby extending the service life.

iv. Concrete surfaces have high abrasion resistance suitable for the scratching of cargo handling equipment.

2) The disadvantages of concrete pavements

i. Concrete pavements require a considerable number of days for curing after the completion of construction till the pavements are finally placed in service.

ii. Concrete slabs become difficult to repair once they start to be destructed, and demolition of destructed slabs requires large efforts.

iii. Even in the case of uneven settlement of the layers below the subgrade of pavements, concrete slabs hardly reflect the occurrence of such an incidence, and, therefore, even minor settlement can cause major destruction.

iv. There have been cases of pavement destruction induced by the contact between pavements and various structures.

(b) Asphalt pavement

1) Advantages of asphalt pavement

i. Paving work can be easily implemented in a phased manner and in accordance with the availability of construction investment. The phased implementation is advantageous to enhance consolidation settlement and strengthen subgrade while using aprons before implementing final paving.

ii. Asphalt pavements can alleviate the adverse effects of uneven settlement in the layers below subgrade to some extent without deteriorating serviceability.

iii. Asphalt pavements require a very short curing period, thereby enabling aprons to be used immediately after construction.

iv. Asphalt pavements are easy to be repaired.

2) Disadvantages of asphalt pavements

i. The service life of asphalt mixtures is relatively short.

ii. Asphalt pavements are weak against static loads with large contact pressure and repetitive loads applied to identical locations and are thereby being subjected to asperity and rutting.

iii. Asphalt pavements are susceptible to oil and heat and therefore require, for example, oil resistance surface treatment for the place with the risk of oil leaks.

iv. Asphalt pavements require complex construction management.

(c) Semi-flexible pavement

1) Advantages of semi-flexible pavements

i. Semi-flexible pavements are superior to asphalt pavements in terms of fluidity, oil and flame resistance.

ii. Semi-flexible pavements require shorter curing periods than concrete pavements.

2) Disadvantages of semi-flexible pavements

i. Semi-flexible pavements have shorter service life than concrete pavements and a risk of fine contraction cracks after construction due to the shrinkage when cement milk hardens or the fluctuations in external temperature.

ii. Semi-flexible pavements require longer curing periods than asphalt pavements because open-graded asphalt mixtures need to be constructed first before impregnating cement milk with them for final curing.

(d) Block pavements

1) Advantages of block pavements

i. Block pavements can alleviate the effects of uneven settlement in the layers below subgrade to some extent.

ii. The damage to block pavements due to settlement can be easily repaired at low costs.

iii. Block pavements enable aprons to be used immediately after construction.

2) Disadvantages of block pavements

i. The joints between blocks are subjected to damage and cause deterioration of vehicles' traveling performance.

ii. Block pavements require complex construction procedures.

(e) Reinforced concrete pavements

Reinforced concrete pavements have a mechanism to close cracks generated in concrete slabs with reinforcement bars arranged inside the slabs and to enable the interlocking effects of aggregates between crack surfaces to transfer loads. Although reinforced concrete pavements have larger strength than unreinforced concrete pavements, the slab thicknesses are normally identical in both cases.

(f) Continuous reinforced concrete pavements

Continuous reinforced concrete pavements use continuous longitudinal reinforcement bars to eliminate transverse joints on concrete slabs in a manner that controls the distribution of cracks in the transverse direction by the longitudinal reinforcement so as to reduce the crack widths of respective cracks. The thicknesses of concrete slabs are normally 80% to 90% of those of unreinforced concrete slabs.

(g) Prestressed concrete pavements

Prestressed concrete pavements achieve improved structural strength in a manner that reduces tensile stress generated in slabs by preliminarily applying compressive stress to concrete slabs.

(h) Interlocking block pavements

Interlocking block pavements are a type of block pavements and use high-strength concrete products formed into deformed blocks instead of rectangle blocks. The interlocking block pavement method can reduce deformation of pavement surfaces through the interlocking effects among blocks and has been frequently used for the aprons in many ports overseas.

⑤ The pier slabs normally have pavements to ensure flat traveling surfaces that can resist vehicle (traffic) loads and to prevent abrasion due to loads on the pier slabs. The pavements on pier slabs are classified into concrete pavement slabs placed directly on pier slabs and asphalt pavements comprising a base course and a surface course. Concrete pavement slabs are further classified into a single layer type where structural pier slabs and pavement slabs are cast simultaneously as integrated slabs with final surface finishing and a double layer type where pavement concrete is cast after the concrete of pier slabs hardens. It is necessary to pay attention to the surface finishing in the case of the single layer type and to the cracks on pavement concrete in the case of the double layer type.[29)]

(2) Fundamentals of performance verification

① The performance verification of apron pavements shall be generally such that pavement structures are stable under the surcharges by cargo handling vehicles and related equipment.

② **Fig. 9.18.1** shows an example of the performance verification procedures of apron pavements.

③ For the performance verification of apron pavements, reference can be made to the **Pavement Design and Construction Guide**[30)]; the **Manual for Pavement Design**[31)]; the **Manual for Airport Pavement Design**[32)]; and the **Manual for Airport Pavement Rehabilitation**[33)].

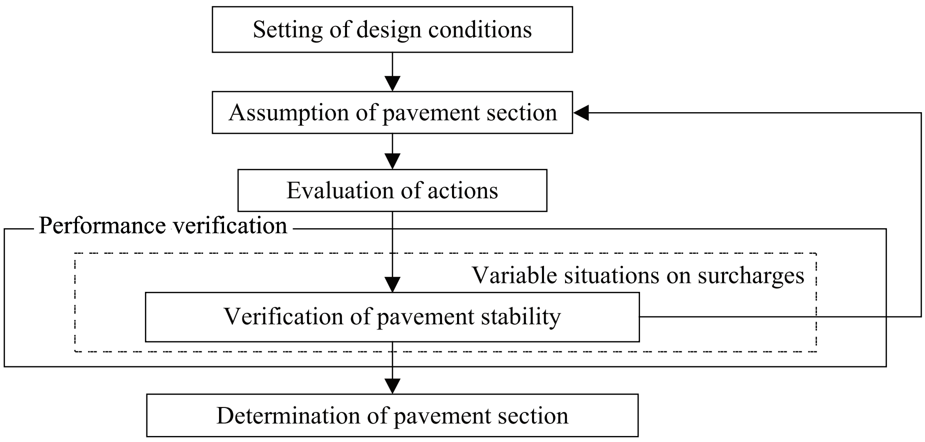


Fig. 9.18.1 Example of Procedures for the Performance Verification of Apron Pavements

(3) Actions

① Actions to be considered in the performance verification of apron pavements are generally the surcharges by mobile cranes (truck cranes, rough terrain cranes, and all terrain cranes), trucks, tractor trailers, fork lift trucks, straddle carriers, etc., depending on the types of cargoes and cargo handling methods. Generally, the performance verification of apron pavements shall be carried out for the maximum surcharges and the ground contact pressures that maximize the pavement thicknesses on the basis of the ground contact areas on which surcharges are applied.

② The characteristic values of the surcharges used for the verification of apron pavements may refer to **Table 9.18.2**.[34)] Outriggers are applied to the cases of movable cranes, where a wheel means a single wheel or dual wheels (two laterally connected wheels). In the cases where the loads of actually used cargo handling equipment can be precisely set, this table may not be used.

③ The values for mobile cranes in **Table 9.18.2** are determined in consideration of the relations between lifting capacity and maximum outrigger loads as well as between maximum outrigger loads and contact areas per outrigger with reference to the actual performance of existing mobile cranes of leading producers, where 95% confidence values and averages are used for the outrigger loads and contact areas, respectively. The ground contact pressure in the table means the values obtained by dividing outrigger loads by contact areas. The values for folk lift trucks are determined in consideration of the relations between loading capacity and maximum loads per wheel as well as between maximum loads per wheel and ground contact pressure, where the maximum loads, contact areas, and ground contact pressure are set in the same ways as is the case for outrigger loads. The values for trucks, tractor trailers, and straddle carriers are determined in accordance with actual performance values of respective machines.

Table 9.18.2 Characteristic Values of the Actions Considered in the Performance Verification of Apron Pavements

Type of action (cargo handling equipment load)		Maximum load of an outrigger or a wheel (kN)	Ground contact area of an outrigger or a wheel (cm^2)	Ground contact pressure (N/cm^2)
Mobile cranes	Type 20	220	1,250	176
Truck crane	Type 25	260	1,300	200
Rough terrain crane	Type 30	310	1,400	221
All terrain crane	Type 40	390	1,650	236
	Type 50	470	1,900	247
	Type 80	690	2,550	271
	Type 100	830	3,000	277
	Type 120	970	3,350	290
	Type 150	1170	3,900	300
Truck	25 ton class	100	1,000	100
Tractor trailer	For 20 ft	50	1,000	50
	For 40 ft	50	1,000	50
Fork lift truck	2 ton	25	350	71
	3.5 ton	45	600	75
	6 ton	75	1,000	75
	10 ton	125	1,550	81
	15 ton	185	2,250	82
	20 ton	245	2,950	83
	25 ton	305	3,600	85
	35 ton	425	4,950	86
Straddle carrier		125	1,550	81

(4) Performance verification for concrete pavements

① **Compositions of concrete pavements**

As shown in **Fig. 9.18.2**, concrete pavements generally have a cross-sectional structure where a base course and a concrete slab are arranged on subgrade. A base course and a concrete slab are collectively called a pavement.

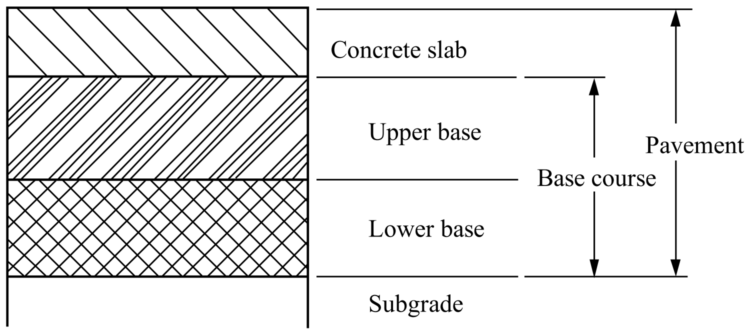


Fig. 9.18.2 Composition of Concrete Pavement

② **Procedures of performance verification**

(a) **Fig. 9.18.3** shows an example of the procedures of the performance verification for concrete pavements.

(b) It is preferable that the performance verification of concrete pavements is carried out for the thicknesses of base courses and concrete slabs in consideration of action conditions, the number of repetitions of actions, and the conditions of bearing capacity of subgrade.

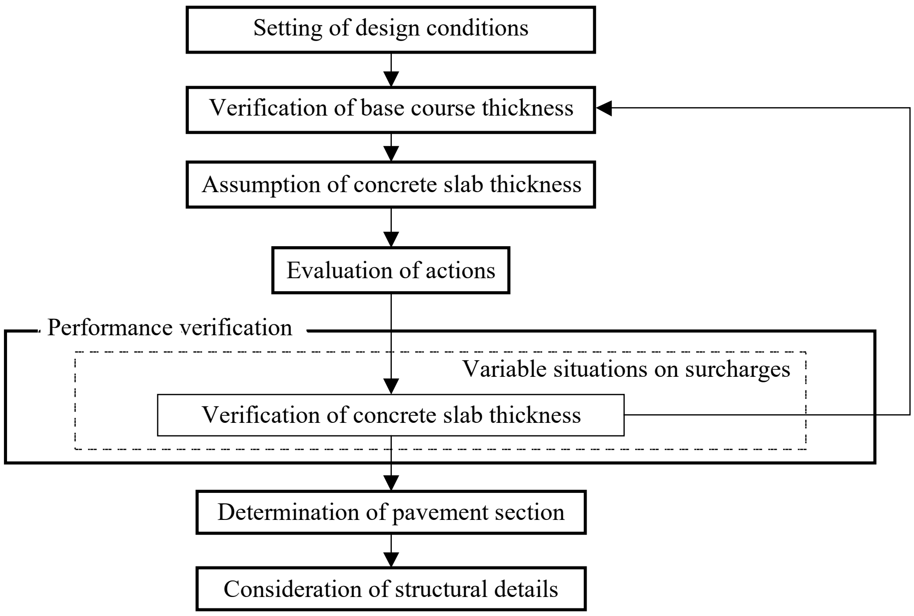


Fig. 9.18.3 Example of the Procedures of Performance Verification for Concrete Pavements

③ **Design conditions**

(a) The general design conditions to be considered in the performance verification are as follows:

1) design working life;
2) action conditions;
3) the number of repetitions of actions;
4) bearing capacity of subgrade; and
5) materials used.

(b) Design working life

The design working life of concrete pavements shall be properly set considering the conditions of use and other related conditions of mooring facilities. The design working life of concrete pavements used for the aprons of quaywalls and other facilities may be generally set at 20 years.

(c) Action conditions

The design action conditions are those requiring the maximum concrete slab thicknesses among the types of actions to be considered. The characteristic values of actions may be set referring to Table 9.18.3. The "Action classification" in Table 9.18.3 is the classification needed when using the empirical method in **Chapter 5, 9.18.3, (4) ④ (d) Empirical method of setting concrete slab thickness**.

Table 9.18.3 Reference Values for the Action Conditions of Concrete Pavements used for the Aprons of Quaywalls and Other Facilities

Action classification	Type of action		Action (kN)	Ground contact radius (cm)
CP_1	Fork lift truck	2 t	25	10.6
	Tractor trailer	for 20 ft, 40 ft	50	17.8
	Fork lift truck	3.5 t	45	13.8
CP_2	Fork lift truck	6 t	75	17.8
CP_3	Truck	25 ton class	100	17.8
	Fork lift truck	10 t	125	22.2
	Straddle carrier		125	22.2
	Fork lift truck	15 t	185	26.8
CP_4	Mobile crane (truck crane, rough terrain crane, all terrain crane)	Type 20	220	19.9
	Fork lift truck	20 t	245	30.7
	Mobile crane (truck crane, rough terrain crane, all terrain crane)	Type 25	260	20.3

(d) Calculation of the number of repetitions of actions

The methods for calculating the number of repetitions of surcharges during design working life is considered to be as follows:

1) the estimation based on the actual performance of other ports with similar scales and

2) the estimation based on the cargo throughput of the port concerned.

One of the references to the method 2) for estimating the number of repetitions of surcharges based on cargo throughput[34)] is the method proposed by Nagao et al. for calculating the number of repetitions of loads for the performance verification of the superstructures of piled piers in the fatigue limit state.[35)]

(e) Bearing capacity of subgrade

In the performance verification of concrete pavements, the bearing capacity of subgrade can be set on the basis of a design bearing capacity coefficient K_{30}.

1) The design bearing capacity coefficient K_{30} of subgrade can be set in accordance with the **Method for Plate Load Test on Soil for Road (JIS A 1215)**. Here, the design bearing capacity coefficient K_{30} is generally set as the value corresponding to a settlement of 0.125 cm. It is preferable to perform the plate loading test at one or two locations per 50 m along the face line directions of quaywalls.

2) When setting the design bearing capacity coefficients K_{30} in areas of subgrade made of identical materials, it is preferable to calculate the values of K_{30} with **equation (9.18.1)** using the measured values of three or more points excluding extreme values.

Bearing capacity coefficient K_{30} of subgrade =

$$\text{The average of bearing capacity coefficients at respective locations.} - \frac{\text{The maximum value of bearing capacity coefficient} - \text{The minimum value of bearing capacity coefficient}}{C} \qquad \textbf{(9.18.1)}$$

where

C : a coefficient used for calculating the bearing capacity coefficients. The values in **Table 9.18.4** may be used.

Table 9.18.4 Reference Values for Coefficient *C*

Number of test points (n)	3	4	5	6	7	8	9	10 or more
C	1.91	2.24	2.48	2.67	2.83	2.96	3.08	3.18

3) When the subgrade has already been constructed, the bearing capacity coefficients can be obtained by performing plate load tests on the subgrade when it has the highest moisture content. In the case where plate load tests cannot be performed under such a condition, the test results are subjected to correction, when calculating the bearing capacity coefficients, using **equation (9.18.2)** in which California Bearing Ratio (*CBR*) test results shall be those of undisturbed test pieces.

$$\text{Bearing capacity coefficient (corrected) of subgrade} = \text{Calculated bearing capacity coefficient using actual measured values} \times \frac{CBR \text{ (with immersion in water for 4 days)}}{CBR \text{ (with natural moisture content)}} \qquad \textbf{(9.18.2)}$$

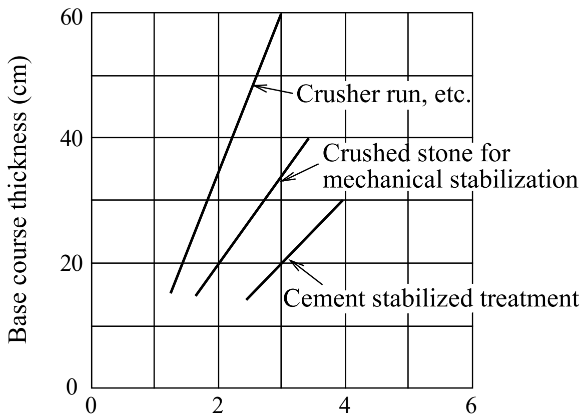
(f) Materials used

The requirements for the quality and grain sizes of respective base course materials can follow the provisions in the **Pavement Design and Construction Guide**.[30)]

④ **Performance verification**

(a) Verification of base course thicknesses

1) It is preferable to obtain the thicknesses of base courses in a manner that prepares test base courses and identifies the thicknesses that achieve the bearing capacity coefficient of 200 N/cm^3. In the cases where the preparation of test base courses is difficult, the base course thicknesses may be directly set using the design curves shown in **Fig. 9.18.4.** The minimum base course thickness is generally set at 15 cm.



$$\frac{\text{Bearing capacity coefficient of base course}}{\text{Design bearing capacity coefficient of subgrade}} = \frac{K_1}{K_2}$$

K_1 is the bearing capacity coefficient of base course K_{30} (200 N/cm^3)

K_2 is the bearing capacity coefficient of subgrade K_{30}

Fig. 9.18.4 Design Curves for Base Course Thicknesses[30)]

2) The base course thicknesses of concrete pavements may be set referring to **Table 9.18.5** that is prepared on the basis of the past records.

Table 9.18.5 Reference Values for Base Course Thicknesses of Concrete Pavements

Design condition	Base course thickness (cm)				
Design bearing capacity coefficient of subgrade K_{30} (N/cm^3)	Upper base		Lower base		Total base course thickness
	Cement stabilized base	Mechanical stabilized material	Mechanical stabilized material	Crusher run, etc.	
50 or more and less than 70	–	40	–	20	60
	20	–	20	–	40
	25	–	-	30	55
70 or more and less than 100	–	20	15	-	35
	–	20	–	20	40
	15	–	15	-	30
	15	–	–	15	30
100 or more	-	20	–	–	20
	15	–	–	–	15

(b) Verification of concrete slab thicknesses

1) Bending strength of concrete slabs

The bending strength of concrete slabs may be generally set at 450 N/cm^2 on the basis of specimens with the age of 28 days. The strength can be set with reference to the **Method of Making and Curing Concrete Specimens (JIS A 1132)** and the **Method of Test for Bending Strength of Concrete (JIS A 1106)**. Using concrete with bending strength enhanced by lowering water–cement ratios is one of the methods to minimize the increase in concrete slab thicknesses. However, it is necessary to pay attention to the fact that lower water–cement ratios may cause the reduction in workability and the increase in the risk of drying shrinkage cracks.

2) **Fig. 9.18.5** shows the relation between concrete slab thicknesses and bending stresses. The bending stresses are calculated using an **equation (9.18.3)** called Picket's formula or Arlington formula. The partial factor in the equation is set at 1.0. The symbols CP_1 to CP_4 in **Fig. 9.18.5** are the classification needed when using the empirical method in **Part II, Chapter 5, 9.18.3, (4) ④ (d) Empirical method of setting concrete slab thickness**.

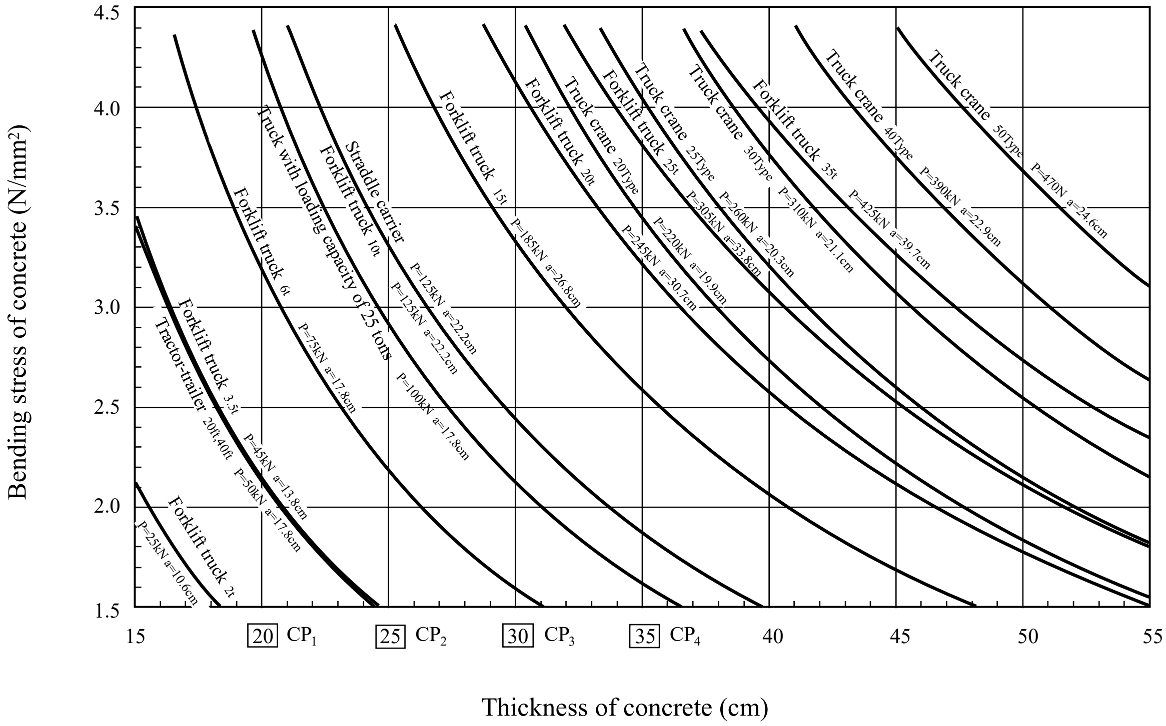


Fig. 9.18.5 Relation between Concrete Slab Thickness and Bending Stress

$$\sigma = \frac{10CP}{h^2}\left(1 - \frac{\sqrt{\dfrac{a}{l}}}{0.925 + 0.22\dfrac{a}{l}}\right) \qquad \textbf{(9.18.3)}$$

where

σ : the maximum stress at a right angle corner of a concrete slab (N/mm^2);

C : a coefficient that can be set at 3.36 when using dowel bars;

P : a weight of a cargo handling machine (kN);

h : a thickness of a concrete slab (cm);

a : a radius of equivalent contact area of the cargo handling machine (cm);

l : a radius of relative stiffness (cm); $l = \sqrt[4]{\dfrac{Eh^3}{12(1-\nu^2)K_{75}}}$

E : the modulus of elasticity of concrete (N/cm^2) that can be normally set at 3,500,000 N/cm^2;

ν : the Poisson ratio of concrete that can be normally set at 0.15; and

K_{75} : the design bearing capacity coefficient of a base course (N/cm^3) that is normally set at $K_{75} = K_{30}/2.8 = 200/2.8 \approx 70$ N/cm^3 on the basis of $K_{30}/K_{75} = 2.8$.

(c) Setting of concrete slab thicknesses

The method of setting the thicknesses of concrete slabs in compliance with the concept of the **Pavement Design and Construction Guide**[30] has been proposed.[34] In this method, the fatigue characteristics of concrete slabs are calculated on the basis of the wheel load stresses imposed on concrete slabs and the number of repetitions of the stresses during design working life. And the relation between the above-mentioned characteristics and the degree of fatigue as a failure criterion is proposed to set the thicknesses of concrete slabs. An outline of the method is shown below:

1) The allowable number of repetitions of wheel load stresses is calculated by the fatigue **equation (9.18.4)**.

$$
\begin{aligned}
N_i &= 10^{\{(1.000-SL)/0.044\}} && 1.0 \geq SL > 0.9 \\
N_i &= 10^{\{(1.077-SL)/0.077\}} && 0.9 \geq SL > 0.8 \\
N_i &= 10^{\{(1.224-SL)/0.118\}} && 0.8 \geq SL
\end{aligned}
\qquad \textbf{(9.18.4)}
$$

where

N_i : an allowable number of wheel load stress i imposed on concrete slab; and

SL: wheel load stress/design reference bending strength (= 450 N/cm^2) with the value of the wheel load stress calculated from **equation (9.18.3)**.

2) Calculation of the degrees of fatigue

The degree of fatigue of a concrete slab is calculated from **equation (9.18.5)**.

$$
FD = \sum \left(\frac{n_i}{N_i} \right) \qquad \textbf{(9.18.5)}
$$

where

FD : a degree of fatigue;

n_i : the number of repetition of wheel load i; and

N_i : the allowable number of repetition of a wheel load i imposed on a concrete slab.

3) Setting of concrete slab thicknesses

Using the degree of fatigue as the failure criterion of concrete slabs, concrete slab thicknesses are set so that the degree of fatigue FD is equal to 1.0 or less.

(d) Empirical method of setting concrete slab thicknesses

1) The concrete slab thicknesses set referring to the empirical values given in **Table 9.18.6** may be considered to have the same performance as the one set using the method stipulated in **Part II, Chapter 5, 9.18.3 (4) ④ (c)** Setting Concrete Slab Thickness. However, the thicknesses are preferably set in accordance with the method stipulated in **Part II, Chapter 5, 9.18.3 (4) ④ (c) Setting Concrete Slab Thickness** in the case of the performance verification of the concrete pavements subjected to large cargo handling machines.

Table 9.18.6 Reference Values for Concrete Slab Thickness

Action classification	Concrete slab thickness (cm)
CP_1	20
CP_2	25
CP_3	30
CP_4	35
Applied to piled pier slab	10

2) The "Action classification" in **Table 9.18.6** corresponds to the one given in **Table 9.18.3**. It should be noted, in classifying actions, that there are cases where the maximum loads are not equivalent to the values shown in **Table 9.18.2**. In such cases, the classifications with the values closest to and larger than the ones in **Table 9.18.2** are used. For example, a truck crane with the maximum load per outrigger of 120 kN and a forklift truck with the maximum load per wheel of 64 kN can be considered as the actions of type 20 truck crane and 6 ton fork lift truck, respectively.

3) For the loads plotted at the right side of a curve of type 25 truck crane in **Fig. 9.18.5**, it is preferable to separately carry out the performance verification of the concrete slab thicknesses using, for example, **equation (9.18.3)**.

4) When setting concrete slab thicknesses, for design loads exceeding CP_4, with reference to the values in **Table 9.18.6**, it is preferable to study the possibility of using prestressed concrete pavements or continuous reinforced concrete pavements because such large design loads significantly increase the slab thicknesses of non-reinforced concrete pavements. When using cranes such as truck cranes having larger ground contact pressure than other cargo handling machines on aprons, it is also preferable to lay iron plates or the like under outriggers to reduce the ground contact pressure per unit area.

⑤ **Structural details**

(a) Frost heave prevention layers

In the cold regions where pavements are subjected to freezing and thawing, it is necessary that pavements are provided with frost heave prevention layers in the cases of pavement thicknesses less than frost penetration depths.

(b) Wire meshes

1) It is effective to place wire meshes in concrete slabs in terms of crack prevention. It is preferable that wire meshes are made of deformed steel bars with a diameter of 6 mm and arranged in concrete slabs at a rate of about 3 kg/m^2.

2) It is preferable to overlap wire meshes at their joints and to properly set overlap lengths and the depths of wire meshes from the surfaces of concrete slabs depending on the thicknesses of concrete slabs.

(c) Joints

It is preferable that concrete pavements are provided with joints that allow concrete slabs to expand, shrink, and warp freely to some extent and thereby reduce stresses.

1) Joints are classified into several types depending on directions and purposes. Longitudinal and transverse joints are the joints installed parallel to and perpendicular to construction directions, respectively. Construction joints are installed for construction purposes such as concrete placement and temporary suspension of construction work. Contraction joints are installed to prevent contraction cracks in a manner that reduces tensile stresses in concrete slabs by allowing the joints to absorb the contraction deformation of concrete slabs due to temperature reductions and drying. Expansion joints are installed to prevent blowup (upward movement of concrete slabs) in a manner that reduces compressive stresses in concrete slabs by allowing the joints to absorb the dilation deformation of concrete slabs due to temperature increases.

2) Joints shall be installed at appropriate locations in accordance with the sizes of aprons, structures of mooring facilities, the types of joints, and loading conditions and have appropriate structures suitable for the types of joints. In contrast, it is necessary to minimize the number of joints because they are structural weak points in concrete pavements and require complex construction procedures that cause construction and maintenance costs to be increased. Considering that joints are to be installed for allowing concrete slabs to freely deform, it is preferable that joints of concrete slabs close to each other are aligned to the extent possible. When intricately arranged as is the case with misaligned transverse contraction joints installed on concrete slabs arranged in rows with a longitudinal joint in between, joints may constrain free deformation of concrete slabs.

3) Longitudinal joints

i. Longitudinal construction joints:

Longitudinal construction joints shall generally have a press-type structure with tie bars (refer to **Part II, Chapter 5, 9.18.3 (4) ⑤ (d) Tie bars and dowel bars**). Tie bars are, however, not used for the pavements on piled pier slabs. It is preferable to set longitudinal joints at proper intervals, less than 5 m in many cases, depending on paving machines used, total pavement widths, and traveling crane rails.

ii. Longitudinal expansion joints:

Longitudinal expansion joints generally have a structure comprising a joint-sealing compound (upper) and a joint filler (lower) with dowel bars (refer to **Part II, Chapter 5, 9.18.3 (4) ⑤ (d) Tie bars and dowel bars**). Dowel bars are, however, not used for the pavements close to the superstructures of quaywalls and sheds as well as on piled pier slabs. It is preferable to place longitudinal expansion joints on the shoulder of backfill, the joints of quaywalls, and the position of sheet-pile anchorages to reduce the effects of structural boundaries below base courses, differences in bearing capacity, and the joints of quaywalls on concrete pavements.

4) Transverse joints

i. Transverse contraction joints:

Transverse contraction joints shall generally have a dummy-type structure with dowel bars. On piled pier slabs, however, dowel bars are not used. It is preferable to set transverse contraction joints at proper intervals, less than 5 m in many cases, depending on construction conditions. It is also preferable for shrinkage joints to be installed on the joints of quaywalls.

ii. Transverse construction joints:

Transverse construction joints shall generally have a press-type structure with dowel bars. On piled pier slabs, however, dowel bars are not used. Transverse construction joints are installed at the end of daily construction work or inevitably installed due to rain during construction or the failures of construction machines or other equipment. It is preferable that the positions of transverse construction joints coincide with those of transverse contraction joints.

iii. Transverse expansion joints:

It is preferable that transverse expansion joints generally have a structure comprising a joint-sealing compound (upper) and a joint filler (lower) with dowel bars. For the joints close to the superstructures of quaywalls and sheds as well as on piled pier slabs, however, dowel bars are not used. It is also preferable to set transverse expansion joints at proper intervals, normally 100 to 200 m when constructed in summer and 50 to 100 m in winter, depending on construction conditions. Because expansion joints are the weakest points in pavements, consideration is needed for minimizing their installation locations.

5) Joint structures

Figs. 9.18.6 to **9.18.9** show standard joint structures.

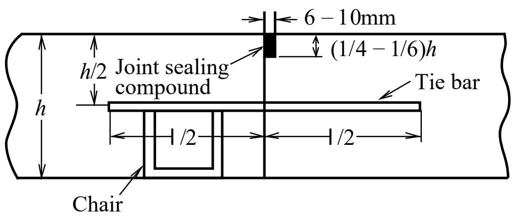


Fig. 9.18.6 Longitudinal Construction Joint

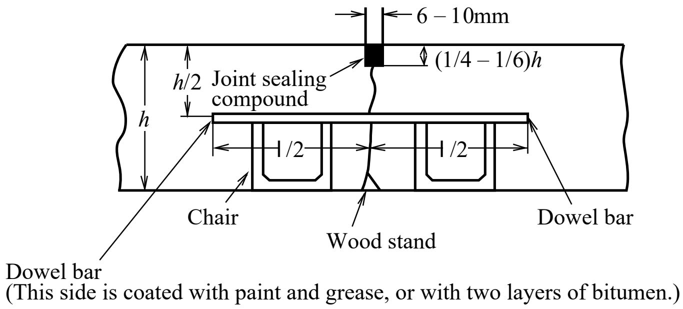


Fig. 9.18.7 Transverse Contraction Joint

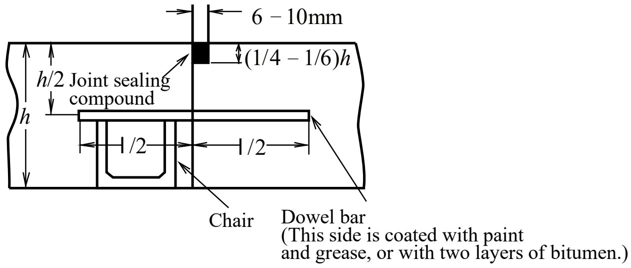


Fig. 9.18.8 Transverse Construction Joint

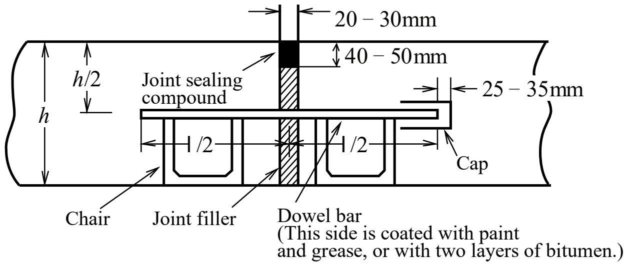


Fig. 9.18.9 Transverse Expansion Joint

(d) Tie bars and dowel bars

1) Tie bars are installed to prevent slabs next to each other from being separated or having differences in level. They are also capable of transmitting loads between slabs. Apron pavements having relatively narrow widths with both ends confined by structures such as quaywalls and sheds have rarely caused concrete slabs to be separated, but tie bars are still used for longitudinal construction joints to enable concrete slabs to cope with uneven settlement of the layers below base courses and traffic loads acting on them in diverse directions unlike in the case of traffic loads on general roads.

2) Dowel bars have function to transfer loads without restricting relative movement of concrete slabs next to each other in the direction perpendicular to joints and to prevent concrete slabs from having differences in level. Generally, dowel bars are used for transverse contraction, construction, expansion joints as well as longitudinal expansion joint to enable loads to be sufficiently transferred.

3) Tie bars and dowel bars shall be properly selected considering the traveling loads imposed on apron pavements in all directions.

4) The specifications and placement intervals of tie bars and dowel bars may refer to the values shown in **Table 9.18.7**.

Table 9.18.7 Reference Values for the Specifications and Placement Intervals of Tie Bars and Dowel Bars

Action classification	Slab thickness (cm)	Tie bar			Dowel bar		
		Diameter (cm)	Length (cm)	Interval (cm)	Diameter (cm)	Length (cm)	Interval (cm)
CP_1	20	25	80	45	25	50	45
CP_2	25	25	100	45	25	50	45
CP_3	30	32	100	40	32	60	40
CP_4	35	32	100	40	32	60	40

Note: The values of tie bars and dowel bars are those of SD295A (deformed steel bar) specified in JIS G 3112 and of SS400 (round steel bar) specified in JIS G 3101, respectively.

(e) End protection

1) Pavements are preferably provided with end protection work at locations with risks of destruction of base courses due to infiltration of rain water or destruction of the concrete slabs and base courses due to heavy loading.

2) When apron pavements are located next to open storage yards or empty land to be paved later, the landward pavement edges are normally brought into contact with soil and, therefore, have risks of destruction of base courses due to the infiltration of rain water and the destruction of base courses and concrete slabs due to transverse traffic loads in the cases of aprons next to open storage yards. Thus, the portions of pavements with these risks shall be provided with end protection to obviate them.

3) Examples of end protection are shown in **Fig. 9.18.10**.

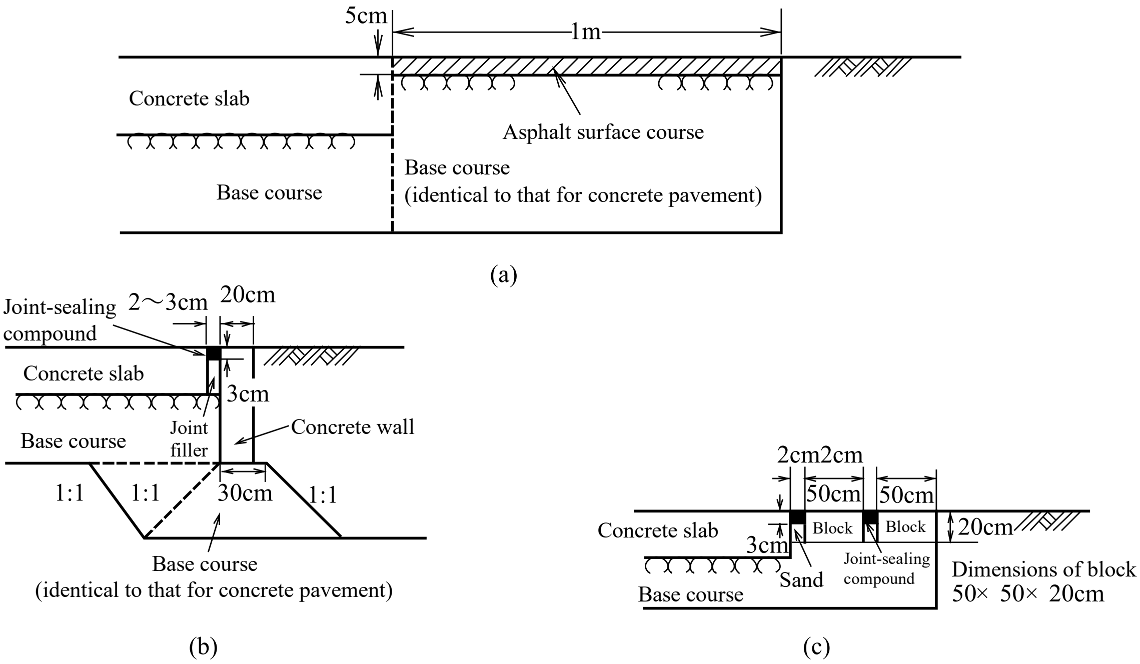


Fig. 9.18.10 Examples of End Protection

(f) Junctions with asphalt pavements

1) When the boundaries between concrete and asphalt pavements are subjected to traffic loads, it is preferable to install transition boards below asphalt pavements in order to prevent the occurrence of the differences in levels between the two types of pavements or damage to the boundaries due to the difference in their bearing capacity.

2) For the dimensions of transition boards, reference can be made to the **Manual for Pavement Design** and the **Manual for Airport Pavement Design**.

(5) Performance verification of asphalt pavements

① Compositions of asphalt pavements

As shown in **Fig. 9.18.11**, an asphalt pavement generally has a cross-sectional structure where a base course, an asphalt mixture layer (surface and binder courses), and base course (upper and lower bases) are arranged on subgrade.

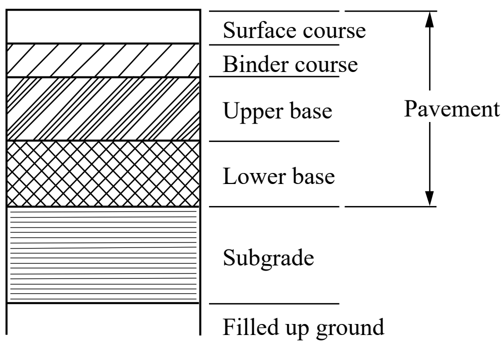


Fig. 9.18.11 Composition of Asphalt Pavements

② Procedures of performance verification

(a) **Fig. 9.18.12** shows an example of the procedures of the performance verification for asphalt pavements.

(b) The performance verification of asphalt pavements can be based on the T_A method (refer to **Part II, Chapter 5, 9.18.3 (4) ④ (a) Verification of asphalt pavement**) in which the compositions of pavements are determined so that pavement thicknesses do not fall below the layer equivalent thicknesses calculated from the bearing capacity of subgrade and the number of repetitions of actions.

(c) In the performance verification of asphalt pavements subjected to the traveling loads of large cargo handling machines, it is preferable to use a theoretical design method in which the compositions of pavements are determined by the strain to be generated in pavements and the fatigue failure frequency calculated on the basis of the multilayer elastic theory.

③ **Design conditions**

(a) The design conditions to be considered in the performance verification are generally as follows:

1) design working life;

2) action conditions;

3) the number of repetitions of actions;

4) bearing capacity of subgrade; and

5) materials used.

(b) Design working life

The design working life of asphalt pavements shall be properly set considering the usage conditions of mooring facilities. The design working life of asphalt pavements used for the aprons of quaywalls may be generally set at 10 years.

(c) Action conditions

Action conditions shall be those, among the kinds of subject actions, which maximize asphalt pavement thicknesses.

(d) Calculation of the number of repetitions of actions

For the calculation of the numbers of repetitions of actions, reference can be made to **Part II, Chapter 5, 9.18.3 (4) ③ (d) Calculation of the number of repetitions of actions**.

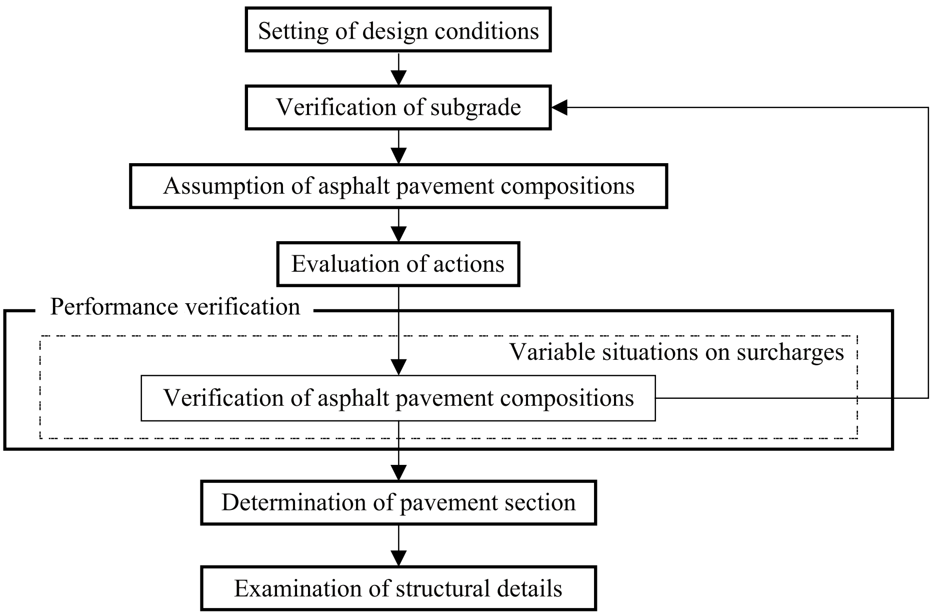


Fig. 9.18.12 Example of Procedures of Performance Verification for Asphalt Pavements

(e) Bearing capacity of subgrade

When obtaining design *CBR*s for the subgrade of pavement sections subjected to the performance verification, CBR tests shall be carried out in a manner that measures CBRs after tamping down subgrade soil containing natural moisture and immersing it in water for 4 days in compliance with the **Test Methods for the CBR of Soils in Laboratory (JIS A 1211)**. In the CBR tests, sampled subgrade soil from which aggregates not less than 40 mm are separated shall be put in molds in three layers with each layer subject to tamping 67 times. In the cases where subgrade is already constructed, design *CBR*s shall be obtained generally through on-site CBR tests with subgrade soil in the wettest condition. If on-site CBR tests cannot be carried out in the ideal wet conditions, design *CBR*s can be obtained by correcting the on-site CBR test results using **equation (9.18.6)**. Here, it should be noted that *CBR*s are applicable to undisturbed soil samples. It is preferable that soil is sampled from one to two locations in every 50 m along the face lines of quaywalls at the depths of 50 cm or more from the surfaces of completed subgrade or exposed surfaces of borrow pits.

$$CBR \text{ (corrected)} = \text{In-situ } CBR \cdot \frac{CBR \text{ (with immersion in water for 4 days)}}{CBR \text{ (with natural moisture content)}} \tag{9.18.6}$$

Design *CBR*s can be obtained by **equation (9.18.7)** using the above-defined *CBR*s excluding extreme values.

$$\text{Design } CBR = \begin{array}{l}\text{The average of CBRs}\\ \text{at respective locations}\end{array} - \frac{\begin{array}{l}\text{The maximum}\\ \text{value of CBR}\end{array} - \begin{array}{l}\text{The minimum}\\ \text{value of CBR}\end{array}}{C} \tag{9.18.7}$$

where C is given in **Table 9.18.4**.

(f) Material used

1) Base course

The requirements for the items related to the quality and grain sizes of base course materials can follow the provisions in the **Pavement Design and Construction Guide**.[30)]

2) Surface and binder courses

i. It is preferable that polymer-modified asphalt (Modification Type II, Type III or special asphalt with dynamic stability equivalent to semi-flexible mixtures) is used for the surface and binder courses in the areas with possible actions of heavy or static loads; a risk of early development of rutting on the basis of previous experience; or the possibility that excessively large rutting pose problems with apron operation.

ii. There may be the cases where pavements of sorting facilities have deep cracks or dents due to locally applied impact loads during container handling and aged deterioration. Because these deep cracks and dents have a risk of being fertile breeding grounds for alien species transported together with containers and the like, it is preferable that these cracks and dents are properly repaired and that pavement design considers local reinforcement measures in accordance with use conditions of sorting facilities, for example, concrete slabs or steel plates laid on the places where containers are planned to be stored.

④ **Performance verification**

(a) Verification of asphalt pavement compositions

1) Setting of pavement sections

Pavement compositions shall be determined so that the layer equivalent thicknesses of set pavement cross sections do not fall below the required layer equivalent thicknesses.

2) Required layer equivalent thicknesses

The required layer equivalent thicknesses T_A can be calculated by **equation (9.18.8)**.

$$T_A = \frac{3.84N^{0.16}}{CBR^{0.3}} \tag{9.18.8}$$

where

T_A : a required layer equivalent thickness (cm); and

N : the value obtained by converting the number of repetition of action during design working life n_i to 49 kN wheel load using the following equation.

$$N = \sum_{i=1}^{m}\left[\left(\frac{P_i}{49}\right)^4 n_i\right] \tag{9.18.9}$$

where

P_i : a wheel load (kN);

n_i : the number of repetitions of wheel load P_i; and

m : the number of loaded states.

3) Layer equivalent thicknesses of assumed sections

The layer equivalent thicknesses T_A' of assumed cross sections can be calculated by **equation (9.18.10)**.

$$T_A' = \sum_{i=1}^{n}\left[a_i h_i\right] \tag{9.18.10}$$

where

T_A' : a layer equivalent thickness of an assumed section (cm);

h_i : a thickness of layer i (cm);

a_i : a layer equivalent value set for material and work method used for each pavement layer (refer to **Table 9.18.8**); and

n : the number of layers.

Table 9.18.8 Equivalent Conversion Factors of Asphalt Pavements

Layer	Construction method/material	Requirement	Layer equivalent value	Remark
Surface and binder courses	Hot asphalt mixture for surface and binder courses	–	1.00	AC I–AC IV
Upper base	Bituminous stabilization	Marshall stability 3.43 kN or greater	0.80	A-treated material II
		Marshall stability 2.45 to 3.43 kN	0.55	A-treated material I
	Hydraulic mechanically stabilizing steel slag	Modified CBR 80 or greater Unconfined compressive strength (14 days) 1.2 MPa	0.55	
	Mechanical stabilized material	Modified CBR 80 or greater	0.35	Mechanical stabilized material
Lower base	Crusher run, slag, sand, etc.	Modified CBR 30 or greater	0.25	
		Modified CBR 20 to 30	0.20	Grain material

(b) Example of empirical verification of asphalt pavement compositions

Table 9.18.10 shows an example of the empirical verification of asphalt pavement compositions. The table is prepared referring to the action conditions shown in **Table 9.18.9**. The symbols H and T_A' in **Table 9.18.10** express total pavement thickness and the equivalent conversion asphalt pavement thickness of the assumed section, respectively. If the design CBR of a subgrade is 2 or more and less than 3, it is preferable to replace it with one using good quality materials or to add a water-sealing layer. If it is less than 2, it is preferable to replace it with good quality materials and set the pavement thickness once again.

(c) The type and material quality of asphalt mixtures can be set as listed in **Table 9.18.11**.

Table 9.18.9 Reference Values for Action Conditions for Asphalt Pavements on Quaywall Aprons

Action classification	Cargo handling machine	
AP_1	Tractor trailer	20 ft, 40 ft
AP_2	Fork lift truck	2 t
	Fork lift truck	3.5 t
	Fork lift truck	6 t
AP_3	Fork lift truck	10 t
	Fork lift truck	15 t
	Truck	25 ton class
	Straddle carrier	
	Mobile crane (truck crane, rough terrain crane, all terrain crane)	Type 20
AP_4	Mobile crane (truck crane, rough terrain crane, all terrain crane)	Type 25

Table 9.18.10 Examples of Compositions of Asphalt Pavement

Condition of actions		Pavement composition								
Classification of actions	Design *CBR* of subgrade (%)	Surface course		Binder course		Upper base		Lower base	Total thickness	
		Type	h_1 (cm)	Type	h_2 (cm)	Type	h_3 (cm)	h_4 (cm)	H (cm)	T_A' (cm)
AP_1	Equal to or above 3 and less than 5	AC I	5	AC III	5	Mechanical stabilized material	25	35	70	25.8
		AC I	5	–	–	A-treated material I	25	35	65	25.8
	Equal to or above 5 and less than 8	AC I	5	AC III	5	Mechanical stabilized material	20	25	55	22.0
		AC I	5	–	–	A-treated material I	20	30	55	22.0
	Equal to or above 8 and less than 12	AC I	5	AC III	5	Mechanical stabilized material	15	20	45	19.3
		AC I	5	–	–	A-treated material I	15	30	50	19.3
	Equal to or above 12 and less than 20	AC I	5	AC III	5	Mechanical stabilized material	15	15	40	18.3
		AC I	5	–	–	A-treated material I	15	20	40	17.3
	Equal to or above 20	AC I	5	AC III	5	Mechanical stabilized material	15	15	40	18.3
		AC I	5	–	–	A-treated material I	15	15	35	16.3
	On the deck slab of piled pier	AC I	5	AC III	4 or greater	–	–	–	9 or greater	–
AP_2	Equal to or above 3 and less than 5	AC II	5	AC IV	5	Mechanical stabilized material	25	35	70	25.8
		AC II	5	–	–	A-treated material I	25	35	65	25.8
	Equal to or above 5 and less than 8	AC II	5	AC IV	5	Mechanical stabilized material	20	25	55	22.0
		AC II	5	–	–	A-treated material I	20	30	55	22.0
	Equal to or above 8 and less than 12	AC II	5	AC IV	5	Mechanical stabilized material	15	20	45	19.3
		AC II	5	–	–	A-treated material I	15	30	50	19.3
	Equal to or above 12 and less than 20	AC II	5	AC IV	5	Mechanical stabilized material	15	15	40	18.3
		AC II	5	–	–	A-treated material I	15	20	40	17.3
	Equal to or above 20	AC II	5	AC IV	5	Mechanical stabilized material	15	15	40	18.3
		AC II	5	–	–	A-treated material I	15	15	35	16.3
	On the deck slab of piled pier	AC II	5	AC IV	4 or greater	–	–	–	9 or greater	–
AP_3	Equal to or above 3 and less than 5	AC II	5	AC IV	15	Mechanical stabilized material	30	45	95	40.0
		AC II	5	AC IV	10	A-treated material II	20	45	80	40.0
	Equal to or above 5 and less than 8	AC II	5	AC IV	15	Mechanical stabilized material	25	30	75	34.8
		AC II	5	AC IV	10	A-treated material II	20	20	55	35.0
	Equal to or above 8 and less than 12	AC II	5	AC IV	15	Mechanical stabilized material	15	20	55	29.3
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	Equal to or above 12 and less than 20	AC II	5	AC IV	15	Mechanical stabilized material	15	15	50	28.3
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	Equal to or above 20	AC II	5	AC IV	15	Mechanical stabilized material	15	15	50	28.3
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	On the deck slab of piled pier	AC II	5	AC IV	4 or greater	–	–	–	9 or greater	–

Condition of actions		Pavement composition								
Classification of actions	Design *CBR* of subgrade (%)	Surface course		Binder course		Upper base		Lower base	Total thickness	
		Type	h_1 (cm)	Type	h_2 (cm)	Type	h_3 (cm)	h_4 (cm)	H (cm)	T_A' (cm)
AP_4	Equal to or above 3 and less than 5	AC II	5	AC IV	15	Mechanical stabilized material	40	60	120	46.0
		AC II	5	AC IV	10	A-treated material II	20	70	105	45.0
	Equal to or above 5 and less than 8	AC II	5	AC IV	15	Mechanical stabilized material	30	45	95	39.5
		AC II	5	AC IV	10	A-treated material II	20	40	75	39.0
	Equal to or above 8 and less than 12	AC II	5	AC IV	15	Mechanical stabilized material	25	30	75	34.8
		AC II	5	AC IV	10	A-treated material II	15	35	65	34.0
	Equal to or above 12 and less than 20	AC II	5	AC IV	15	Mechanical stabilized material	15	25	60	303
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	Equal to or above 20	AC II	5	AC IV	15	Mechanical stabilized material	15	15	50	28.3
		AC II	5	AC IV	10	A-treated material II	15	15	45	30.0
	On the deck slab of piled pier	AC II	5	AC IV	4 or greater	–	–	–	9 or greater	–

Note: In the case of the deck slab of piled pier, the boxes for the binder course in Table 9.18.10 refer to the values for binder courses including filling materials that are not limited to asphalt mixtures.

Table 9.18.11 Types and Quality of Asphalt Mixtures

Type	AC I	AC II	AC III	AC IV
Use	For surface course		For binder course	
Number of blows for Marshall stability test	50 times	75 times	50 times	75 times
Marshall stability (kN)	4.9 or greater	8.8 or greater	4.9 or greater	8.8 or greater
Flow value (1/100 cm)	20–40	20–40	15–40	15–40
Air void (%)	3–5	2–5	3–6	3–6
Degree of saturation (%)	75–85	75–85	65–80	65–85

Note: The number of blows of 75 times for Marshall stability test is applied to the cases where the ground contact pressure is not less than 70 N/cm^2 or rutting is expected because of particularly heavy traffic of large vehicles.

⑤ **Structural details**

(a) In the cold regions where pavements are subjected to freezing and thawing, it is necessary that pavements are provided with frost heave prevention layers in the cases of pavement thicknesses less than frost penetration depths.

(b) For end protection, reference can be made to **Part II, Chapter 5, 9.18.3 (4) ⑤ (e) End protection**.

⑥ **Semi-flexible pavements**

(a) Semi-flexible pavements mostly have the same pavement compositions as asphalt pavements with semi-flexible mixtures used for the surface course. When increasing the thickness of semi-flexible mixture layers, it is necessary to ensure that cement milk fully infiltrates the layers from their surfaces to their bottoms.

(b) The design working life of semi-flexible pavements shall be properly set considering the usage conditions of mooring facilities. The design working life of semi-flexible pavements used for the aprons of quaywalls may be generally set at 10 years.

(c) Semi-flexible pavements are subjected to fine cracks on their surfaces due to the drying shrinkage when cement milk hardens and the shrinkage associated with the fluctuation of external temperature.[36] These cracks do not pose structural problems unless they penetrate semi-flexible mixture layers. Thus, application of sealing material shall be implemented with attention to the prevention of rainwater from infiltrating the layers through cracks. Also, the boundaries of semi-flexible pavements having different bearing capacity may have early development of cracks due to the difference in the levels of deformation when subjected to traveling loads. Thus, it is preferable that joints are installed along such boundaries in a manner that cuts pavements after they harden.[37)]

9.19 Foundations for Cargo Handling Equipment

[Public Notice] (Perform–ance Criteria of the Foundations for Cargo Handling Equipment)

Article 74

1 The performance criteria of the foundations for cargo handling equipment shall be as prescribed respectively in the following items in consideration of the types of cargo handling equipment and the structural types of foundations:

(1) The foundations shall have the dimensions necessary for enabling the safe and smooth operation of cargo handling tasks, transportation of cargo handling equipment.

(2) The foundations shall satisfy the following criteria under the variable situation, in which the dominating actions are Level 1 earthquake ground motion and surcharge load:

(a) For pile-type structures, the risk that the axial force acting on a pile may exceed the resistance force with which underground failure occurs shall be equal to or less than the threshold level.

(b) For pile-type structures, the risk that the stress in a pile may exceed its yield stress shall be equal to or less than the threshold level.

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

(d) For pile-less structures, the risk of beam sliding shall be equal to or less than the threshold level.

(3) The beam deflection shall be equal to or less than the threshold level under the variable situation in which the dominating action is surcharge load.

2 In addition to the provisions of the preceding paragraph, the performance criteria for the foundations for the cargo handling equipment to be installed in high earthquake-resistance facilities shall be such that the degree of damage under the accidental situation, in which the dominating action is Level 2 earthquake ground motion, is equal to or less than the threshold level corresponding to the performance requirements.

[Interpretation]

11. Mooring Facilities

(18) The performance criteria of the foundations for cargo handling equipment (Article 33 of the Ministerial Ordinance and the interpretation related to Article 74 paragraph 1 of the Public Notice)

① **General provisions for the foundations for cargo handling equipment**

(a) The performance requirements for the foundations for cargo handling equipment shall focus on serviceability. Serviceability is defined as the appropriate setting of the dimensions of cargo handling equipment in accordance with the types of cargo handling equipment and the structural types of foundations so that the safe and smooth operation of cargo handling tasks, transportation of cargo handling equipment, etc., are ensured.

(b) In addition to the above provision, the performance requirements for the foundations for cargo handling equipment under the variable situation in which the dominating actions are Level 1 earthquake ground motion and surcharge shall focus on serviceability. Furthermore, the performance verification items and the standard indexes of the determination of the limit values against the actions are shown in **Attached Table 11-35**.

Attached Table 11-35 Performance Verification Items and Standard Indexes of the Determination of the Limit Values in Each Design State for the Structure of the Foundations for Cargo Handling Equipment (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index of the determination of the limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
33	1	2	74	1	2 a	Serviceability	Variable situation	L1 earthquake ground motion (Surcharge*3))	Self weight, earth pressure	Axial force acting on pile*1)	Ratio of bearing capacity–related action on pile-to-resistance capacity (pushing, pulling)
					2 b					Yield of pile*1)	Design yield stress
					2 c					Section failure of the beam	Design resistance value of section
					2 d					Sliding of the beam*2)	Ratio of sliding related action-to-resistance capacity
					3			Surcharge*3)	Self weight, earth pressure	Deflection of the beam	Deflection

*1): This is only applicable to structures with piles for the foundations for cargo handling equipment.
*2): This is only applicable to structures without a pile for the foundations for cargo handling equipment.
*3): This is the action exerted by cargo handling equipment on the foundations and should be properly set according to the design state.

② **The foundations for cargo handling equipment in high earthquake-resistance facilities** (the interpretation of the context of Article 33 of the Ministerial Ordinance and Article 74, Paragraph 2 of the Public Notice)

The performance requirements for the foundations for cargo handling equipment under the accidental situation in which the dominating action is Level 2 earthquake ground motion shall focus on restorability. Furthermore, the performance verification item and the standard index of the determination of the limit value against the action are shown in **Attached Table 11-36**. It should be noted that the verification item in **Attached Table 11-36** shows "Damage," which is meant to be a comprehensive item, because these items differ in terms of the structure and structural types of the facilities concerned. The indexes to determine the limit values shall be designated appropriately for every performance verification item.

Attached Table 11-36 Performance Verification Item and Standard Index of the Determination of the Limit Value of the Foundations for Cargo Handling Equipment in High Earthquake-Resistance Facilities under an Accidental Situation

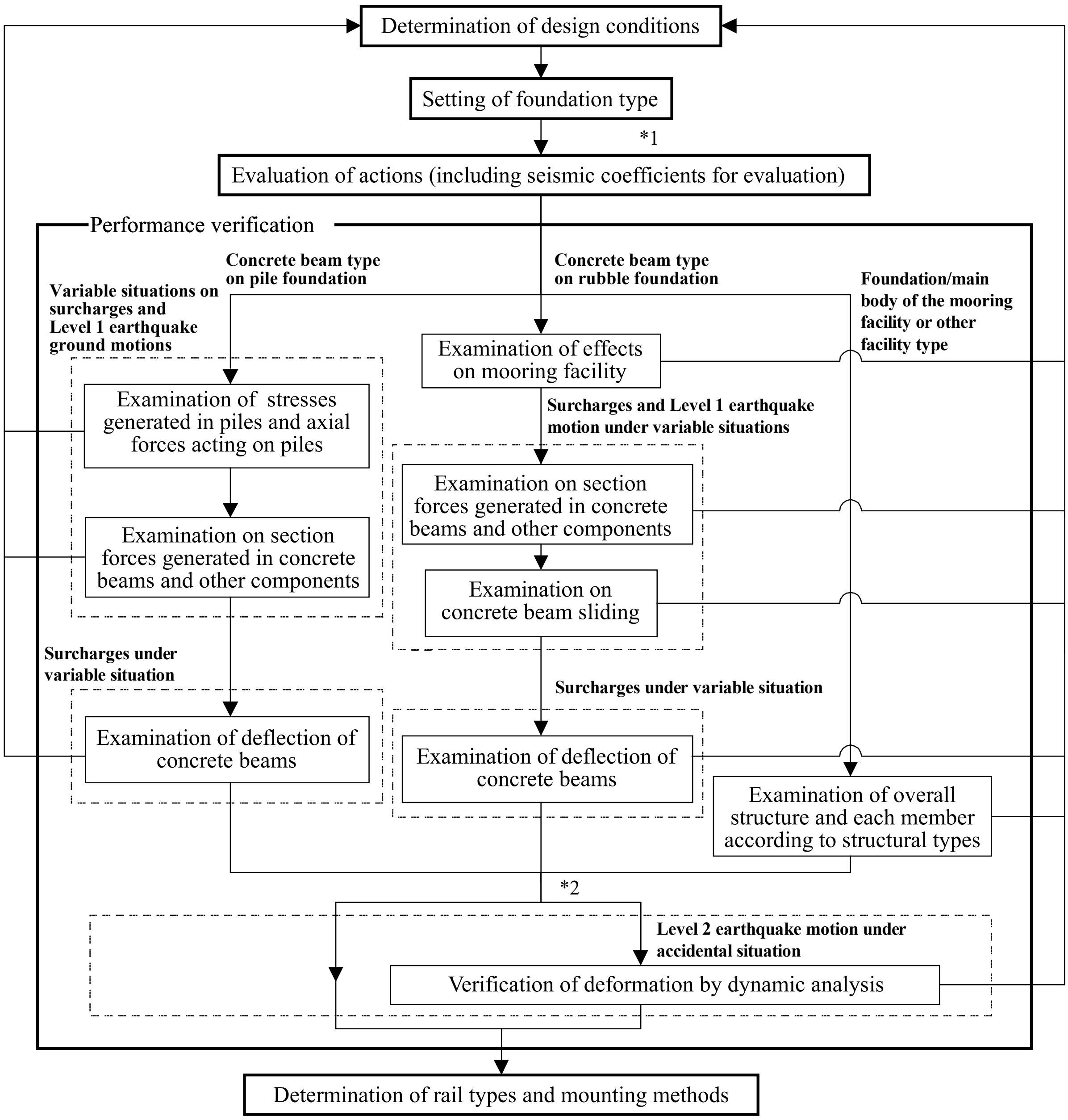
Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index of the determination of limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
33	1	2	74	2	–	Restorability	Accidental situation	L2 earthquake ground motion	Self weight, surcharge, earth pressure	Damage	–

9.19.1 General

(1) The dimensions of the foundations for cargo handling equipment need to be properly set according to the types of cargo handling equipment and the structural types of foundations to allow the safe and smooth operation of cargo handling tasks, transportation of cargo handling equipment, etc.

(2) The foundations for rail-mounted cargo handling equipment need to be determined appropriately in consideration of the action that is exerted on them, their displacement limit value, the difficulty of their maintenance, their effect on the main body of mooring facilities, their economic value, etc.

(3) **Fig. 9.19.1** shows the procedure for the performance verification of the foundations for cargo handling equipment as an example. However, the evaluation of the effect of liquefaction that may be caused by earthquake ground motion is not shown in this figure. Therefore, the possibility of occurrence of liquefaction and the countermeasures need to be evaluated appropriately by referring to **[Part II: Actions and Material Strength Requirements], Chapter 7 Ground Liquefaction**. For the variable situation in Level 1 earthquake ground motion, the seismic coefficient method may be applied to the verification. However, in the case of high earthquake-resistance facilities, the displacement should be evaluated by using nonlinear seismic response analysis in which the dynamic interaction between the ground and the structure is taken into consideration. In addition to the foundations shown in the figure, sleepers on ballasted track bed foundation type similar to railway ballast, which is used in sites where the ground settlement is large or the wheel load of a light type crane is small, and other types are included.



*1: The evaluation of the effects of liquefaction is not included in this chart; therefore, it should be considered separately.
*2: The foundations for cargo handling equipment installed in high earthquake-resistance facilities shall be verified on Level 2 earthquake ground motion.

Fig. 9.19.1. Example of the Procedure for the Performance Verification of the Foundations for Cargo Handling Equipment

(4) Types of Rail-Mounted Foundations

① Foundation type in which the piles are connected by reinforced concrete beams on a piled foundation

This type of foundation is used on soft ground where differential settlement is expected. It is also used for foundations for large cargo handling equipment built on Sandy Ground in good conditions.

② Foundation type in which the main body of mooring facilities or other facilities are utilized

This type of foundation utilizes the main body of mooring facilities such as the reinforced concrete beams of a piled pier and the superstructure of a caisson type quaywall or the anchorage wall of a sheet pile quaywall as a foundation for cargo handling equipment; the performance verification of these facilities shall be conducted in advance by considering the action to be exerted by the cargo handling equipment. In this case, it becomes economical in terms of the facilities as a whole. When one leg is on the main body of a mooring facility and the other leg is on an independent foundation, caution is needed to avoid differential settlement. It should be noted that the action of earthquake ground motion may cause the displacement of a crane foundation, thus resulting in the displacement or derailing of the crane leg. Furthermore, a column fixed to a beam structure of a portal crane is not normally mounted on a piled pier. Considering that the end of a jetty-type piled pier is a weak spot to the actions of ship berthing force or tractive force or action by earthquake ground motion, reinforcement should be considered.

③ **Foundation type in which concrete beams are built on a rubble foundation**

This type of foundation is applicable to the ground relatively in good conditions with a slight possibility of settlement.

(5) Limit Value of Displacement of Rails

① The limit value of displacement of rails mounted on the foundations of cargo handling equipment needs to be set in with consideration to the relation between the critical displacement values of the cargo handling equipment concerned and its production costs, the precision of building the foundations, the stability and efficiency, etc., when the cargo handling equipment is in operation.

② When a relative displacement is assumed at the leg part of the equipment concerned on a gravity-type or a sheet-pile-type quaywall, it is necessary to examine the deformation performance.

③ In the performance verification and the execution of foundations, it is necessary that the critical displacement values designated for foundations by the manufacturer of the equipment concerned should be examined, a structure that ensures the lowest differential settlement should be selected, and an execution method that ensures the highest precision should be adopted.

④ The maintenance inspection of foundations for cargo handling equipment must be performed. When displacement exceeding the standard value specified for the maintenance is found, the foundation concerned must be adjusted by liner adjustment or a filling.

⑤ It should be noted that the displacement of rails increases with time; therefore, it is desirable that the execution error be made as small as possible. Although allowable displacement varies depending on manufacturer, the average laying standards and criteria are in **Table 9.19.1**.[38)]

Table 9.19.1. Example of Traveling Rails Laying Standards

Item	Installation standards	Criteria (critical values for usage)
Span	±10 mm or less for the entire rail length	±10 mm or less for the entire rail length
Lateral and vertical warps of the rail	5 mm or less per 10 m of rails	10 mm or less per 10 m of rails
Elevation difference between seaward and landward rails	1/1000 of rail span or less	1/500 of rail span or less
Gradient in the traveling direction	1/500 or less	1/250 or less
Straightness	±50 mm or less for the entire rail length	±80 mm or less for the entire rail length
Rail joints	Vertical and lateral differences: ±0.5 mm or less	Vertical and lateral differences: ±1 mm or less
	Gap: 5 mm or less	Gap: 5 mm or less
Wear of the head of the rail	–	10% or less of the original dimension

(6) The rails are fastened to the foundation structure with rail clips: Normally, 37 or 50 kg rails are used for the track structure of cranes handling sundry goods and for small cranes, whereas 73 kg crane rails are used for large unloaders handling containers and minerals. A shock absorber such as a rubber pad should be applied to avoid crack generation in concrete, which can be caused by the movement of cranes, under usage conditions because the wheel load of a crane is large and because direct contact occurs between concrete and rails or plates.

(7) When the action of earthquake ground motion is large, rocking motion is generated on a traveling-type crane, thus resulting in damage to the crane leg; therefore, consideration should be given to a seismicity.

(8) There are two types of structures for crane legs: a column fixed to beam type and a hinged column (or a rear leg) type. A hinged column is often applied to the sea side where a large horizontal force cannot be exerted on the sea side foundation (in piled pier structure, in sheet pile structure, etc.). The span of rails is very broad, thus making it difficult to meet the level of rail setting criteria. By contrast, a column fixed to a beam type is applied to the land side in the aforementioned cases, as well as to both the sea and land sides as far as there is no such restriction (all columns fixed to beam). Furthermore, the horizontal force acting on rails in a hinged column structure on the sea side is smaller than that in a column fixed to a beam structure because of the effect of a hinge system, although the horizontal force acting on rails generally becomes large as a column fixed to a beam type on the land side exerts a larger horizontal force accordingly on rails than all columns fixed to a beam type. Furthermore, these crane leg structures do not make a difference in regard to vertical force.

(9) The crane locking devices comprise buffer stops, turnover prevention apparatus, and rail clamps. The buffer stop is a device that prevents the crane from running off to its traveling direction owing to a wind load at the time of a storm, and the turnover prevention apparatus is a device that prevents the crane from overturning from a wind load during a storm. The rail clamps prevent the crane in operation from running off to its traveling direction owing to a wind load being exerted by a gust.

(10) The examples of the arrangement of crane locking devices for a container crane are shown in **Fig. 9.19.2**. Crane locking devices can be an integrated type, which has both functions of buffer stops and turnover prevention apparatus, or a separated type, which has the functions of the aforementioned two independent units. **Fig. 9.19.3** shows the examples of metal fittings to the foundations of buffer stops and those to the foundations of a turnover prevention apparatus of a separated type.

(11) In renewing a worn-out wheel, they must be jacked up. However, it may not be possible to jack them up where the ground is soft; therefore, jacks should be prevented from sinking by installing the metal fittings to jack-up both sides of the rails (**Fig. 9.19.4**).

(12) The metal fittings of end stoppers are commonly fixed to foundations at the ends of traveling rails (**Fig. 9.19.5**) to mechanically stop the accidental running of the crane.

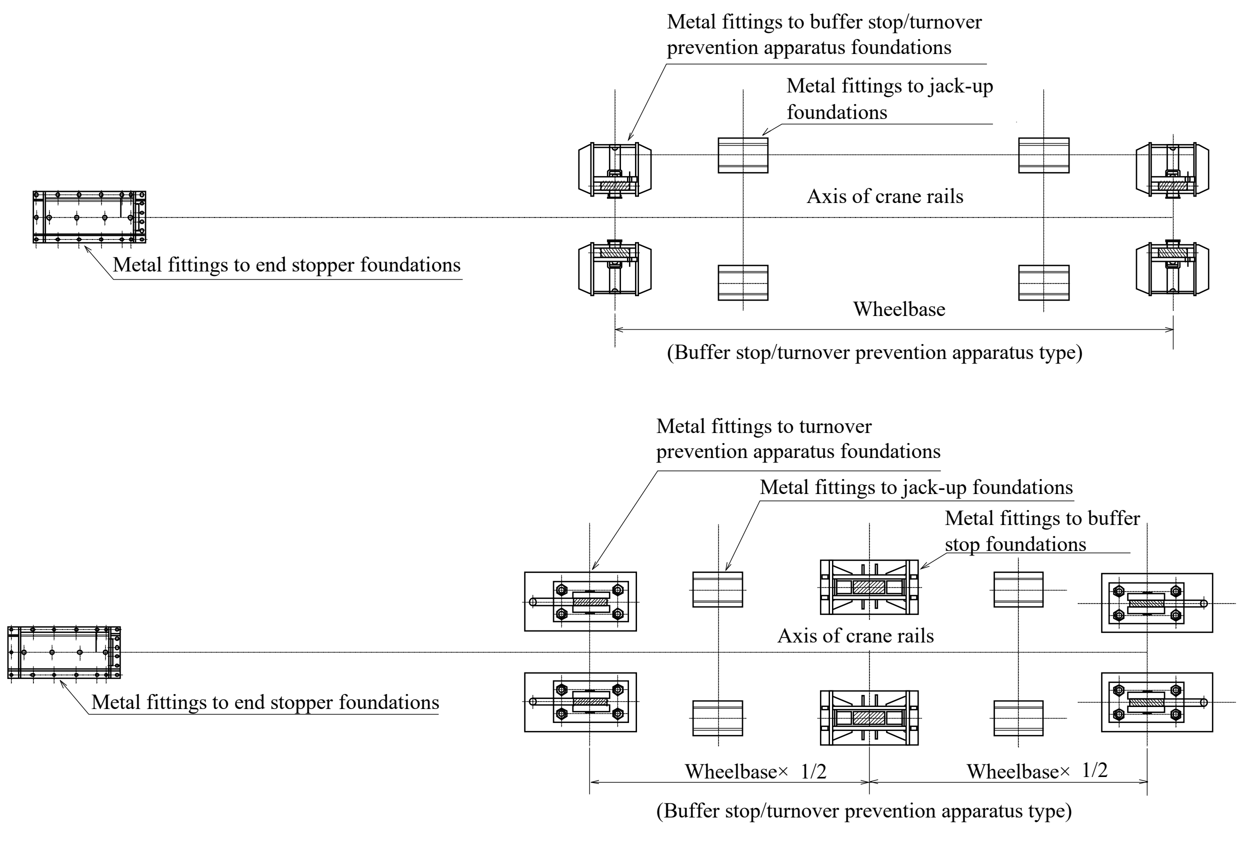
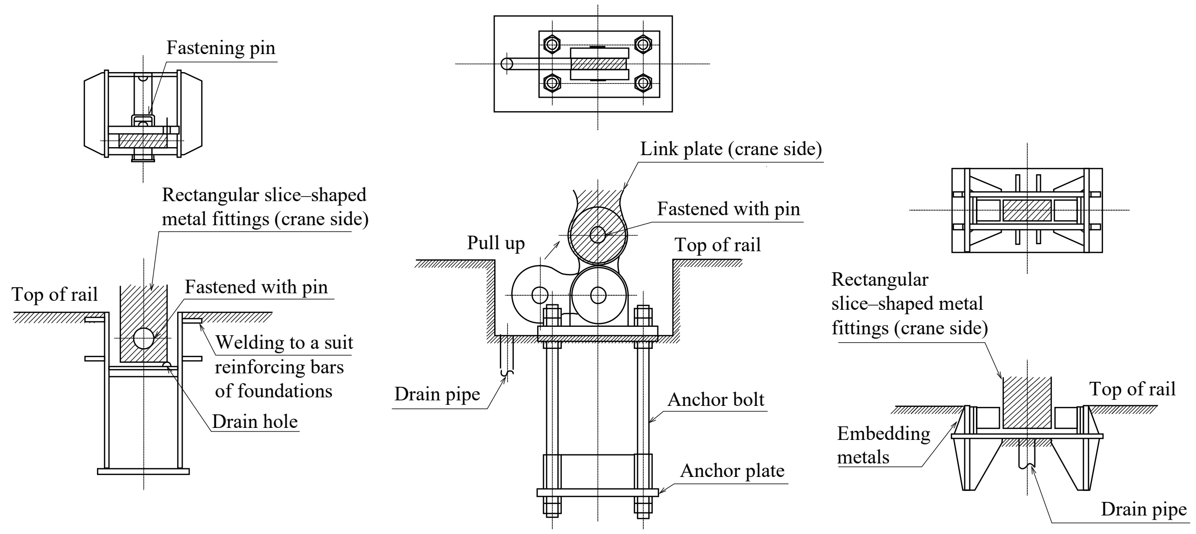


Fig. 9.19.2. Examples of the Arrangement of Locking Devices for Container Cranes[39)]



Metal fittings to buffer stop/turnover prevention apparatus foundations

Metal fittings to turnover prevention apparatus foundations

Metal fittings to buffer stop foundations

Fig. 9.19.3 Examples of the Metal Fittings of Locking Devices to the Foundations for a Container Crane[39)]

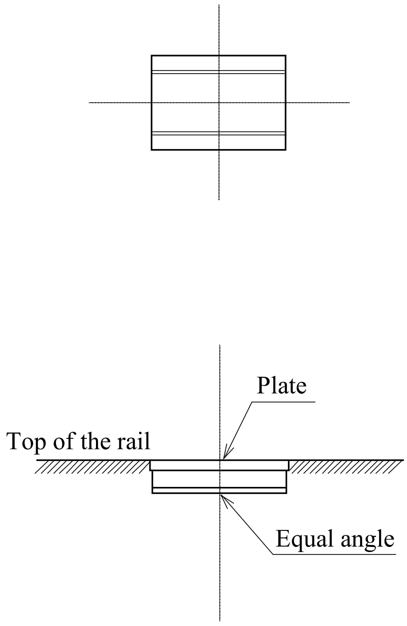


Fig. 9.19.4. Metal Fittings to Foundations for Jack-Up[39)]

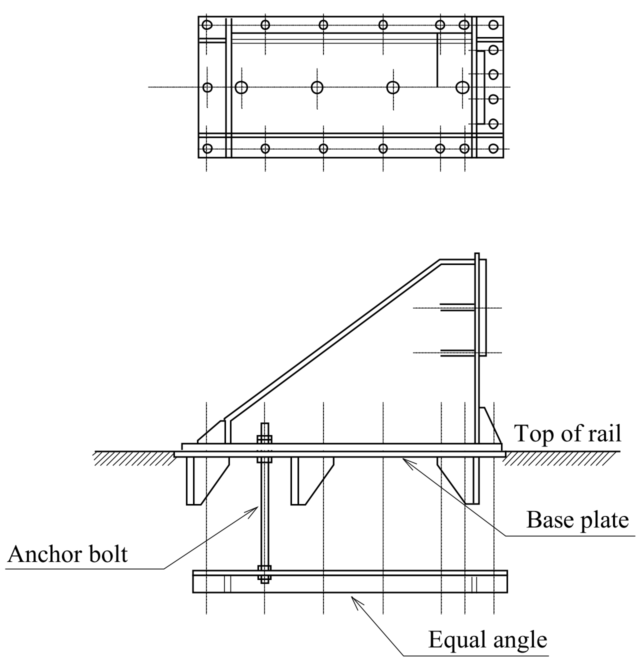


Fig. 9.19.5 Metal Fittings to the Foundations for End Stoppers[39)]

9.19.2 Actions

(1) The action exerted on the foundations for cargo handling equipment shall be determined appropriately in due consideration of the crane type, operation conditions, and other factors.

(2) For the action on the foundations for cargo handling equipment, the values calculated on the basis of **the Calculation Standards for Steel Structures of Cranes (JIS B8821)** or **Structure Standards for Cranes (the Public Notice of the Ministry of Labour)** may be used. For the wheel loads of container cranes being installed in high earthquake-resistance facilities and being under the conditions of an earthquake, refer to **[Part III: Facilities] Chapter 7, 2.2 Container Cranes**. Furthermore, the maximum wheel loads can be tabulated as shown in **Table 9.19.2**. The simultaneous generation of the maximum wheel loads at both sea and land sides must be taken into consideration in the performance verification.

Table 9.19.2 Maximum Wheel Load Items[40] (Unit: kN/wheel)

Columns State of the crane			Sea side	Land side	Notes
During operation	Vertical		○○○	○○○	Capacity of the crane (t/h) Self weight of the crane (kN) Wheelbase (m) Rail span (m) Number of wheels: sea side (wheels) land side (wheels) Distance between wheels (m) Traveling speed of crane (m/min) Column fixed to a beam, hinged column
	Horizontal	Square to rail	○○	○○	
		Parallel to rail	○○	○○	
During storm	Vertical		○○○	○○○	
	Horizontal	Square to rail	○○	○○	
		Parallel to rail	○○	○○	
During earthquake	Vertical		○○○	○○○	
	Horizontal	Square to rail	○○	○○	
		Parallel to rail	○○	○○	

(3) The action shall be considered acting on the entire length of the rails during the operation and during an earthquake, whereas the action shall be considered acting only at the place where the crane is locked during a storm.

(4) For the wheel load during operation, a 20% increase in the maximum wheel pressure of a crane is generally considered a traveling load, whereas a 10% increase is considered a traveling load when the traveling speed of the crane is less than 60 m/min.

9.19.3 Performance Verification of Pile-Type Foundations

(1) Concrete Beams

① The performance verification of concrete beams constructed on pile foundations may be conducted assuming that they are continuous beams supported by pile heads. In this case, the effects of beams contacting the ground are ignored.

② The concrete beams constructed on pile foundations need to be stable against the contact pressure between the rails and concrete, against the stress transmitted from the rails, etc.

③ The underside of the supporting points of beams is subject to tension when elastic settlement occurs on piles by axial force. The beams are generally calculated by assuming that no settling occurs on the supporting points and that the effects of such elastic settlement is taken into account in the arrangement of reinforcement; however, the effects of settling on the supporting points need to be considered when a large wheel load is present,[41] and **[Part III: Facilities], Chapter 2, 3.4.5 Pile Head Displacement by Axial Force** may be used as a reference for performance verification.

④ To avoid elastic settlement or to reduce it, it is advisable either to fill the inside of steel piles with concrete or to employ large-sized steel piles in diameter.

⑤ To calculate the stress on tracks, a solution that assumes beams as infinite beams on elastic foundations is used commonly. This method is particularly applied to the calculation of cases in which elastic compressive materials, such as rubber pads, are inserted between the rails and concrete to distribute the action and prevent the compression failure of the foundation concrete.

⑥ Calculation Method of the Infinite Continuous Beam Supported by Elastic Foundations

The rail stress and the contact pressure between the rails and concrete can be calculated by referring to **9.19.4 Performance Verification in the Cases of Pile-less Foundation, (2) Concrete Beams** in this chapter, described later. In this case, the symbols in **equation (9.19.1)** should be read as follows:

E_c : modulus of elasticity of the rail

I_c : moment of inertia of the rail

K : modulus of elasticity of the material placed under the rail (when tie pads are used, use their modulus of elasticity)

When the bearing stress is excessively large, it should be reduced by inserting elastic plates under the rails.

⑦ The fastening force between the rail and the foundation can be calculated by using the beam theory on elastic foundation[42], but it is necessary to have a sufficient allowance to avoid the effect of the impact. For the calculation of the fastening force for the cases where the double elastic fastening method is employed, refer to Minemura[43]. In many cases, bolts with a diameter of approximately 22 mm are used at intervals of approximately 50 cm.

(2) Maximum Static Resistance Forces of Piles

① The piles shall be stable against the actions caused by cargo handling equipment and foundations.

② The action that is exerted on the piles shall be the reaction force at each supporting point calculated in accordance with above **(1) Concrete Beams**.

③ The maximum static resistance forces of piles may be calculated by referring to **[Part III: Facilities], Chapter 2, 3.4 Pile Foundations**.

④ In cases wherein piles are affected by the failure surface of active earth pressures, the performance verification of the relieving platform piles described in **2.8 Quaywalls with Relieving Platforms** in this chapter may be used as a reference.

⑤ When piles are under the influence of the active earth pressure failure surface, the required embedment length differs between seaward piles and landward piles; however, it is common practice to use foundation piles of the same length for both sides to avoid a differential settlement of the foundation. Nevertheless, the same embedment length is not required when the piles at each side are driven into the bearing stratum.

9.19.4 Performance Verification in Cases of Pile-Less Foundations

(1) Analysis of the Effect on the Quaywall[44]

① When the foundation for cargo handling equipment is not a piled structure, the effect of the actions by the cargo handling equipment and its foundations on the main body of mooring facilities shall be examined.

② A surcharge on the area behind a gravity-type structure increases the earth pressure and may cause the forward sliding of the quaywall. The influence of a concentrated load on the earth pressure is significant at its loading point around the ground surface and becomes distributed depending on the depth. This has significant influence particularly on structures with a short wall height and length (of the direction of alignment); therefore, this type of load should be considered. Furthermore, if the facility is loaded directly on its top, the subgrade reaction increases. In particular, when the load is applied on a quaywall at its front end, the subgrade reaction at the front toe becomes significantly large. In quaywalls with a small width and short length, this tendency is significant and should be noted.

③ In ordinary sheet pile quaywalls, the maximum stress occurs between the tie rod installation point and the sea bottom; however, when a concentrated load acts on the area behind the sheet pile wall, the maximum stress may occur at the level of the tie rod installation point. However, this rarely causes an adverse effect on the embedded part of the sheet pile. It is preferable to provide a sufficient earth covering thickness for the tie rods to avoid adverse effects.

(2) Concrete Beams

① The reinforced concrete beams placed on rubble foundations laid on the ground shall ensure stability against bending moments, shear forces, and deflection, and their settlement shall be less than the limit value of the settlement.

② The characteristic values of the bending moments, shear forces, and deflection of the reinforced concrete beams placed on rubble foundations can be obtained from **equations (9.19.1)** to **(9.19.6)**. Here the variables with subscript *k* denote the characteristic values.

(a) In cases where loads act near the middle of a beam,

$$M_k = \sqrt[4]{\frac{E_c I_c}{64K}} \sum W_i e^{-\beta x_i} (\cos \beta x_i - \sin \beta x_i) \tag{9.19.1}$$

$$S_k = \frac{1}{2} \sum W_i e^{-\beta x_i} \cos \beta x_i \tag{9.19.2}$$

$$y = \sqrt[4]{\frac{1}{64E_cI_cK^3}}\sum W_i e^{-\beta x_i}(\cos\beta x_i + \sin\beta x_i) \quad \textbf{(9.19.3)}$$

(b) In cases where loads act on beams ends or junctions,

$$M = \sum \frac{W_i}{\beta} e^{-\beta x_i} \sin\beta x_i \quad \textbf{(9.19.4)}$$

$$S = \sum W_i e^{-\beta x_i}(\sin\beta x_i - \cos\beta x_i) \quad \textbf{(9.19.5)}$$

$$y = \sum \frac{2W_i\beta}{K} e^{-\beta x_i} \cos\beta x_i \quad \textbf{(9.19.6)}$$

where

M : bending moment on subject cross section (N·mm)

S : shearing force on subject cross section (N)

y : deflection on subject cross section (mm)

$$\beta = \sqrt[4]{\frac{K}{4E_cI_c}}$$

E_c : modulus of elasticity of concrete (N/mm^2)

W_i : wheel load (N)

I_c : moment of inertia of concrete foundation (mm^4)

K : modulus of elasticity of ground $K = Cb$ (N/mm^2)

C : pressure needed for a unit area of ground to settle by unit depth (N/mm^3)

b : bottom width of concrete beam (mm)

x_i : distance from wheel load point to subject section (mm)

③ The reinforced concrete beams placed on rubble foundations are assumed to be supported by the continuous elastic foundations of a uniform section over the entire length. In other words, it is assumed that the reaction forces of loaded beams are continuously distributed, and their strengths are directly proportional to the deflection at each point. By defining the bending moment generated at a point of a distance x from the traveling wheel as M and the deflection as y, M and y are expressed by **equations (9.19.7)** and **(9.19.8)** by elastic theory, respectively.[45), 46)]

$$M_k = W\sqrt[4]{\frac{E_cI_c}{64K}}e^{-\beta x}(\cos\beta x - \sin\beta x) = W\sqrt[4]{\frac{E_cI_c}{64K}}\phi_1 \quad \textbf{(9.19.7)}$$

$$y = \frac{W}{\sqrt[4]{64E_cI_cK^3}}e^{-\beta x}(\cos\beta x + \sin\beta x) = \frac{W}{\sqrt[4]{64E_cI_cK^3}}\phi_2 \quad \textbf{(9.19.8)}$$

When two or more wheels are close to each other, the bending moment directly under any one wheel is obtained from **equation (9.19.9)**.

$$M_{1k} = W_1\sqrt[4]{\frac{EI}{64K}} \quad \textbf{(9.19.9)}$$

By expressing the distance to another wheel as x_2 and ϕ_1 for βx_2 as ϕ_{12}, the bending moment is calculated from **equation (9.19.10)**.

$$M_{2k} = W_2\sqrt[4]{\frac{EI}{64K}}\phi_{12} \quad \textbf{(9.19.10)}$$

The resultant moment directly under the first wheel can be determined from $M = M_1 + M_2$. **equation (9.19.1)** can be derived from this expression. The deflection can be obtained in the same way. The values given by the following expression may be used for the values of C.[45), 47)]

$C = 5.0 \times 10^{-2}$ to 0.15 (N/mm^3)

④ **Verification of the Sliding of Concrete Beams**

In the verification of sliding of concrete beams, **equation (9.19.11)** may generally be used. In the following equations, γ represents a partial factor related to the subscript concerned, and subscripts k and d indicate the characteristic value and the design value, respectively.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$R_k = LR_P + f\left(LW_c + \sum W_i\right) \tag{9.19.11}$$
$$S_k = P + LE_A$$

where

P : horizontal force (action by earthquake ground motion, wind pressure) (kN)

E_P : passive earth pressure (kN/m)

E_A : active earth pressure (kN/m)

W_c : weight of concrete beam (kN/m)

ΣW_i : sum of wheel loads acting on subject calculation section (kN)

f : friction coefficient between rubbles and concrete $f_k = 0.6$

L : length of one block or 10 m, whichever is shorter (m)

In examining the sliding of a concrete beam, an appropriate value of 1.2 or more may be used as the modification coefficient m, and 1.0 may be used for all other partial factors.

⑤ In examining the bearing stresses on concrete beams exerted from between the rails and concrete, the stresses of rails, etc., refer to **9.19.3 Performance Verification of Pile-type Foundations, (1) Concrete Beams** in this chapter.

⑥ The subgrade reaction may be examined by assuming that the loads are equally distributed to a beam of one block concerned or 10 m in length, whichever it is shorter, and by referring to **[Part III: Facilities] Chapter 2, 3.2.2 Bearing Capacity of foundations on Sandy Ground**.

9.20 Transitional Parts[48)]

9.20.1 General

(1) Generally, transitional parts can be divided into the four types listed below.

① Part in which the front water depth changes
② Part in which different type structures are connected
③ Corner part
④ Transitional part to a breakwater, etc.

(2) In designing a transitional part, the items listed below shall be considered.

① **Natural conditions**

The ground near a transitional part can often change in complicated ways, so the ground conditions must be thoroughly understood. In addition, waves often concentrate in corners, so careful attention is required.

② **Differential settlement**

The structural types near transitional parts often vary. These parts may cause breakage due to differential settlements, particularly when making an installation on an existing facility on soft ground.

③ **Outflow of backfilled earth and sand**

When the structural types vary, backfilled earth and sand could flow out from the transitional part, so careful attention is required.

④ **Differences in rigidity**

When the structural types vary at a transitional part, deformation will vary due to differences in rigidity, which often becomes a cause of breakage.

⑤ **Relationship with existing facilities**

Attention shall be paid to transitional parts so that they do not affect the existing facilities. In addition, when a transitional part is expected to be extended in the future, it is desirable for attention to be paid to make future extensions easier.

(3) Attention shall be paid to construction procedures for transitional parts to prevent reworking and make it possible for the construction to follow the procedures. In addition, construction machines to be used are preferably the same as those used for the main body part. Construction methods that require completely different machines should be avoided.

9.20.2 Notes on Sections in Which the Front Water Depth Changes

(1) For sections in which the front water depth changes, the design conditions of the facility for which the depth of the connected water is deeper shall be used.

(2) For performance verification of facilities in sections in which the front water depth changes, the applicable structural types can be referred to.

(3) For sections in which the front water depth changes, there will be issues with determining the design conditions and the stability of the slope. Ideally, the seabed slope in the transitional section is preferably steep, especially from the viewpoint of utilization and cost. However, the seabed slope is preferably determined in consideration of a stable gradient based on the ground, the influence of waves, slope protection, slope gradient of dredging, and other factors. Usually, the seabed slope for sandy soil can be approximately 1:3.

(4) The design water depth can gradually be changed based on the stiffness of the facilities, deformation at the time of seismic vibrations (actions), cost, and other factors.

① When blocks are used, the design depth should ideally is preferably changed in stages as shown in **Fig. 9.20.1** using the height of the blocks as units.

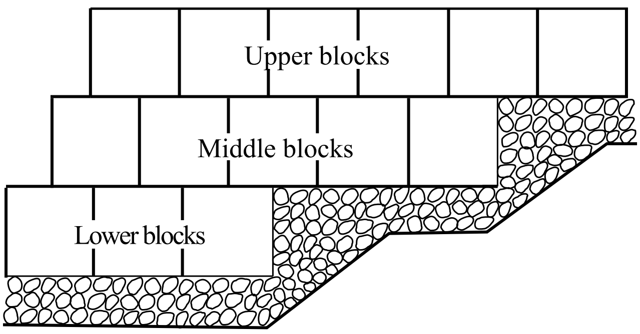


Fig. 9.20.1 Installation by Piling Blocks When the Front Water Depth Changes

② For a sheet pile structure, the design depth is often changed in stages by approximately 2 to 3 m as shown in **Fig. 9.20.2**.

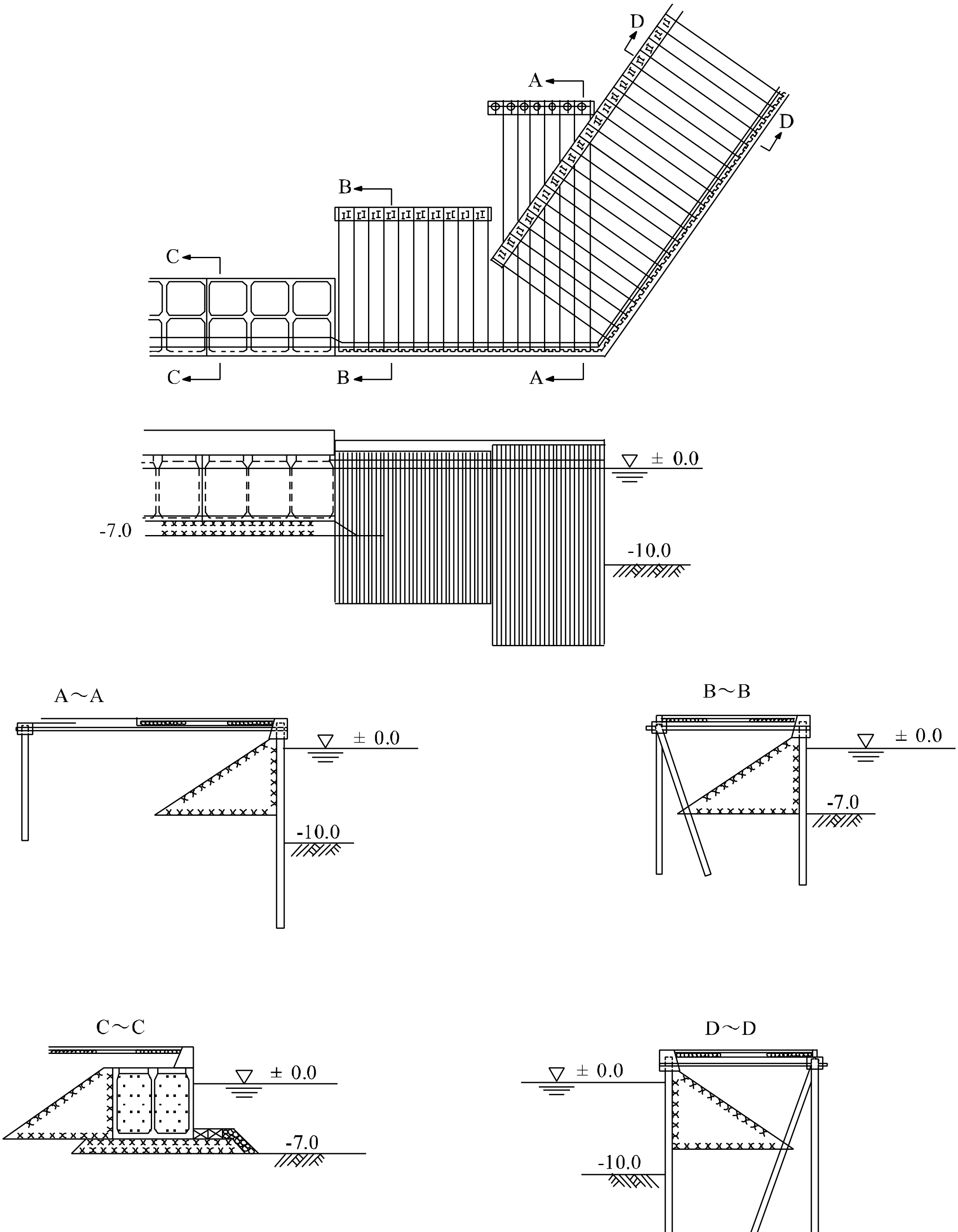


Fig. 9.20.2 Installation Using a Sheet Pile Structure When the Front Water Depth Changes

9.20.3 Notes on Sections in Which Different Facilities Are Connected

For sections in which different facilities are connected, there are cases where facilities having different structural types are directly connected, and cases where a connection facility is provided between facilities of different structural types. When an in-between connection facility is provided, the strictest design conditions from the two facilities in terms of stability should be applied.

9.20.4 Notes on Corner Sections

(1) For the facility design conditions for corner areas, the strictest design conditions from the two facilities in terms of stability should be applied.

(2) Sharp corners make configuration and construction difficult, so it is desirable to avoid making corners sharp as much as possible.

(3) When sheet pile structures are connected to each other

There are various design examples, but one issue is the type of shoring used. When the anchor slab type is used, an anchor slab may get into an active earth pressure area, or an area may be formed where passive earth pressures work against each other, so there is an issue with determining the earth pressure that acts as resistance. Therefore, it is desirable to avoid using the anchor slab type.

(4) Example cross sections of transitional parts that can be regarded as acceptable for the structures shown below.

In straight-pile shoring, the lateral resistance of the straight pile receives tension from the tie rod. In this case, it is desirable that the angle between the tie rod and sheet pile wall be a right angle. For performance verification, refer to **Part III, Chapter 5, 2.3 Sheet Pile Quaywalls**. **Fig. 9.20.3** illustrates example cross sections when straight-pile shoring was installed at a corner.

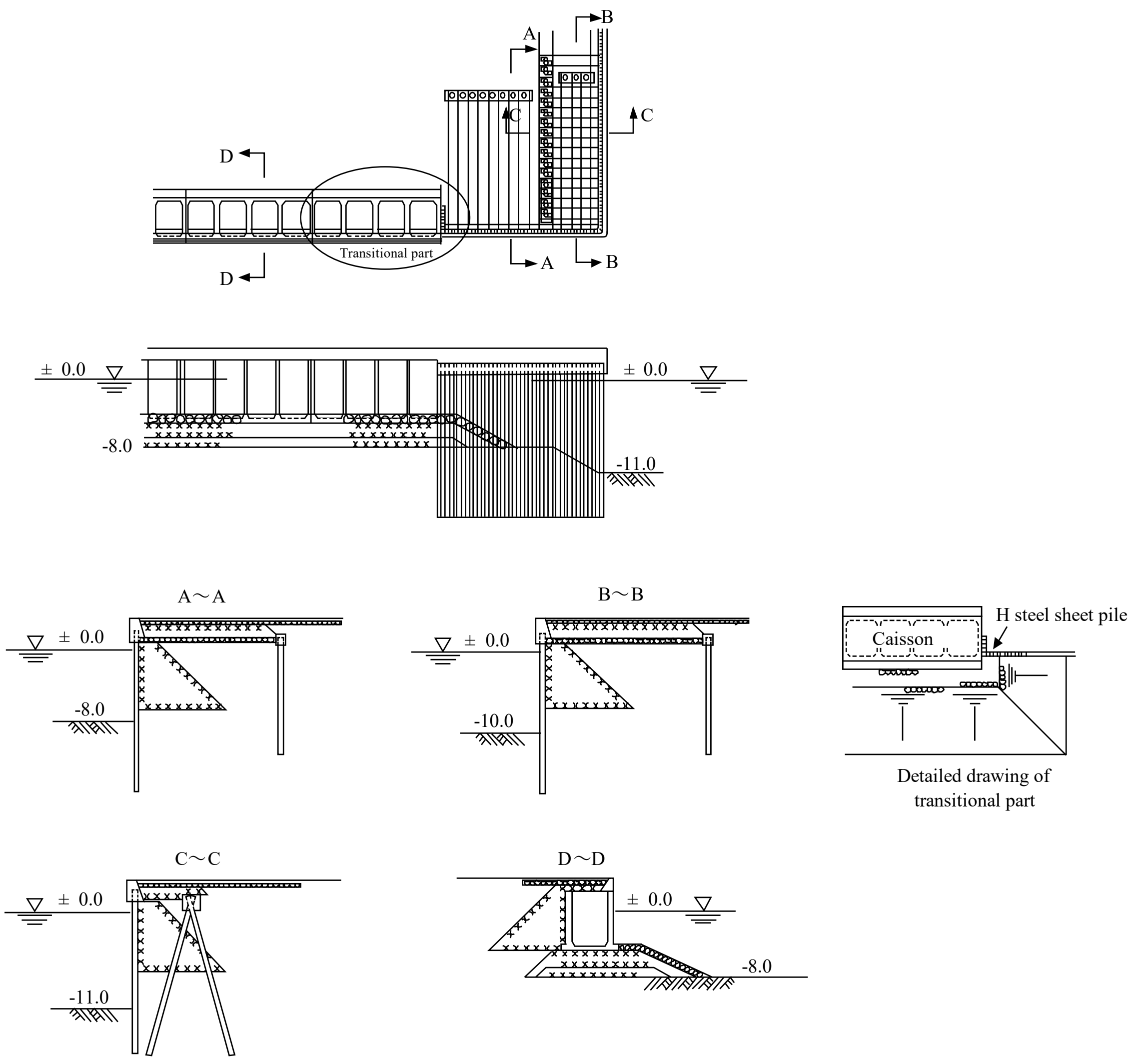


Fig. 9.20.3 Example Cross Sections When Straight-Pile Shoring is Installed at a Corner

② **When a structure with a relieving platform is installed at a corner**

(a) Structures with relieving platforms do not require complicated shoring, so they are used relatively frequently. For performance verification, the items listed below generally shall be considered.

1) For bending moments and axial forces, those in the x and y directions shown in **Fig. 9.20.4** and those in the resultant force direction shall be considered. Those in the resultant force direction can be calculated as a vector sum of the values calculated for the x and y directions.

2) For the embedment of piles, the embedment length of a pile that will be the most dangerous is preferably used for the others.

3) For performance verification of structures with relieving platforms, refer to **Part III, Chapter 5, 2.8 Quay Walls with Relieving Platforms**.

(b) **Fig. 9.20.5** illustrates example cross sections when a structure with a relieving platform was installed at a corner.

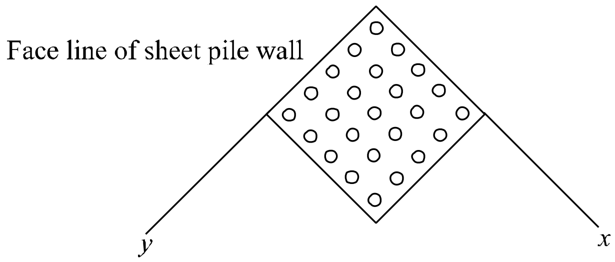


Fig. 9.20.4 When a Structure with a Relieving Platform is Installed at a Corner

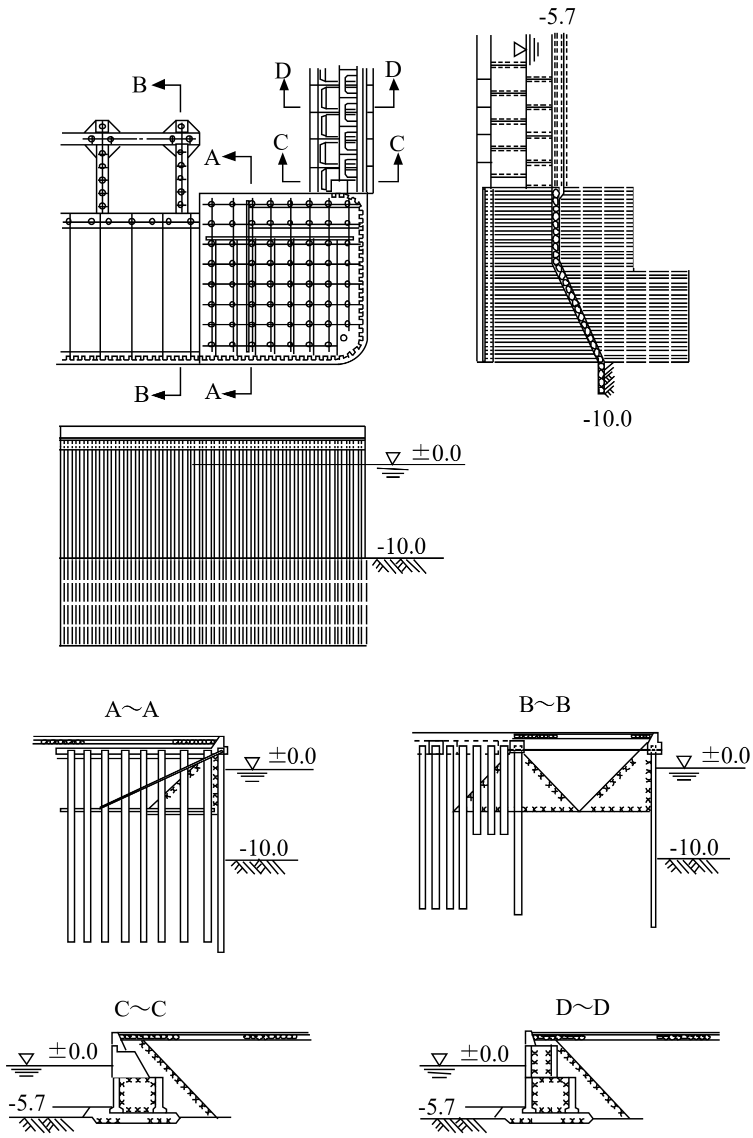


Fig. 9.20.5 Example Cross Sections When a Structure with a Relieving Platform is Installed at a Corner

③ **When a double sheet pile structure is installed at a corner**

Double sheet pile structures are often used particularly when the water depth is shallow. For performance verification of double sheet pile structures, refer to **Part III, Chapter 5, 2.7 Double Sheet Pile Quay Walls**.

④ **When a sheet pile structure is connected to a cantilevered sheet pile structure**

Cantilevered sheet pile structures do not require shoring, so they are sometimes used when the ground is fine or for wharves with shallow water depth (shallow draft wharves). In this combination, usually the displacement of the cantilevered sheet pile structure is larger than that of the sheet pile structure, and thereby a large force may be applied to the tie rod of the transitional part. Therefore, consideration is desirable, for example, when the rigidity of the cantilevered sheet pile near the transitional part is made higher, or when the tie rod is made thicker. For performance verification of cantilevered sheet pile structures, refer to **Part III, Chapter 5, 2.4 Cantilevered Sheet Pile Quaywalls**. **Fig. 9.20.6** illustrates example cross sections when a sheet pile structure and a cantilevered sheet pile structure are connected.

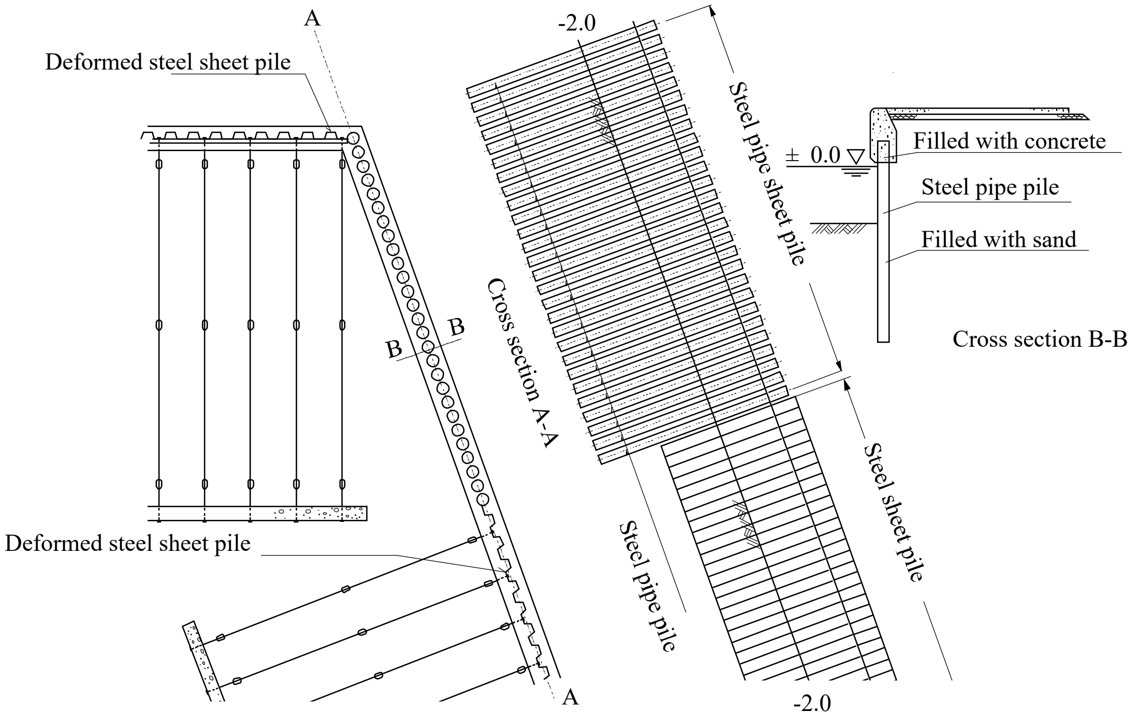


Fig. 9.20.6 Example Cross Sections When a Sheet Pile Structure and a Cantilevered Sheet Pile Structure are Connected

⑤ **When a caisson is installed at a corner**

When a caisson is installed at a corner, the force in the x and y directions shown in **Fig. 9.20.7** shall be considered, and it is desirable that the force in the resultant force direction (direction z) be considered. For performance verification of caissons, refer to **Part III, Chapter 5, 2.2 Gravity-Type Quaywalls**.

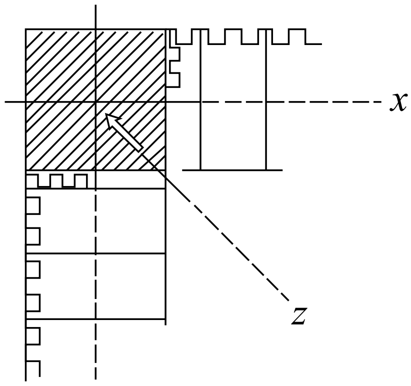


Fig. 9.20.7 When a Caisson is Installed at a Corner

⑥ **When a cellular-bulkhead structure is installed at a corner**

Methods to install a cellular-bulkhead structure or well at a corner are also available, but generally the construction costs tend to be high. For performance verification of cellular-bulkhead structures, refer to **Part III, Chapter 5, 2.9 Cellular-Bulkhead Quaywalls with Embedded Sections**. **Fig. 9.20.8** illustrates example cross sections when a well was installed at a corner.

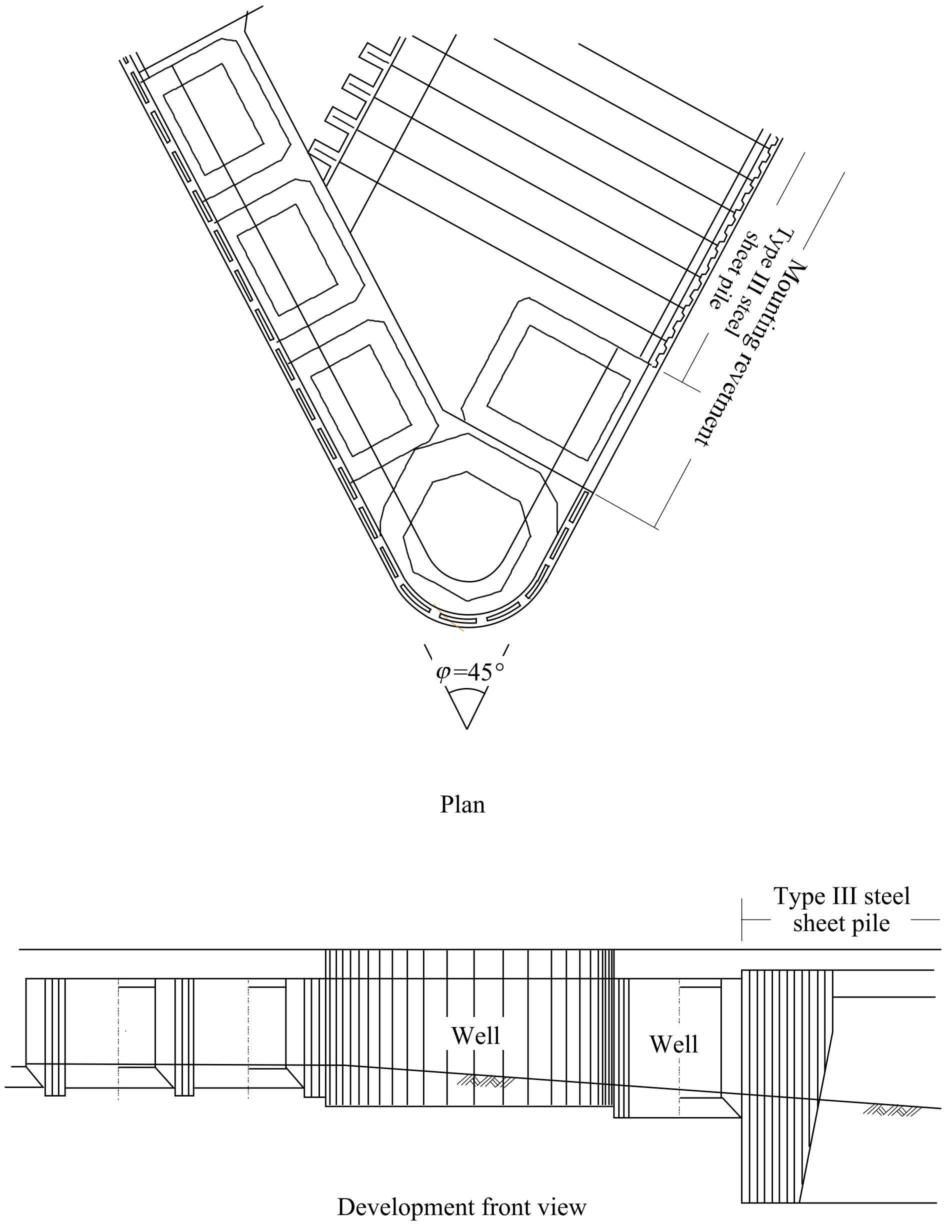


Fig. 9.20.8 Example Cross Sections When a Well is Installed at a Corner

[References]

1) Nishioka, S., S. Iyama, M. Miyata, H. Yoneyama, D. Tatsumi and H. Kihara: Study on Safety Consideration of Mooring and Detachment Work in Installation of Ancillary Equipment of Mooring Facilities, Technical Note of NILIM, No.957, 2017. (in Japanese)

2) Inagaki, M., K. Yamaguchi and T. Katayama: Standard Design of Mooring Post (Draft), Technical Note of PHRI, No.102, 1970. (in Japanese)

3) Japan Port Association: Standard Design of Port Structures (Vol.1), 1971. (in Japanese)

4) Yoneyama, H.: Study on Tractive Forces of Ships Acting on Mooring Posts and Bollards, Technical Note of PARI, No.1341, 2018. (in Japanese)

5) Ueda, S. and E. Ooi: On the Design of Fending Systems for Mooring Facilities in a Port, Technical Note of PHRI, No.596, 1987. (in Japanese)

6) Kitajima, S., H. Sakamoto, S. Kishi, T. Nakano and S. Kakizaki: On Some Problems Being Concerned with Preparation for the Design Standards on Port and Harbour Structures, Technical Note of PHRI, No.30, 1967. (in Japanese)

7) Japan Port Association: Examples of Design Calculation of Port Structures (Vol.1), pp.112-153, pp.257-300, 1992. (in Japanese)

8) Coastal Development Institute of Technology: Guideline for Design of SPS (Single Pile Structure), 1992. (in Japanese)

9) Kiuchi, S., M. Matsushita, M. Takahashi, M. Kakee, S. Isozaki and M. Suzuki: Full-scale loading tests on lateral resistance of single piles under horizontal loads in sand, Proceedings of Civil Engineering in the Ocean, Vol.6, pp.107-112, 1990. (in Japanese)

10) Kikuchi, Y., K. Takahashi and M. Suzuki: Lateral Resistance of Single Piles under Large Repeated Loads, Rept. of PHRI, Vol.31, No.4, pp.33-60, 1992. (in Japanese)

11) PIANC: Guidelines for the Design of Fender Systems: 2002, MarCom Report of WG 33, 2002.

12) PIANC: Guidelines for the Design of Fender Systems: 2002 (Japanese Version), MarCom Report of WG 33, 2005. (in Japanese)

13) Kawakami, M., H. Shinkawa, K. Tanaka and J. Kurasawa: Relation between structural strength of hull and fender, Report of School of Engineering, Hiroshima Univ., Vol. 24, Part I, pp.29-45, 1975. (in Japanese)

14) Tukayama, A.: Strength of ships for docking, Journal of Nippon Kaiji Kyokai, No.151, 1975. (in Japanese)

15) Nagasawa, J.: Berthing force and strength of outer plate of ship, Ships, Vol.40, No.3, pp.46-50, 1967. (in Japanese)

16) PIANC: Report of the International Commission for Improving the Design of Fender Systems, Supplement to Bulletin, No.45, 1984.

17) Vasco Costa, F.: The Berthing Ship: The effect of impact on the design of fenders and berthing structures, The Dock & Harbour Authority, Vol.XLV, No.523-525, 1964.

18) Ueda, S. and S. Shiraishi: On the Design of Fenders Based on the Ship Oscillations Moored to Quay Walls, Technical Note of PHRI, No.729, 1992. (in Japanese)

19) Japan Road Association: Standard and Commentary of Highway Lighting Facilities, Maruzen Publishing, 2007 (in Japanese)

20) Japanese National Committee of CIE: Lighting of Outdoor Work Places, 2005 (in Japanese)

21) Koga,Y.,Yoshida,K. ,Shigetomi,Y.,Ninomiya,T. Egashira,K. : TOWARDS THE NEW GENERATION OF LIGHTEING DESIGN FOR ADVANCED CONTAINER TERMINLS, pp. 468 -476, PROCEEDINGS of CIE 2016 “Lighting Quqality and Energy Ef.ciency”, 2016 .

22) Koga, Y.: Trend of Lighting in Ports, Kowan Niyaku, Vol.62 No.4, 2017 (in Japanese)

23) The illumination Engineering Institute of Japan: Maintenance rate for lighting design and maintenance planning, Technical Guideline of The illumination Engineering Institute of Japan,, JIEG-001, Maruzen, 1987

24) Coastal Development Institute of Technology: Car Stop Design Manual, 1994 (in Japanese)

25) Japan Road Association: Guidelines for Road Earth Work, 2009 (in Japanese)

26) Japan Road Association: Specifications for Highway Bridge Part I Common IV Substructure, 2017

27) Japan Road Association: Standards and Commentary for the Installation of Protection Fences 2016 Edition, 2016

28) SATO, T., KATO, E., KAWABATA, Y. and OKAZAKI, S.: Report on Cavity Detected in Back-Fill of Mooring Facilities, Journal of Japan Society of Civil Engineers, Ser. B3 (Ocean Engineering), Vol.70, No.2, pp.I_552-I_557, 2014. (in Japanese)

29) SATO, K., H. MORIGUCHI, T. ASAJIMA and H. SHIBUYA: Control of shrinkage Cracking of Concrete Pavements on Pier Slabs, Rept. of PHRI Vol. 14, No. 2, pp. 111-138, 1975 (in Japanese)

30) Japan Road Association: Pavement Design and Construction Guide, 2006. (in Japanese)

31) Japan Road Association: Manual for Pavement Design, 2006. (in Japanese)

32) Civil Aviation Bureau and National Institute for Land and Infrastructure Management in Ministry of Land, Infrastructure, Transport and Tourism: Manual for Airport Pavement Design, Specialists Center of Port and Airport Engineering, 2008. (in Japanese)

33) Civil Aviation Bureau and National Institute for Land and Infrastructure Management in Ministry of Land, Infrastructure, Transport and Tourism: Manual for Airport Pavement Rehabilitation, Specialists Center of Port and Airport Engineering, 2011. (in Japanese)

34) Ozawa, K. and S. Kitazawa: Setup method of deciding number of loads by cargo handling machine, in designing of pavement wharf apron, Technical Note of National Institute for Land and Infrastructure Management No.285, 2006 (in Japanese)

35) NAGAO,T., Hiroshi YOKOTA,Koichiro TAKECHI,Susumu KAWASAKI and Noboru OKUBO: Fatigue Limit State Design Method for Superstructures of Open Type Wharves in view of Cargo Handling Machine Loads, Rept. of PHRI Vol.37 No.2, pp.177-220, 1998 (in Japanese)

36) Sato, N.: Question and Answer of Pavement Technology, pp.257-259, 1991. (in Japanese)

37) Otsuka, Amano, Kamiya, Tominaga, Okada, Sato and Saitoh: Crack Prevention Measure for Semi-Flexible Pavement of International Flight Apron in Tokyo International Airport, Proceedings of Annual Conference of the Japan Society of Civil Engineers, Vol.70, V-316, 2015. (in Japanese)

38) Japan Association of Cargo-handling Machinery System: Survey report on standardization of related facilities, (6ht Report), 1998 (in Japanese)

39) Japan Association of Cargo-handling Machinery System: Survey report on standardization of related facilities, (5th Report), 1997 (in Japanese)

40) Japan Association of Cargo-handling Machinery System: Report of Survey and Study Committee of Container cargohandling facilities, 1993 (in Japanese)

41) Yokoyama, Y.: Design and construction of steel piles, Sankai-do Publishing, pp.99-11l, 1963 (in Japanese)

42) Japan Society of Mechanical Engineers: Mechanical Engineering Lectures Cargo handling equipment, p.239,1959 (in Japanese)

43) Minemura, Y.: Lecture note for rail connection and maintenance course, Japan Railway Maintenance Association, p,4,1958 (in Japanese)

44) KITAJIMA, S. and O. HORII: The Influence of Mobile cranes on Quaywalls, Technical Note of PHRI No.29, pp4-62, 1967 (in Japanese)

45) Kuniyuki, I.: Handbook of Cargo-handling Mechanical Engineering, Corona Publishing, ,p526,1961 (in Japanese)

46) Kitabatake, T., K. Katayama:Timoshenko’s Material Mechanics of material (Vol. 2), Corona Publishing, p.9,1955 (in Japanese)

47) Yasojima, Y: Railway track, Giho-do Publishing, p.302,1967 (in Japanese)

48) KATAYAMA, T. Muneaki SEGAWA, Ken-ichi FURUHATA and Yumiko MOMOSE: A Collection of Detail Design of Connected and Corner Part of Quay Wall, Technical Note of PHRI No.114, 1971 (in Japanese)

10. Mooring Facilities for Electric Power Generation Systems using Renewable Energy

(English translation of this section from Japanese version is currently being prepared.)

10.1 Fundamentals of Performance Verification

(English translation of this section from Japanese version is currently being prepared.)