Chapter 4 Protective Facilities for Harbors

1 General

[Ministerial Ordinance] (General Provisions)

Article 13

Protective facilities for harbors shall be installed at appropriate locations in light of geotechnical characteristics, meteorological characteristics, sea states, other environmental conditions, navigation channels, and other usage conditions of water areas around the facilities.

[Ministerial Ordinance] (Necessary Items concerning Protective Facilities for Harbors)

Article 24

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

[Public Notice] (Protective Facilities for Harbors)

Article 33

The items to be specified by the Public Notice under Article 24 of the Ministerial Ordinance concerning the performance requirements of protective facilities for harbors shall be as prescribed in the following Article through Article 46.

1.1 Purposes of Protective Facilities for Harbors

The purpose of protective facilities for harbors includes ensuring harbor calmness, maintaining water depths, preventing beach erosion, controlling the rise of water levels in the areas using protective facilities during storm surges, diminishing invading waves by tsunamis and protecting harbor facilities and hinterland from waves, storm surges, and tsunamis.

In the deliberation of the measures against tsunamis and storm surges for harbors, it is necessary to appropriately set the targets of protecting harbors according to the magnitude and occurrence frequency of tsunamis and storm surges, after considering sufficiently their impacts on human lives, property and socioeconomic activities.

Recently, there has been demand for water intimate amenity functions enabling people to enjoy the proximity to marine environment and play with water. In general, many protective facilities for harbors are provided with additional facilities to fulfill some of these functions. Accordingly, the performance verification shall consider the usability enabling each protective facility for harbors to fulfill these purposes.

1.2 Points of Caution When Constructing Protective Facilities for Harbors

(1) When constructing protective facilities for harbors, their layouts and structural types shall be decided after carefully considering the influences that will be exerted on the nearby water areas, facilities, topography, and water currents. The influences caused by the protective facilities for harbors are as follows:

① When the protective facilities for harbors are constructed on sandy coasts, they may cause various topographic changes to the surrounding area such as beach accretion or erosion.

② Construction of breakwaters may increase the wave heights outside the protective facilities for harbors because of reflected waves.

③ The calmness of water areas inside of harbors may be disturbed because of the multiple wave reflection triggered by the construction of new protective facilities for harbors or the induction of harbor resonance due to the changes in harbor shapes.

④ Construction of the protective facilities for harbors may bring about changes in the surrounding tidal currents or flow conditions in rivers, thereby causing localized changes in water quality.

(2) The damage to protective facilities for harbors may affect the safety of ships in harbors, mooring facilities, and other facilities in hinterland. Thus, during the construction, improvement, and maintenance of protective facilities for harbors, sufficient deliberation is required for the protection of such damage according to the required performance of respective ships in harbors, mooring facilities, and other facilities in hinterland.

1.3 Role of Protective Facilities for Harbor in Multilevel Protection Concept

In the deliberation of the measures against tsunamis and storm surges in harbors, it is important to adopt the concept of providing protection at multiple levels (multilevel protection) by the entire harbor facilities including forefront breakwaters and seawalls while taking full functions of existing stocks.

Considering that most industrial and logistic facilities in harbors are located at seaward side of protection lines protecting urban areas at the back of harbor areas from disasters, it is effective to use harbor facilities such as breakwaters for protecting the functions of the industrial and logistic facilities from tsunamis and storm surges.

However, because there are cases where protective facilities for harbors such as forefront breakwaters and seawalls may not be able to completely protect the functions from tsunamis and storm surges, it is important to establish plans to evacuate people working in industrial and logistic facilities in harbors and harbor users.

1.4 Environmentally conscious protective facilities for harbors

Because the protective facilities for harbors also provide the habitat for marine organisms such as fish, marine plants, and plankton, it is preferable to determine the layouts and structural types of the facilities, with considering the habitat for these marine organisms as needed.

When installing protective facilities for harbors close to natural park districts or cultural facilities, it is preferable to consider not only the original functions of the protective facilities for harbors but also their external appearances such as shapes and colors. Additionally, in situations where water intimate amenity functions will be added to the protective facilities for harbors, convenience and safety of users must also be considered.

2 Common Items for Breakwaters

[Ministerial Ordinance] (Performance Requirements for Breakwaters)

Article 14

1 The performance requirements for breakwaters shall be as prescribed in the following items depending on the structure type for the purpose of securing safe navigation, anchorage and mooring of ships, ensuring smooth cargo handling, and preventing damage to buildings, structures, and other facilities in the port by maintaining the calmness in the harbor water area.

(1) Breakwaters shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable the reduction of the height of waves intruding into the harbor.

(2) Damage to breakwaters, etc. due to self-weight, variable waves, Level 1 earthquake ground motion, etc. shall not impair the functions of the breakwaters and shall not adversely affect the continuous use of the breakaters.

2 In addition to the provisions of the preceding paragraph, the performance requirements for the breakwaters in the following items shall be specified respectively in those items:

(1) "Performance requirements for breakwaters which are required to protect the hinterland of the breakwaters from storm surges or design tsunamis" shall be such that the breakwaters satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable the appropriate reduction of the rise in water level and flow velocity due to storm surges or design tsunamis in the harbor.

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

(3) "Performance requirements for breakwaters to be utilized by an unspecified large number of people" shall be such that the breakwaters satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to ensure the safety of the users of the breakwaters.

(4) "Performance requirements for breakwaters in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity if they are stricken by disaster" shall be such that damage to breakwaters, etc. due to design tsunamis, accidental waves, Level 2 earthquake ground motions, etc. does not have a serious impact on the structural stability of the breakwaters in consideration of the structure type even in cases where functions of the breakwaters are impaired. Provided, however, that in cases where performance requirements for the breakwaters which are required to protect the hinterland of the breakwaters from design tsunamis, the damage due to design tsunamis, Level 2 earthquake ground motion, etc. shall not adversely affect restoration through minor repair works of the functions of the breakwaters.

3 In addition to the provisions of the preceding two paragraphs, the performance requirements for breakwaters in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity, shall be such that a serious impact on the structual stability of the breakwaters caused by damage, etc. due to the actions of the tsunamis, etc. even in cases where tsunami with intensity exceeding the design tsunami occurs at the place where the breakwaters are located, shall be delayed as much as possible in consideration of the structure type.

[Public Notice] (Performance Criteria for Breakwaters)

Article 34

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

(1) Breakwaters shall be located appropriately so as to satisfy the harbor calmness provided in Article 31, item (iii), and shall have the dimensions which enable the transmitted wave height to be equal to or less than the allowable level.

(2) Breakwaters having wave-absorbing structures shall have the dimensions which enable full performance of the required wave-absorbing function.

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

(1) "Performance criteria for breakwaters which are required to protect the hinterland from storm surges" shall be such that the breakwaters are located appropriately so as to reduce the rise of water level and flow velocity in the harbor due to storm surges and have the dimensions necessary for functions of breakwaters.

(2) "Performance criteria for breakwaters required to protect the hinterland from design tsunamis" shall be such that the breakwaters are located appropriately so as to reduce the rise of water level and flow velocity in the harbor due to design tsunamis and have the dimensions necessary for functions of breakwaters.

(3) "Performance criteria for breakwaters for the purpose of environmental conservation" shall be such that the breakwaters shall have the necessary dimensions so that they can contribute to conservation of the environment of ports without impairing their original functions in consideration of the environmental conditions, etc. to which the facilities are subjected.

(4) "Performance criteria for breakwaters utilized by an unspecified large number of people" shall be such that breakwaters have the dimensions necessary to secure the safety of users in consideration of the environmental conditions, usage conditions, etc. to which the facilities are subjected.

(5) "Performance criteria for breakwaters in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity by the damage to the breakwaters" shall be such that the degree of damage under the accidental situation, in which the dominating actions are design tsunamis, accidental waves, or Level 2 earthquake ground motions, is equal to or less than the threshold level in consideration of the performance requirements.

[Interpretation]

10. Protective Facilities for Harbors

(1) Performance Criteria Common to Breakwaters (Article 14, Paragraph 1 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 1 of the Public Notice)

① Breakwaters shall have serviceability as their common performance requirement. The term "serviceability" refers to that the harbor calmness in the ports is secured.

② The dimensions for securing harbor calmness shall indicate a structure including shape and crown height that affects the transmitted wave height or transmission ratio of waves. In setting the crown height in the performance verifications of breakwaters, appropriate consideration shall be given to the effect of ground settlement.

③ The allowable transmitted wave height is the limit value of the wave height of waves transmitted from outside the harbor to inside the harbor over the breakwaters. Provided, however, that the index of the limit value in the performance verifications is not limited to the transmitted wave height, but also includes cases in which the wave transmission ratio is used.

④ In the performance verifications of breakwaters, the allowable transmitted wave height or wave transmission ratio shall be set appropriately to secure harbor calmness. Furthermore, the allowable transmitted wave height or wave transmission ratio shall generally be calculated considering the type of structure and crown height of the breakwater.

(2) Performance Criteria for Specific Breakwaters

① Storm surge protection breakwaters (Article 14, Paragraph 2 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 1 of the Public Notice)

(a) Storm surge protection breakwaters refer to breakwaters that shall protect the hinterland from storm surges. In addition to the provisions common to breakwaters, the items listed below apply to them.

(b) Storm surge protection breakwaters shall have serviceability as their performance requirement. The term "serviceability" refers to the performance that demonstrates a peak cut effect in reducing the rise of the water level and flows of water due to storm surges.

(c) The dimensions of storm surge protection breakwaters shall indicate the crown height, opening width, and water depth at the opening. In setting the layout, crown height, opening width, and water depth at the opening in performance verifications of storm surge protection breakwaters, appropriate consideration shall be given to the effect of storm surge and tide levels such that the performance above is demonstrated.

② **Tsunami protection breakwaters** (Article 14, Paragraph 2 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 2 of the Public Notice)

(a) Tsunami protection breakwaters refer to breakwaters that shall protect the hinterland from design tsunamis.

(b) Tsunami protection breakwaters shall have serviceability as their performance requirement. The term "serviceability" refers to the performance that demonstrates a peak cut effect in reducing the rise of the water level and flows of water due to tsunamis.

(c) The dimensions of tsunami protection breakwaters shall indicate the crown height, opening width, and water depth at the opening. In setting the layout, crown height, opening width, and water depth at the opening in the performance verifications of tsunami protection breakwaters, appropriate consideration shall be given to the effect of tsunamis and tidal levels such that the performance above is demonstrated.

③ **Symbiosis breakwaters** (Article 14, Paragraph 2, Item 2 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 3 of the Public Notice)

(a) Symbiosis breakwaters refer to breakwaters that aim at preserving the environments. In addition to the provisions common to breakwaters, the items listed below apply to them.

(b) Symbiosis breakwaters shall have serviceability as their performance requirement. The term "serviceability" refers to the performance that contributes to the preservation of the port environments (e.g., living things and ecosystems) without impairing original functions of breakwaters.

(c) The dimensions of breakwaters that aim at preserving the environments refer to the structure, cross-sectional dimensions, and ancillary facilities. In setting the structure and cross-sectional dimensions in the performance verifications of breakwaters that aim at preserving the environments and in installing ancillary equipment, appropriate consideration shall be given to factors that affect the targets that contribute to the preservation of the port environments (e.g., living things and ecosystems) without impairing original functions of the breakwaters.

④ **Amenity-oriented breakwaters** (Article 14, Paragraph 2, Item 3 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 4 of the Public Notice)

(a) Amenity-oriented breakwaters refer to breakwaters that an unspecified large number of people use. In addition to the provisions common to breakwaters, the items listed below apply to them.

(b) Amenity-oriented breakwaters shall have serviceability as their performance requirement. The term "serviceability" refers to the performance that can secure the user safety in consideration of the environmental conditions to which the revetments concerned are subjected, their utilization conditions, and other conditions.

(c) The dimensions of amenity-oriented breakwaters shall indicate the structure, cross-sectional dimensions, and ancillary facilities. In setting the structure and cross-sectional dimensions in the performance verifications of amenity-oriented breakwaters and in installing ancillary equipment, consideration shall be given to the effects of wave overtopping and spray, prevention of slipping, overturning, and falling into the water of users, smooth execution of rescue activities for users who fall into the water, and ancillary equipment such as falling prevention fences shall be installed appropriately.

⑤ **Breakwaters of facilities prepared for accidental incidents** (Article 14, Paragraph 2, Item 4 and Paragraph 3 of the Ministerial Ordinance and the interpretation related to Article 34, Paragraph 2, Item 5 of the Public Notice)

(a) Breakwaters of facilities prepared for accidental incidents refer to breakwaters in the place where there is a risk of serious impact on human lives, properties, or socioeconomic activities by the damage.

(b) Breakwaters of facilities prepared for accidental incidents (excluding tsunami protection breakwaters) shall have safety as their performance requirement in accidental situations where the dominating actions are Level 2 earthquake ground motions, design tsunamis, and accidental waves.

Attached Table 10-1 shows performance verification items and standard indexes to determine limit values for such actions. Necessary performance verification items shall be appropriately selected depending on the type of structure of the breakwater concerned. The reason for indicating “damage” in the “Performance verification item” column of **Attached Table 10-1** is that it is necessary to use a comprehensive term considering that the performance verification items will vary depending on the type of structure. Indexes to determine limit values shall be appropriately determined in the performance verifications.

Attached Table 10-1 Performance Verification Items and Standard Indexes to Determine Limit Values for Accidental Actions on Breakwaters of Facilities Prepared for Accidental Incidents (excluding Tsunami Protection Breakwaters)

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Performance verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
14	2	4	34	2	5	Safety	Accidental	L2 earthquake ground motion [Accidental wave]	Self-weight, water-pressure	Damage	-
								Design tsunami	Self-weight, water pressure, water flows		

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

(c) Tsunami protection breakwaters shall have restorability as their performance requirement against accidental situations where the dominating actions are Level 2 earthquake ground motion and design tsunamis. Also, Tsunami protection breakwaters shall have safety as their performance requirement in accidental situations where the dominating actions are accidental waves. **Table 10-2** shows performance verification items and standard indexes to determine limit values for such actions. Necessary performance verification items shall be appropriately selected depending on the type of structure of the breakwater concerned. Indexes to determine limit values shall be appropriately determined in the performance verifications.

Attached Table 10-2 Performance Verification Items and Standard Indexes to Determine Limit Values for Accidental Actions on Tsunami Protection Breakwaters among Breakwaters of Facilities Prepared for Accidental Incidents

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
14	2	4	34	2	5	Restorability	Accidental	Level 2 earthquake ground motion	Self-weight, water pressure	Deformation of breakwater body	Residual deformation
								Design tsunami	Self-weight, water pressure, water flows	Sliding and overturning of breakwater body, bearing capacity of foundation ground	Action-resistance ratio of sliding Action-resistance ratio of overturning Action-resistance ratio of bearing capacity

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
						Safety		Accidental waves	Self weight, water pressure	Sliding and overturning of breakwater body, bearing capacity of foundation ground	Action-resistance ratio of sliding Action-resistance ratio of overturning Action-resistance ratio of bearing capacity

(d) It may be noted that, as the performance criteria in connection with the accidental situations that are common to breakwaters of facilities prepared for accidental incidents, in addition to these provisions, the settings in connection with the Public Notice, Article 22 Performance Criteria Common to Members Comprising Facilities subject to the Technical Standards shall be applied as necessary.

(e) The structure of breakwaters of facilities prepared for accidental incidents shall be well configured to allow them to be as much stable as possible even when they are subjected to actions (e.g., tsunami with intensity exceeding the design tsunami) at places where they are installed so that the disaster mitigation effects can be demonstrated and the harbor calmness is secured immediately after the disaster.

2.1 Matters relating to Breakwaters with Basic Functions

2.1.1 General

(1) Breakwaters are generally classified as shown in **Fig. 2.1.1** by the type of structure and functions or purposes. The characteristics of each structural type are described in the applicable section for each type.

- Breakwater
  - Ordinary breakwater (breakwater having basic functions)
  - Breakwater with combination of added functions
    - Symbiosis breakwaters
    - Amenity-oriented breakwater
    - Tsunami protection breakwater
    - Storm surge protection breakwater
    - Breakwater for timber handling facilities

(a) Classification by function

- Breakwater
  - Composite breakwater
  - Upright breakwater
  - Sloping breakwater
  - Breakwater covered with wave-dissipating blocks
  - Gravity type special breakwater
    - Upright wave-dissipating block type breakwater
    - Wave-dissipating caisson type breakwater
    - Sloping top caisson breakwater
  - Pile type breakwater
  - Floating breakwater
  - Breakwater with wide footing on soft ground

Types of structures enclosed in broke line are gravity type breakwaters.

(b) Classification by type of structure

Fig. 2.1.1 Classification of Breakwaters

(2) In design and performance verifications of breakwaters, it is preferable to consider their layout, effects on the topographical features of the surrounding areas, harmonization with the surrounding environments, design conditions, structural type, whether they are used for multiple purposes, verification procedures, construction methods, and economy.

(3) Maintenance of harbor calmness shall be examined from the following two viewpoints: the enabling of cargo handling in the basin and the condition of waves enabling refuge during rough weather. For harbor calmness in the basin and the condition of waves during rough weather, **Part II, Chapter 2, 4.6 Concept of Harbor Calmness** and **Part III, Chapter 3, 3 Basins** can be used as references.

(4) Reflected waves from a breakwater may become large depending on its structure, which may hinder ships from sailing outside the port. Such large waves affect small-sized ships significantly, in particular, so it is preferable to adopt structure with lesser reflected waves depending on the sailing pattern.

2.1.2 Layout

(1) Breakwaters shall be appropriately arranged to keep the inside of the ports, such as waterways and basins, calm.

(2) Breakwaters are constructed to maintain the harbor calmness, facilitate smooth cargo handling, ensure the safety of ships during navigation or anchorage, and protect facilities in ports. To fulfill these requirements, the following objectives must be met:

① Breakwaters should be located such that the harbor entrance is at the location not facing the direction of the most frequent waves and the direction of the highest waves to reduce waves entering into the harbor.

② Breakwater alignment should be arranged to protect the harbor from the most frequent waves and the highest waves.

③ The harbor entrance should have a sufficient effective width so that it will not present an obstacle to ship navigation, and it should orient the navigation channel in a direction that makes navigation easy.

④ Breakwaters should be located at the place where the speed of tidal currents is as slow as possible. In cases in which the speed of tidal currents is high, it is necessary to take appropriate countermeasures.

⑤ The influences of reflected waves, Mach-stem waves, and wave concentration on the waterways and basins should be minimized.

⑥ Breakwaters should enclose a sufficiently large water area that is needed for ship berthing, cargo handling, and ship anchorage.

These objectives are also mutually contradictory goals, however. A narrow harbor entrance width, for example, is best to achieve calmness in a harbor, but is inconvenient for navigation. The direction of the most frequent waves and the direction of the highest waves are not necessarily the same. In such a situation, the breakwater layout should be determined through a comprehensive investigation of all the factors such as conditions of ship use, construction cost, construction works, and ease or difficulty of maintenance.

(3) In situations in which concerns for deterioration of water quality exist, consideration is preferably given to the exchangeability of seawater with the outside sea so that seawater within the harbor does not stagnate.

(4) In the construction of breakwaters, economy should also be examined considering the natural conditions and construction conditions. In particular, it is preferable to consider the following.

① Layouts that cause wave concentrations should be avoided.

② Locations where the ground is extremely poor should be avoided, considering constructability and economy.

③ The layout should consider the effects of topographical features such as capes and islands.

④ On sandy beaches, the layout should consider invasion of littoral drift into the harbor.

⑤ Adequate consideration should be given to the effect on adjacent areas after the construction of the breakwater.

For wave concentration, **Part II, Chapter 2, 4.4.4 Wave Reflection, [3] Transformation of Waves at Concave Corners near the Heads of Breakwaters and around Detached Breakwaters** can be used as a reference; for breakwaters to be constructed on sandy beaches, **Part II, Chapter 2, 7.4 Littoral Drift** can be used as a reference.

(5) Breakwaters should be located such that they do not form an obstacle to the future development of the harbor.

(6) The "effective harbor entrance width" means the width of the waterway at the specified water depth, not merely the width of the harbor entrance. The speed of the tidal currents cutting across the harbor entrance is preferably 2 to 3 knots or less.

(7) In the areas surrounding shoals, the wave height often increases because of wave refraction. In some cases, impact wave forces will act on the breakwater constructed on a seabed with steep slope. It should be noted that a significantly large structure may be required when a breakwater is placed on or directly behind a shoal.

(8) For detached breakwaters that are to be constructed in isolation offshore, if the length of the breakwater is equal to or less than several times that of the incident waves, the distribution of the wave heights behind the breakwater will fluctuate greatly because of the effect of diffracted waves from the two ends of the breakwater, which will affect the stability of the breakwater body; therefore, exercising caution is necessary. For the effects of diffracted waves, **Part II, Chapter 2, 4.4.2 Wave Diffraction** and **Part II, Chapter 2, 4.4.4 Wave Reflection, [3] Transformation of Waves at Concave Corners near the Heads of Breakwaters and around Detached Breakwaters** can be used as references.

2.1.3 Selection of Structural Type and Setting of Cross Section

(1) In setting the cross sections of breakwaters, the type of structure shall be selected based on a comparative examination of the layout conditions, natural conditions, use conditions, importance, construction conditions, economy, term of construction work, ease of obtaining materials, and ease of maintenance, considering the features of respective types of structures.

(2) In determining the cross-sectional dimensions of the wave-dissipating work in the wave-dissipating function of a breakwater, it is necessary to provide adequate consideration to hydraulic characteristics so that the specified wave-dissipating function is demonstrated. In particular, it is preferable that the crown height of the wave-dissipating section be approximately the same as that of the breakwater body so that impulsive breaking wave pressure will not act on the breakwater body.

(3) In cases in which the layout of a breakwater includes a corner, the wave height around the corner will increase. Therefore, it is preferable to adopt a low reflective structure around corners.

(4) The crown section of a submerged breakwater installed at an opening may be damaged by the dragging power of tsunamis and waves and the foundation mound and the ground under it may be scoured by formed flow of seawater, etc. passing through the foundation mound, etc. Therefore, appropriate scour prevention measures are required, when needed.

(5) It shall be noted that when the seawater's permeability becomes higher due to the structure of the foundation, etc., the effect for reducing storm surges becomes smaller. In addition, the difference in the tide levels between the inside and outside of the port due to storm surges may form the flow of seawater, etc. that pass through the inside of the foundation mound, etc. Such flow may scour the ground at the lower section of the foundation mound, etc. Therefore, appropriate scour prevention measures are required, when needed.

(6) Selection of a permeable-type breakwater structure is advantageous for promoting circulation of seawater in the harbor. However, because this also invites inflow of littoral drift and an increase in the height of transmitted waves, adequate consideration of the merits and demerits is necessary.

(7) Breakwaters become important bases for the life of living beings inside and outside the ports and bases to which they are attached in some cases. Therefore, in setting the structure and cross-sectional dimensions of breakwaters, they may be designed considering that the port environment is well preserved.[1), 2), 3), 4), 5), 6), 7), 8), 9), 10)] (**Reference (Part I), Chapter 3, 2 Symbiosis Port Structures**)

2.1.4 Matters to be Considered to Maintain the Harbor Calmness in Ports

(1) In the installation of breakwaters, the crown height of the breakwater, relationship between the position of the breakwater and waterways and basins, and position and direction of the harbor entrance should be examined so as to maintain the harbor calmness necessary for cargo handling and refuge. In the performance verifications of the harbor calmness of basins, **Part II, Chapter 2, 4.6 Concept of Harbor Calmness** can be used as a reference. Furthermore, it is preferable that conditions be set to enable protection of the port facilities behind the breakwater, including during typhoons and other rough weather.

(2) The crown height of a breakwater necessary in securing harbor calmness can generally be set to an appropriate height at least 0.6 times the significant wave height ($H_{1/3}$) used in examination of the safety of the breakwater above the mean monthly-highest water level. In this case, the appropriate height is set considering harbor calmness in the basin behind the breakwater, preservation of facilities in the harbor behind the breakwater, and other factors. In the existing breakwaters, there are many examples in which the crown height is determined as follows.

① In a harbor where large ships call, where the water area behind the breakwater is so wide that wave overtopping is allowed to some extent, the crown height is set at $0.6H_{1/3}$ above the mean monthly-highest water level in situations in which it is not necessary to consider the influence of storm surges.

② In a harbor where the water area behind the breakwater is small and is used for small ships, overtopping waves should be prevented as much as possible. Hence, the crown height is set at $1.25H_{1/3}$ above the mean monthly-highest water level.

③ The crown height values above are often seen in design examples in the past. Documents 11) and 12) show examination results of actual breakwater crown height determined based on such height values, volume of wave overtopping, and transmission ratio and can be referred to.

(3) For ports for which effects of storm surges shall be considered, the tide level that is calculated by adding appropriate deviation to the mean monthly-highest water level based on the past records shall preferably be used as a reference surface to calculate the crown height.

(4) Even in case of a harbor where large ships call which has a wide water area behind the breakwaters at the harbor where large storm waves close to the design waves attack frequently with long duration, the activities of harbor may be limited by the influence of waves overtopping the breakwaters, if the crown height is set at of $0.6H_{1/3}$ above

the mean monthly-highest water level. Accordingly, in such a harbor, the crown height is preferably set higher than $0.6H_{1/3}$ above the mean monthly-highest water level.

(5) For the effects of reflected waves on the harbor calmness in ports, **Part II, Chapter 2, 4.4.4 Wave Reflection** can be used as a reference.

2.2 Matters relating to Breakwaters of Facilities Prepared for Accidental Incidents

The descriptions provided below shall be referred to for breakwaters of facilities prepared for accidental incidents.

(1) Gravity-type Breakwaters of Facilities Prepared for Accidental Incidents

① Accidental situations where the dominating actions are Level 2 earthquake ground motions

(a) Deformation volume

When the limit value of the degree of damage in accidental situations in which the dominating actions are Level 2 earthquake ground motions is used as the breakwater body's deformation volume, the breakwater body's allowable residual deformation volume shall be appropriately determined. In setting the allowable residual deformation volume, the degree of allowable damage can be a level at which the breakwater body does not fall down, it does not slide down from the foundation mound, and settlement more than the allowable value does not occur.

② Accidental states where the dominating actions are design tsunamis

(a) Consideration of the effects of earthquake ground motions

In performance verifications for design tsunamis, when supposed design tsunamis are caused by an earthquake for which the hypocenter is near the facility concerned, it shall be appropriately considered that the facility concerned is subject to actions by the earthquake ground motions before receiving actions by the design tsunamis. In this case, performance verifications for the design tsunamis shall be performed considering the effects of the actions of the earthquake ground motions that come before the design tsunamis. It shall be noted that the assumed earthquake ground motions coming before design tsunamis in such a case are not always equal to Level 2 earthquake ground motions.

(b) Points to note in performance verifications of breakwaters

In setting limit values of the degrees of damage in accidental situations where the dominating actions are design tsunamis, how protective facilities for the harbors (e.g., revetments on the back and floodgates) and other facilities in the vicinity have been maintained, software measures for disaster prevention and mitigation in the areas, and other factors shall be comprehensively considered, in addition to the functions of the breakwaters.

(c) Stability against design tsunamis and tsunamis with the intensity exceeding the design tsunamis

For tsunami-resistant design in which the stability against design tsunamis and tsunamis with the intensity exceeding the design tsunamis is considered, one can refer to the **Guideline for Tsunami-Resistant Design of Breakwaters**.[13] However, the guideline handles composite breakwaters and breakwaters covered with wave-dissipating blocks as the target structural types. Therefore, appropriate consideration is necessary for other structural types.

③ Accidental states where the dominating actions are accidental waves

(a) Consideration of the effects of storm surges

In performance verifications for accidental waves, storm surges that are caused at the same time with supposing waves shall be appropriately considered. In setting accidental wave conditions, **Part II, Chapter 2, 4.1.2 Setting of Wave Conditions for Verification of Serviceability of the Structural Members** and **Part II, Chapter 2, 3.2 Storm Surges** can be referred to.

(b) Points to note in performance verifications of breakwaters

In setting limit values of the degrees of damage in accidental situations where the dominating actions are accidental waves, how protective facilities for the harbors (e.g., revetments on the back and floodgates) and other facilities in the vicinity have been maintained, software measures for disaster prevention and mitigation in the areas, and other factors shall be comprehensively considered, in addition to the functions of the breakwaters.

(2) Floating Breakwaters of Facilities Prepared for Accidental Incidents

① Points to note in performance verifications of breakwaters

In verifications of the stability of mooring anchors and other equipment against accidental situations where the dominating actions are design tsunamis and accidental waves, consideration is necessary to prevent the floating structures from drifting because of design tsunamis and accidental waves and thereby significantly affecting the sounding areas.

[References]

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3 Ordinary Breakwaters

3.1 Gravity-type Breakwaters (Composite Breakwaters)

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

Article 35

The performance criteria for gravity-type breakwaters are prescribed respectively in the following items:

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

(2) Under the variable situation , in which the dominating actions are variable waves and Level 1 earthquake ground motion, the risk of failures due to the sliding and overturning of breakwater body and the insufficient bearing capacity of the foundation ground shall be equal to or less than the threshold level.

[Interpretation]

10. Protective Facilities for Harbors

(3) Performance Criteria of Gravity-type Breakwaters (Article 14, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 35 of the Public Notice)

① The required performance of gravity-type breakwaters under the permanent action situation in which the dominant action is self-weight and the variable action situation in which the dominant actions are variable waves and Level 1 earthquake ground motions shall focus on usability. The performance verification items and standard indexes to determine the limit values with respect to the actions shall be those shown in **Attached Table 10-3**, except those for sloping breakwaters, which are separately shown in **Attached Table 10-4**.

Attached Table 10-3 Performance Verification Items and Standard Indexes to Determine the Limit Values of Gravity-type Breakwaters (Except Sloping Breakwaters)

<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">14</td><td rowspan="2">1</td><td rowspan="2">2</td><td rowspan="2">35</td><td rowspan="2">-</td><td>1</td><td rowspan="2">Serviceability</td><td>Permanent</td><td>Self-weight</td><td>Water pressure</td><td>Circular slip failure of ground</td><td>Action–resistance ratio with respect to circular slip failure</td></tr>
<tr><td>2</td><td>Variable</td><td>Variable waves (L1 earthquake ground motion)</td><td>Self-weight, water pressure</td><td>Sliding/overturning of breakwater body, bearing capacity of foundation ground</td><td>Action–resistance ratios with respect to sliding, overturning and bearing capacity</td></tr>
</table>

* [ ] means the alternative dominant action to be studied as design situations.

Attached Table 10-4 Performance Verification Items and Standard Indexes to Determine the Limit Values of Sloping Breakwaters

Ministerial Ordinance			Public Notice			Performance requirements	Design state			Verification item	Standard index to determine the limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non-dominating action		
14	1	2	35	-	1	Serviceability	Permanent	Self-weight	Water pressure	Circular slip failure of ground	Action–resistance ratio with respect to circular slip failure
					2		Variable	Variable waves	Self-weight, water pressure	Sliding and overturning of superstructure	Action–resistance ratios with respect to sliding and overturning
										Bearing capacity of foundation ground	Action–resistance ratio with respect to bearing capacity
								L1 earthquake ground motion	Self-weight, water pressure	Bearing capacity of foundation ground	Action–resistance ratio with respect to bearing capacity

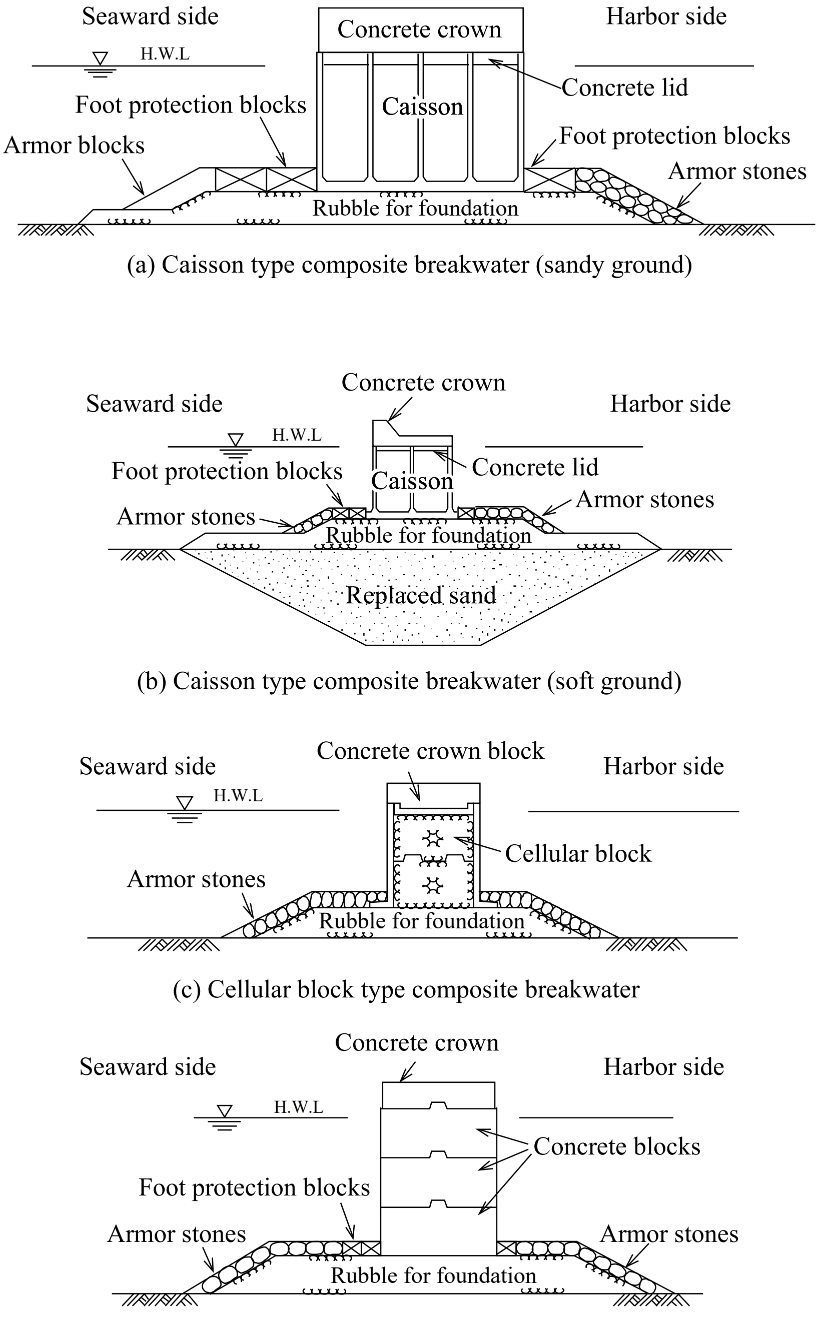
② In addition to the above, gravity-type breakwaters shall be subjected to the following: the requirements and commentaries in Paragraph 3, Article 22 of the Public Notice (Scouring and Outflow) and Article 28 of the Public Notice (Performance Criteria of Armor Stones and Blocks) as needed; the requirements and commentaries in Articles 23 to 27 of the Public Notice depending on the types of members constituting breakwaters.

③ In addition to the above, breakwaters with wave-dissipating structures (breakwaters covered with wave-dissipating blocks, upright wave-absorbing block breakwaters, wave-absorbing caisson breakwaters, etc.) shall be subjected to the requirements in Item 2, Paragraph 1, Article 34 of the Public Notice (Serviceability with Respect to Wave-Dissipating Function).

3.1.1 General

(1) Composite breakwaters with upright breakwater bodies placed on rubble mound foundations are the most typical structure of gravity-type breakwaters in Japan. Therefore, the general descriptions of gravity-type breakwaters in this section are those for composite breakwaters.

(2) **Fig. 3.1.1** shows examples of cross sections of composite breakwaters.



(a) Caisson type composite breakwater (sandy ground)

(b) Caisson type composite breakwater (soft ground)

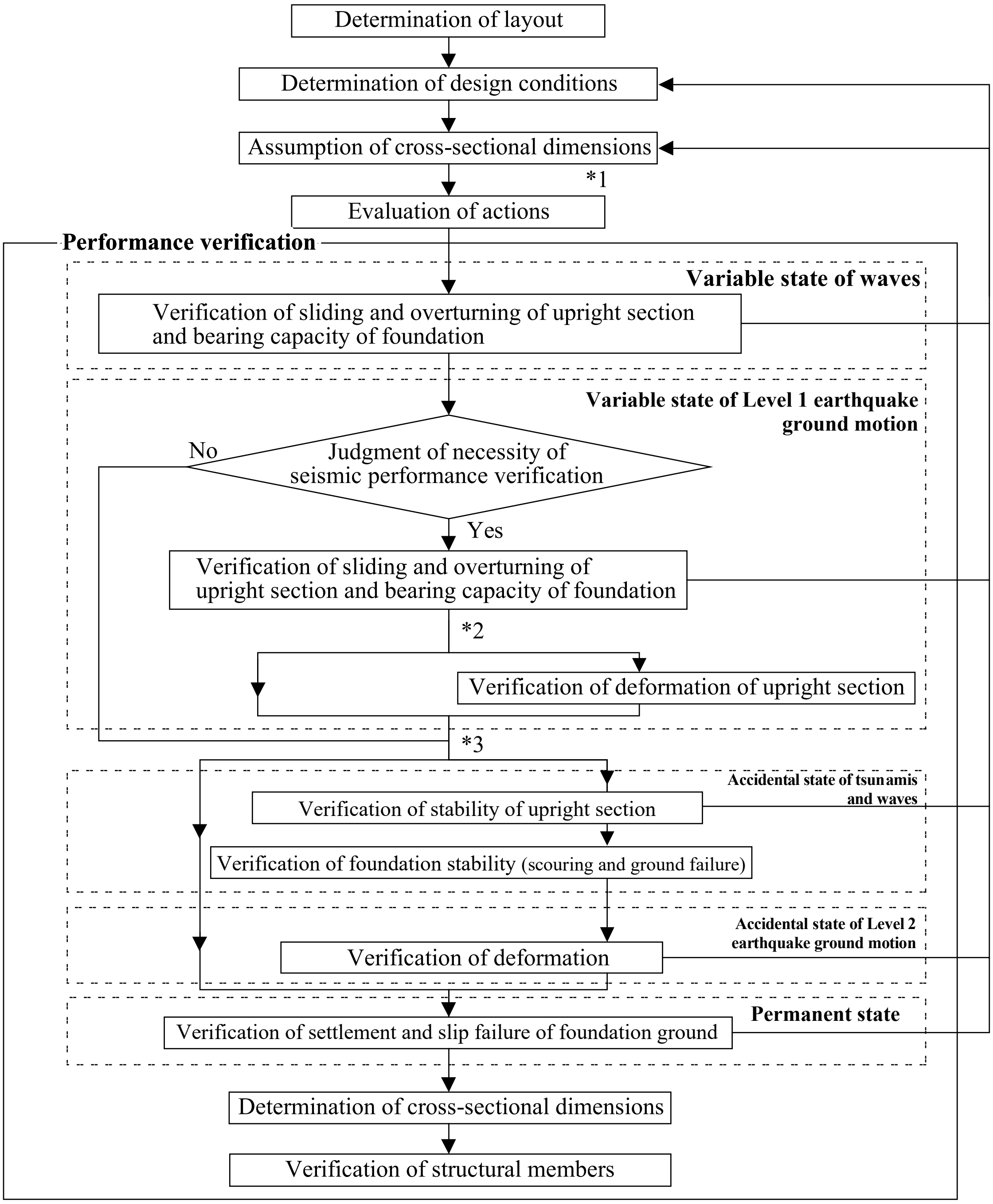
(c) Cellular block type composite breakwater

(d) Concrete block type composite breakwater

Fig. 3.1.1 Examples of Cross Sections of Composite Breakwaters

(3) Given that structures with upright breakwaters are placed on rubble mound foundations, composite breakwaters have characteristics that are closer to sloping breakwaters and upright breakwaters as the ratio of the depths of the crown levels of rubble mounds to wave heights becomes smaller and larger, respectively.

(4) **Fig. 3.1.2** shows an example of the performance verification procedure for composite breakwaters.



*1: The evaluation of the effects of liquefaction and settlement are not shown; therefore, this must be separately considered.

*2: The analysis of deformation due to Level 1 earthquake ground motions may be carried out by dynamic analysis when necessary. For facilities where damage to the objective facilities is assumed to have a serious impact on life, property, and social activity, it is preferable to conduct an examination of deformation by dynamic analysis.

*3: For facilities where damage to the objective facilities is assumed to have a serious impact on life, property, and social activity, it is preferable to conduct a verification for the accidental situations when necessary. For the verification of the accidental situation, reference can be made to **Part III, Chapter 4, 2.2 Items concerning Breakwaters as Facilities Prepared for Accidental Incidents**. Verification for accidental situations associated with waves shall be conducted in cases where facilities handling hazardous cargoes are located directly behind the breakwater and damage to the objective facilities would have a catastrophic impact.

Fig. 3.1.2 Example of the Performance Verification Procedure for Composite Breakwaters

3.1.2 Setting of Basic Cross Sections

(1) The basic cross sections of breakwaters shall be set after clarifying their relations with the lowest astronomical tides, mean monthly-highest (lowest) tide levels, mean tide levels, highest high (lowest low) tide levels, high tide levels during storm surges, Tokyo Bay's mean sea level, etc. The relation between the lowest astronomical tides and datum levels for construction work shall also be clarified if they are different from each other. Furthermore, it is preferable to study the durations of storm surges and the probabilities of their occurrence as needed. For further details on tides, refer to **Part II, Chapter 2, 3 Tide Levels**.

(2) The design tide levels for calculating wave force are generally set in a manner that places breakwaters into the most unstable states, e.g., mean monthly-highest or lowest tides in cases of breakwaters for ports with no necessity of considering the effects of storm surges and tide levels obtained by applying appropriate deviations to mean monthly-highest or lowest tides in cases of breakwaters for ports with the necessity of considering the effects of storm surges.

(3) In cases wherein the foundation ground is soft and settlement can be expected, the crown heights of breakwaters shall be set with preliminarily allowances for settlement or the structures of breakwaters shall facilitate the future leveling of crown heights.

(4) The following factors cause settlement of breakwaters:

① Consolidation settlement of foundation ground
② Washing out of foundation ground
③ Lateral flow of foundation ground
④ Sinking of rubbles and blocks into foundation ground
⑤ Contraction of rubble mounds due to the reduction in porosity

For the settlement allowance for factor ① above, refer to **Part II, Chapter 5, 1 Ground Settlement**. Given that the effects of factors ② to ⑤ above vary depending on the mass of upright sections and the thicknesses of rubble mounds, the allowances for these factors cannot be generalized but can be roughly set on the basis of the actual cases of past construction. For the consolidation settlement of foundation ground after the installation of breakwater bodies, the settlement allowances can be set for either rubble mounds or superstructures, and settlement allowances shall be appropriately set in either case in consideration of construction conditions and the like.

(5) In cases wherein the foundation ground is soft and remarkable settlement or extensive rubble sinking is conceivable, countermeasures shall be taken in a manner that improves soft ground or disperses actions on breakwater bodies via mattresses laid under rubble mounds.

(6) The crown heights of breakwaters in shallow sea areas, particularly in shallow sandy beaches, shall be determined with consideration to the prevention of possible siltation inside ports due to sand carried by overtopping waves.

(7) The crown heights of breakwaters to be used for the protection of swimming beaches, for water intake, and other special purposes shall be determined after fully understanding the purposes of constructing the breakwaters.

(8) In terms of disaster prevention, the thicknesses of the superstructures of upright breakwater bodies are preferably 1 m or more in cases wherein significant wave heights in the front of breakwaters are 2 m or more or at least 50 cm or more even in cases wherein significant wave heights are less than 2 m. For breakwaters with breakwater bodies made of layers of blocks, it is preferable that superstructure concrete has sufficiently large mass as weights to enable the breakwaters to resist against sliding failures. **Fig. 3.1.3** shows the relationship between the thicknesses of superstructures and design wave heights by using practical examples.

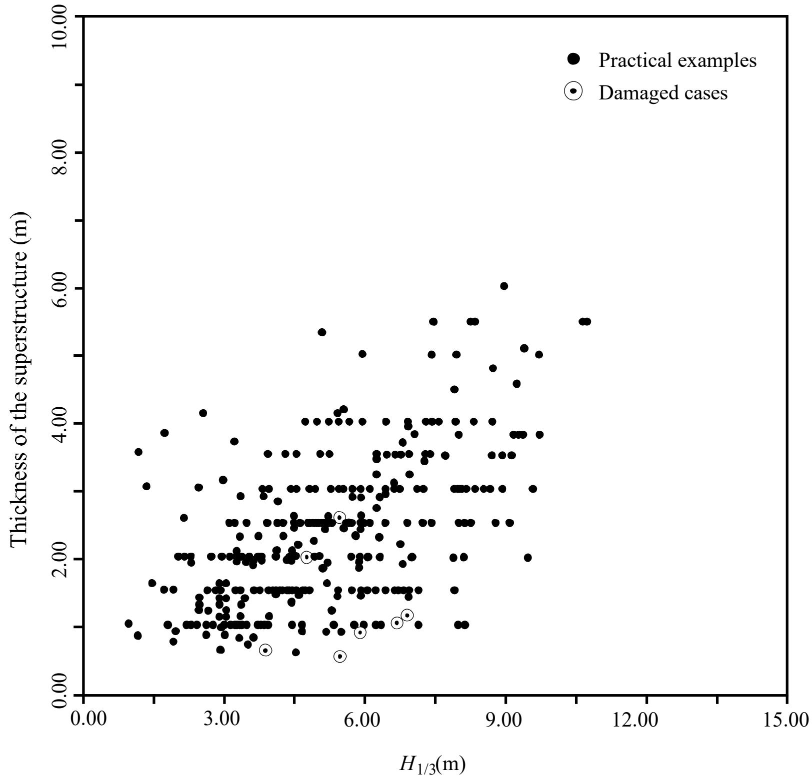


Fig. 3.1.3 Relationship between the Thicknesses of Superstructures and Design Wave Heights (Practical Examples)

(9) Considering that caisson designs with low top surface levels impose constraints on the work to install caissons, fill sand, and cast lid and superstructure concrete, the top surface levels are generally set higher than the mean monthly-high tide levels. In the case of block-type breakwaters, it is preferable that the top surface levels of the uppermost blocks or cellular blocks are set at least higher than the mean tide level or higher than the mean monthly-highest tide level to facilitate the construction of superstructures.

(10) The crown levels of rubble mounds should be as deep as possible to protect the rubble mounds from impulsive breaking waves, except in the case of using caissons as upright sections, for which the crown levels shall be set to make caissons installable. Furthermore, the berm widths at the seaward side of the rubble mounds (excluding footing sections) shall be sufficiently wide depending on the wave height to reduce the unfavorable effects of the action of impulsive breaking wave force as much as possible with reference to **Part II, Chapter 2, 6.2.4 Impulsive Breaking Wave Force**.

(11) The required mass of rubble below armor units and blocks shall be appropriately set in accordance with site conditions to prevent materials from being washed out. It is preferable that the weights of rubble below armor units and blocks are approximately 1/20 or more of the mass of armor units, and the mass of the materials to be laid under the rubble below armor units and blocks are approximately 1/20 or more of that of the rubble and blocks.[1)]

(12) The berm widths of the rubble mounds shall be set to secure the specified stability against the slip failure of ground and eccentric and inclined loads (refer to **Part III, Chapter 2, 3.2.5 Bearing Force against Eccentric and Inclined Actions**). Furthermore, it is preferable that the berm widths at the seaward side are set at not less than 5 m, excluding the footing sections, to reduce the effects of the action of impulsive breaking wave force to the greatest extent possible. However, this shall not apply in the case of hybrid caissons and other special structural types. The berm widths of the rubble mounds at the harbor side can be approximately 2/3 of those at the seaward side.

(13) There may be a case of high rubble backing for reinforcing the sliding resistance of upright sections. However, caution is necessary in such a case because rubble is easily scattered by overtopping waves. It is preferable that the rubble is provided with the armor of cubic blocks or deformed blocks as needed. The performance verification of cross sections shall be appropriately performed by referring to the following provisions in **Part III, Chapter 4, 3.1.6 Performance Verification and Cautions When the Harbor Side of Upright Sections is Reinforced**.

(14) Rubble mound foundations should have thicknesses of 1.5 m or more to enable them to produce the effects of broadly distributing the loads transferred through the weights of upright sections, providing the upright sections with flat installation ground, and preventing waves from causing scouring.

(15) Although the slope gradients of rubble mound foundations shall be determined on the basis of stability calculations, they can be generally set at 1:2 to 1:3 and 1:1.5 to 1:2 for the seaward side and harbor side of breakwaters, respectively.

3.1.3 Actions

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

In the stability verification of composite breakwaters, the following actions shall be considered in respective design situations provided that the performance verification of accidental action situation can be omitted in cases wherein design object breakwaters are not categorized as facilities prepared for accidental incidents.

① **Permanent action situation**

The dominant action to be considered shall be the self-weight of breakwater bodies. For the setting of self-weight, refer to **Part II, Chapter 10, 2 Self-Weight**.

② **Variable action situation**

(a) Variable waves and Level 1 earthquake ground motions shall be the dominant actions to be considered. For the setting of variable waves and Level 1 earthquake ground motions, refer to **Part II, Chapter 2, 4.1 Setting of Wave Conditions** and **Part II, Chapter 6, 1.2 Level 1 Earthquake Ground Motions Used in Performance Verification of Facilities**, respectively.

(b) The necessity of the performance verification of earthquake resistance in terms of sliding and overturning due to Level 1 earthquake ground motions can be determined on the basis of the relationship between the cross-sectional dimensions of breakwater bodies and Level 1 earthquake ground motions under a variable action situation with respect to variable waves.[2), 3)]

(c) The necessity of the performance verification of earthquake resistance can be determined using the relationship between the B_w/h ratios of the widths of breakwater bodies B_w, excluding footings to their installation depths h and the maximum engineering acceleration at bed rock (**Fig. 3.1.4**). The performance verification of earthquake resistance can be omitted when the concerned breakwaters are plotted below the curve in the figure. The figure is established on the basis of 30 cm as an allowable residual deformation of the upright sections of breakwaters subjected to Level 1 earthquake ground motions. Therefore, when adopting other values for the allowable residual deformation, the specific verification of their appropriateness should be implemented.

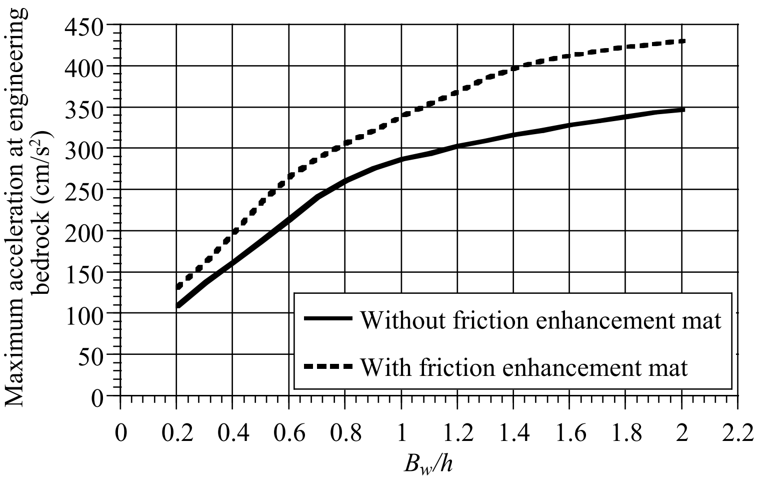


Fig. 3.1.4 Figure for Determining the Necessity of the Performance Verification of Earthquake Resistance

③ **Accidental action situation**

(a) For the setting of actions under the accidental action situation in which the dominant action is Level 2 earthquake ground motions, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Prepared for Accidental Incidents**.

(b) For the setting of actions under the accidental action situation in which the dominant action is design tsunamis, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Prepared for Accidental Incidents**.

(c) For the setting of actions under the accidental action situation in which the dominant action is accidental waves, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Prepared for Accidental Incidents**.

(2) Points of Caution When Setting Actions

① In the performance verification, there are cases in which the most dangerous tide levels differ depending on the verification items and objects of verification.

② The wave parameters necessary in the performance verifications are wave heights, wave directions, wavelengths, periods, etc. In determining these parameters, refer to **Part II, Chapter 2, 4 Waves** and **3.6 Design Tide Level Conditions**. For the data on wind used in wave hindcasting, refer to **Part II, Chapter 2, 2.3 Wind Pressure**.

③ The duration of waves is also considered an element that influences the stability of breakwaters. However, at present, such influences have not been fully understood. Therefore, it shall be noted that there are cases wherein repeated waves over an extended period of time are considered to have caused damage to breakwaters, particularly breakwater mounds, facing the open sea. Furthermore, because there are cases of damage to facilities during construction, it is necessary to decide the parameters for waves during construction in consideration of the construction plans and processes.

④ Rubble mounds with high crown heights and moderately wide berm widths may induce impulsive breaking wave force. Therefore, due consideration shall be given to the possible occurrence of impulsive breaking wave force by referring to **Part II, Chapter 2, 6.2 Wave Force on Upright Walls**. It shall also be noted that there may be a case that the intensity of wave pressure on breakwaters is increased as their crown heights increase.

⑤ In the performance verification, it shall be noted that the most dangerous waves to the stability of upright sections may be different from those in the calculation of the required mass of armor units.

⑥ In cases of the differences in still tide levels inside and outside breakwaters, it is preferable to consider the hydrostatic pressure equivalent to the differences in tide levels.

⑦ It is necessary to consider the buoyancy of the breakwater bodies below the still tide levels. In cases wherein differences exist in still tide levels inside and outside breakwaters, buoyancy can be considered for the portions of breakwater bodies below the lines connecting the still tide levels inside and outside breakwaters.

⑧ The influences of wind pressure, earth pressure, impulsive force of ships and floating objects, and currents shall be considered as needed.

⑨ In cases wherein erosion, sedimentation, and changes in the gradients of sea bottoms can be expected after the construction of breakwaters, the influences of those phenomena shall also be considered.

⑩ For dynamic water pressure during earthquakes, refer to **Part II, Chapter 4, 3.2 Dynamic Water Pressure**.

3.1.4 Performance Verification of the Overall Stability of Breakwater Bodies

(1) Performance Verification Items for the Overall Stability of Breakwater Bodies

For the performance verification items when conducting the performance verification of the overall stability of breakwater bodies under respective design situations on the basis of the static equation of equilibrium, refer to **Part III, Chapter 4, 3.1 [interpretation], Attached Table 10-3**. The performance verification of the variable action situation in which the dominant action is Level 1 earthquake ground motions can be performed on the basis of **Fig. 3.1.4 Figure for Determining the Necessity of the Performance Verification of Earthquake Resistance**. The performance verification of an accidental action situation can be omitted in cases wherein design object breakwaters are not categorized as facilities prepared for accidental incidents.

(2) Performance Verification of the Overall Stability of Breakwater Bodies under Permanent Action Situation

The performance verification of the overall stability of breakwater bodies under a permanent action situation in which the dominant action is self-weight shall be performed for the circular slip failures of foundation ground in general.

① The verification of the circular slip failures of foundation ground under a permanent action situation with respect to the self-weights of breakwater bodies can be performed using **equation (3.1.1)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.1**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

② The partial factors shown in **Table 3.1.1** were set with reference to the safety levels in past standards.[4] Furthermore, the coefficients of the variation CV of cohesive soil in the table can be determined using the coefficients of variation CV corresponding to the correction factor b_1 from the process of calculating the characteristic values of adhesion in **Part II, Chapter 3, 2.1 Estimation of the Physical Property of Ground**. In such a case, among the soil layers (excluding thin ones) where circles can pass through, the soil layer that has the largest coefficient of variation CV can be the representative soil layer.

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

$$R_k = \sum \left[\left\{ c'_k\, s + (w'_k + q_k)\cos^2\theta \tan\phi'_k \right\} \sec\theta \right] \tag{3.1.1}$$

$$S_k = \sum \left[(w_k + q_k)\sin\theta \right]$$

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 water surface or underwater weight when below water surface);

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

ϕ' : apparent shear resistance angle on the basis of effective stress (°);

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

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.1.1 Partial Factors Used for the Performance Verification of the Circular Slip Failure of Foundation Ground

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

Verification object	Coefficient of variation of cohesive soil in the representative soil layer CV	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
	Not less than 0.25	— (1.00)	— (1.00)	1.30

③ In the performance verification of the sliding of ground, the most dangerous tide levels should be used for the stability of breakwaters. For the setting of tide levels, refer to **Part II, Chapter 2, 3 Tide Levels**.

④ When improving foundation ground, the verification of circular slip failure can be performed by referring to **Part II, Chapter 2, 5 Ground Improvement Method**.

(3) Performance Verification of the Overall Stability of Breakwater Bodies under a Variable Action Situation (Variable Waves)

① **General**

The performance verification of the overall stability of breakwater bodies under a variable action situation in which the dominant action is variable waves shall be performed for the sliding and overturning of breakwater bodies and the bearing capacity of foundation ground.

② **Examination of the sliding of breakwater bodies**

(a) The verification of the sliding of breakwater bodies with respect to variable waves can be performed using **equation (3.1.2)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.2**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$R_k = \{f_k (W_k - P_{B_k} - P_{U_K})\} \qquad \textbf{(3.1.2)}$$
$$S_k = P_{Hk}$$

where

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

W : weight of a breakwater body (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift (kN/m);

P_H : horizontal wave force (kN/m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.1.2 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of a breakwater body (Variable state of waves)	0.83	1.08	– (1.00)

(b) The partial factors shown in **Table 3.1.2** were set with reference to the safety levels in past standards.[5] Furthermore, the partial factors above are set under the conditions that the topographies of seabed where breakwaters are installed have gradients of less than 1/30. In cases wherein the topographies of seabed have gradients larger than 1/30, partial factors shall be appropriately set with reference to the descriptions in **Reference 5)**.

(c) For the verification of the sliding failures of composite breakwaters with the harbor side of upright sections reinforced, refer to **Part III, Chapter 4, 3.1.6 Performance Verification and Cautions When the Harbor Side of Upright Sections is Reinforced**.

(d) The tide levels to be used for the verification of the sliding and overturning of wall bodies, as well as bearing capacity, are generally either mean monthly-lowest water levels (L.W.L) or mean monthly-highest water levels (H.W.L).

(e) In cases wherein caissons with footings have rectangular cross sections at both seaward and landward sides, buoyancy P_B can be calculated using the following equation. In this equation, subscript k indicates the characteristic value. For footings with other shapes and hunched sections, buoyancy shall be appropriately set.

$$P_{B_k} = \rho_w g \{(wl_k + h)B_c + 2h_f B_f\} \tag{3.1.3}$$

where

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

wl : tide level (m);

h : installation depth (m);

B_C : width of breakwater body (m);

h_f : height of footing (m);

B_f : width of footing (m).

(f) For the calculation of wave force, refer to **Part II, Chapter 2, 6.2 Wave Force Acting on Vertical Walls**.

(g) For the unit weights and friction coefficients to be used in the performance verification, refer to **Part II, Chapter 10 Self-weight and Surcharges** and **Part II, Chapter 11, 9 Friction Coefficients**. There are cases wherein friction enhancement mats are laid under the bottom faces of upright sections to increase the friction coefficients between the upright sections and foundation mounds. For the friction enhancement mats, refer to **Part II, Chapter 11, 9 Friction Coefficients**.

③ **Examination of the overturning of breakwater bodies**

(a) The verification of the overturning of breakwater bodies due to variable waves can be performed using **equation (3.1.4)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.3**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

$$\begin{aligned} & m \cdot \frac{S_d}{R_d} \le 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k \\ & R_k = (a_1 W_k - a_2 P_{B_k} - a_3 P_{U_K}) \\ & S_k = a_4 P_{H_k} \end{aligned} \tag{3.1.4}$$

where

W : weight of a breakwater body (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift force (kN/m);

P_H : horizontal wave force (kN/m);

a_1–a_4 : arm lengths of actions (m) (refer to **Fig. 3.1.5**);

R_k : resistance term (kN·m/m);

S_k : load term (kN·m/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

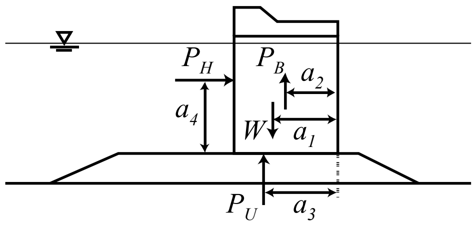


Fig. 3.1.5 Arm Lengths When Calculating Moment

Table 3.1.3 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	0.95	1.14	— (1.00)

(b) The partial factors shown in **Table 3.1.3** were set with reference to the safety levels in past standards.[5] Furthermore, the partial factors above are set under the conditions that the topographies of seabed where breakwaters are installed have gradients of less than 1/30. In cases wherein topographies of seabed have gradients larger than 1/30, partial factors shall be appropriately set with reference to the descriptions in **Reference 5)**.

(c) In cases wherein caissons with footings have rectangular cross sections at both seaward and landward sides, buoyancy can be calculated using **equation (3.1.3)**. For footings with other shapes and hunched sections, buoyancy shall be appropriately set.

④ **Examination of the bearing capacity of foundation ground**

(a) The verification of the stability of the bearing capacity of foundation ground at the bottom faces of the upright section against variable waves can be conducted in accordance with the simplified Bishop method (refer to **Part III, Chapter 2, 4 Slope Stability**), which is one of the circular slip calculation methods based on the slicing method. The simplified Bishop method was adopted in the verification because it has been proven by experiments in centrifugal fields to be a model that can best explain the stability of bearing capacity compared with the modified Fellenius' method and the friction circle method.[6]

(b) The performance verification of the bearing capacity of foundation ground can be performed using **equation (3.1.5)**, which was obtained using the simplified Bishop method. In the equation, the partial factors in the equation can be selected from the values in **Table 3.1.4**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification of convenience. Furthermore, in the equation, subscripts k and d indicate the characteristic value and design value, respectively. When using **equation (3.1.5)**, first an auxiliary parameter F_f needs to be determined via repeated calculation so that F_f satisfies $R_k = F_f \times S_k$ (with attention to the fact that R_k is a function of F_f), and the stability verification of bearing capacity can be performed using R_k and S_k obtained as a result of the repeated calculation.

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

$$F_f = \frac{R_k(F_f)}{S_k}$$

$$R_k = \sum \left[\frac{\{c'_k s + (w'_k + q_k)\tan\phi'_k\}\sec\theta}{1 + \tan\theta\tan\phi'_k / F_f} \right] \qquad \textbf{(3.1.5)}$$

$$S_k = \sum \{(w'_k + q_k)\sin\theta\} + \frac{dP_{H_k}}{r}$$

where

P_H : horizontal wave force (kN/m);

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

s : width of a segment (m);

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

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

ϕ' : apparent shear resistance angle on the basis of effective stress (°);

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

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

d : arm length of horizontal wave force P_H (a length of a vertical line from the center of a circle to an acting force vector);

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

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : an adjustment factor.

Table 3.1.4 Partial Factors Used for the Performance Verification of the Bearing Capacity of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Bearing capacity of foundation ground (Variable state of waves)	— (1.00)	— (1.00)	1.00

(4) Performance Verification of the Overall Stability of Breakwater Bodies under a Variable Action Situation (Level 1 Earthquake Ground Motions)

① **Examination of the sliding failures of breakwater bodies**

(a) Although the verification of the stability of breakwater bodies against Level 1 earthquake ground motions is often omitted, in cases wherein breakwaters have deep installation depths and small design wave heights, there may be the cases wherein Level 1 earthquake ground motions can be a dominant action. In such cases, the performance verification shall be performed for the earthquake resistance.

(b) For the determination of the necessity of the performance verification of earthquake resistance and the method for calculating the seismic coefficients when performing the performance verification of earthquake resistance, refer to **Fig. 3.1.4** and **Reference (Part III), Chapter 1, 1 Details of Seismic Coefficients for Verification**.

(c) The verification of the sliding failures of breakwater bodies due to Level 1 earthquake ground motions on the basis of the equation of equilibrium can be performed using **Equation (3.1.6)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.5**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification of convenience. The seismic coefficients for verification in **equation (3.1.6)** can be calculated by the method described in **Reference (Part III), Chapter 1, 1 Details of Seismic Coefficients for Verification**.

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$R_k = \mu W'_k \qquad \textbf{(3.1.6)}$$
$$S_k = k_{h_k} W_k + 2P_{dw_k}$$

where

k_h : seismic coefficient for verification;

W : weight of a breakwater body (kN/m);

P_{d_w} : resultant force of dynamic water pressure (kN/m), which can be calculated by **equation (3.1.7)**.

$$P_{d_w} = \frac{7}{12} k_h \rho_w g H^2 \qquad \textbf{(3.1.7)}$$

where

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

H : installation depth of a breakwater body (m);

W' : effective weight of a breakwater body in water (= $W - P_B$) (kN/m);

P_B : buoyancy (kN/m);

μ : friction coefficient between a breakwater body and a rubble mound;

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.1.5 Partial Factors Used for the Performance Verification of the Sliding Failure of Breakwater Bodies Due to Level 1 Earthquake Ground Motions

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding failure of breakwater body (Variable state of Level 1 earthquake ground motions)	— (1.00)	— (1.00)	1.20

② **Examination of overturning of breakwater bodies**

(a) The verification of the overturning of breakwater bodies due to Level 1 earthquake ground motions on the basis of the equation of equilibrium can be performed using **equation (3.1.8)**. In the equation, subscripts k and d indicate the characteristic value and design value, respectively, and the buoyancy acting on breakwater bodies can be calculated by **equation (3.1.3)**. The partial factors in the equation can be selected from the values in **Table 3.1.6**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

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

$$R_k = a_3 W'_k \tag{3.1.8}$$

$$S_k = a_1 k_{hk} W_k + 2a_2 P_{dwk} W_k$$

where

k_h : seismic coefficient for verification;

W : weight of a breakwater body (kN/m);

P_{d_w} : resultant force of dynamic water pressure (kN/m), which can be calculated by **equation (3.1.7)**.

Table 3.1.6 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies Due to Level 1 Earthquake Ground Motions

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of Level 1 earthquake ground motions)	— (1.00)	— (1.00)	1.10

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

(a) The verification of the bearing capacity of foundation ground against Level 1 earthquake ground motions can be performed with due consideration to the actions of earthquake ground motions and with reference to **Part III, Chapter 2, 3.2 Shallow Foundations**. However, for breakwaters that are expected to have major problems with the bearing capacity of foundation ground and the stability against settlement, it is preferable to perform detailed examinations, including dynamic analysis.

(5) Performance Verification of Breakwater Bodies under Accidental Situation

① For the performance verification of breakwater bodies under an accidental situation in which the dominant action is Level 2 earthquake ground motions, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters**

Categorized as the Facilities Prepared for Accidental Incidents and **Part III, Chapter 4, 7.4.1 Setting and Impact Assessment of Earthquake Ground Motions Preceding Tsunamis**.

② For the performance verification of breakwater bodies under an accidental situation in which the dominant action is design tsunamis, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Categorized as the Facilities Prepared for Accidental Incidents** and **Part III, Chapter 4, 7 Tsunami Protection Breakwaters**.

③ For the performance verification of breakwater bodies under an accidental situation in which the dominant action is accidental waves, refer to **Part III, Chapter 4, 2.2 Items Related to Breakwaters Categorized as the Facilities Prepared for Accidental Incidents** and **Part III, Chapter 4, 6 Storm Surge Prevention Breakwaters**.

3.1.5 Performance Verification and Points of Cautions for Other Items about the Overall Stability of Breakwater Bodies

(1) Performance Verification of the Stability of Sloped Sections

① For the performance verification of the stability of rubble sections, refer to **Part III, Chapter 2, 3.2.5 Bearing Capacity against Eccentric Inclined Actions**.

② The armor units of rubble sections shall have mass to achieve sufficient stability against wave force and thicknesses to prevent the inner materials from being washed out.

③ For the calculation of the required mass of armor units, refer to **Part II, Chapter 2, 6.6.2 Required Mass of Armor Stones and Blocks of Composite Breakwater Mounds against Waves.**

④ For the performance verification of the sloped sections covered by sand mastic, refer to past cases and existing study results.[7)]

(2) Points of Caution When Performing the Performance Verification of Head and Corner Sections

① Unlike the trunk sections, the head sections of composite breakwaters have factors that have not been fully elucidated, such as the washing out of and actions on foundations. Therefore, it is preferable that the mass of armor stones and the blocks of head sections are larger than that of the trunk sections. For the calculation of the mass of armor materials, refer to **Part II, Chapter 2, 6.6.2 Required Mass of Armor Stones and Blocks of Composite Breakwater Mounds against Waves**.

② In cases of soft ground, the performance verification shall be performed for the sliding failures of breakwaters in their face line directions. In such cases, the performance verification can take into consideration the friction resistance at foundation sides.

③ For the sliding failures of breakwaters in their face line directions, refer to **Part III, Chapter 2, 4 Slope Stability**.

④ The performance verification of corner sections shall take into consideration the increases in wave heights.

⑤ When there are corner sections on the face lines of composite breakwaters, the corner sections not only allow waves to converge on them but also cause the increase in wave heights around them owing to the superposition of reflection waves from respective sections along the face lines. There have been cases of damage to breakwaters due to such wave actions at corner sections. Therefore, the determination of face lines and the stability calculations of composite breakwaters shall be made with reference to **Part II, Chapter 2, 4.4 Wave Deformation** and **Part II, Chapter 2, 6.2.8 Calculation of Wave Force in Consideration of the Effects of the Shapes of Face Lines**.

⑥ Head sections provided with beacons shall ensure their structural safety with the beacons, including the ancillary facilities necessary to maintain the functions of the beacons. For the wind pressure acting on beacons, refer to **Part II, Chapter 2, 2 Wind**.

⑦ It shall be noted that there have been cases of damage to the base sections of breakwaters extended from beaches because the structures of the base sections were simplified.

(3) Examination of Settlement

The performance verification of the settlement due to consolidation or other reasons shall be performed with due consideration to the characteristics of the ground and structures. For the settlement, refer to **Part III, Chapter 2, 3.5 Settlement of Foundations**.

(4) Other Performance Verification

① The stability verification of breakwater bodies against variable waves can be performed with reference to design methods[8) to 13)], which take into consideration the expected sliding distances.

② The performance verification of the deformation and stresses of breakwater bodies or ground with respect to earthquake ground motions can be performed using the seismic response analysis based on the finite element method. For the points of caution when using the seismic response analysis, refer to the contents in **Reference (Part III), Chapter 1, 2 Basic Items for Seismic Response Analysis**.

3.1.6 Performance Verification and Cautions When the Harbor Side of Upright Sections is Reinforced

(1) General

① One of the typical methods for reinforcing the harbor side of upright sections is embankment widening work, which installs rubbles and blocks at the back of upright sections. Properly arranged rubbles and blocks enable upright sections to increase resistance against the sliding and bearing capacity of foundations. It shall be noted that embankment widening work needs to be implemented to avoid the disturbance of navigation, sheltering, and mooring of ships inside harbors. The items described in this section are based on the research outcomes of Takahashi et al.[14)] and Sato et al.[15)] In addition to the embankment widening work using rubble and blocks, there are other alternative methods. For the details of these alternative methods, refer to **Reference 16)**.

② In the verification of the stability of breakwater bodies against sliding failure due to wave force on upright sections without considering the embankment widening work at the back of breakwater bodies, the action–resistance ratios calculated with a partial factor of 1.0 shall be less than 1.0 in essence because large action–resistance ratios have risks of causing upright sections to slide until the embankment widening work deforms to a level that produces sufficient resistance to stabilize the upright sections or to slide or be overturned seaward owing to backwash.

③ Embankment widening work shall have enough protection from damage due to overtopping and longshore waves, as well as overflowing tsunamis. Furthermore, full-scale embankment widening work causes an increase in the uplift force to be applied to shielding work. Therefore, it is necessary to examine the stability of breakwater bodies in consideration of the uplift force or to provide adequate openings on shielding work so that water pressure can be released.

④ In cases wherein there are water level differences between the seaward and landward sides of breakwater bodies due to tsunamis, the embankment widening work made of foundation mounds and rubble receives seepage force and that made of blocks and shielding work receive uplift force. The seepage force that acts on the embankment widening work made of foundation mounds and rubble reduces the maximum resistance force of the embankment widening work in the performance verification for slide failures and reduces the stability of the entire ground in the performance verification for bearing capacity. Likewise, the uplift force that acts on the embankment widening work made of blocks reduces the maximum resistance force of the embankment widening work in the performance verification for slide failures and reduces the stability of the entire ground in the performance verification for bearing capacity. Furthermore, the stability of shielding work shall be examined with due consideration to the effects of uplift force by preferably referring to **Part III, Chapter 4, 7 Tsunami Protection Breakwaters**.

⑤ When rubble is used for embankment widening work, height a and width b of embankment widening work are basically not less than 1/3 of the heights of upright sections (including superstructures) (refer to **Fig. 3.1.6**). In cases wherein the embankment widening work is smaller than the above, it is necessary to conduct performance verification by using centrifugal model tests or finite element analyses, which can appropriately assess the behavior of ground in addition to the verification described below. In actual construction, it is also necessary to avoid the use of rubble stones that are rounded and have small diameters because the foundation ground made of rounded rubble stones has small shear strength and rubble stones with small diameters are likely to cause piping.

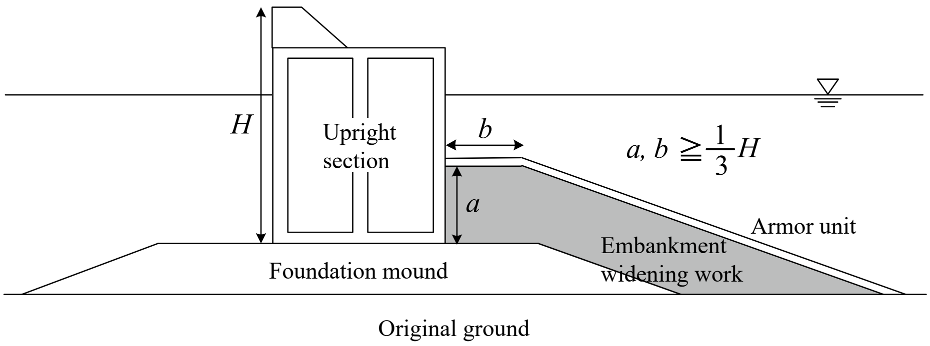


Fig. 3.1.6 Breakwater Reinforced with the Harbor Side Reinforced with Rubble Stones

⑥ When reinforcing breakwaters with embankment widening work made of blocks such as concrete blocks, it is necessary to perform embankment widening work without leaving gaps between the upright sections and the blocks. These blocks shall have sufficiently long durability. Furthermore, a study[14)] reported that blocks installed at the bottom levels that are different from those of upright sections can increase the resistance against sliding. Therefore, there is a possibility of providing breakwater bodies with larger resistance against sliding by devising the methods for installing blocks.

⑦ Basically, the performance verification of bottom slabs shall not consider the reaction force from embankment widening work because the characteristics of the reaction force from embankment widening work have not been clarified. However, such reaction force may be incorporated into performance verification in cases wherein the characteristics of the reaction force can be appropriately examined through model tests.

(2) Verification of Sliding Failures

① The verification of sliding failures can be performed using **equation (3.1.9)**. In this equation, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Table 3.1.7**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

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

$$R_k = \left\{ f_k (W_k - P_{Bk} - P_{Uk} - P_{Vk}) + P_{H2\max k} \right\} \qquad \textbf{(3.1.9)}$$

$$S_k = P_{Hk}$$

where

f : friction coefficient between the bottom face of an upright section and a foundation mound;

W : atmospheric weight of an upright section (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift force (kN/m);

P_H : horizontal wave force (kN/m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

$P_{H2\max}$: maximum resistance from reinforcing rubble or block (kN/m);

P_V : friction force between the bottom face of an upright section and rubble stones (kN/m);

m : adjustment factor.

Table 3.1.7 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies When the Harbor Side of Upright Sections Is Reinforced

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

② The maximum resistance $P_{H2\max}$ when reinforcing breakwaters with embankment widening work made of rubble stones can be obtained by **equation (3.1.10)**. This equation is the simplified Bishop method (expressed in the form of effective stress) with a partial factor of 1.0 to obtain $P_{H2\max}$ in a manner that assumes a shallow circular slip surface started from a rear toe of an upright section (**Fig. 3.1.7**). It is necessary to change the positions of the circular slip surface to identify the position that achieves the least $P_{H2\max}$. Here parameters such as shear strength (c' and ϕ') shall be set in accordance with the values of foundation mounds.

$$\sum\left[\frac{\left\{c'_k s+\left(w'_k+q_k\right)\tan\phi'_k\right\}\sec\theta}{1+\tan\theta\tan\phi'_k}\right]=\sum\left\{\left(w'_k+q_k\right)\sin\theta\right\}+\frac{a_2 P_{H2\max k}}{r} \tag{3.1.10}$$

where

c' : undrained shear strength for cohesive soil ground or apparent adhesion of the ground made of stone materials (kN/m^2);

ϕ' : shear resistance angle under a drained condition for sandy soil or the ground made of stone materials (°);

s : width of a segment (m);

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

q : vertical load acting on a segment (including q_V and P_V) (kN/m) (where P_V = tan 15°·$P_{H2\max}$);

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

$P_{H2\max}$: maximum resistance from reinforcing rubble stones (kN/m) (with the working height set at 1/3 of the height a of embankment widening work);

a_2 : arm length of $P_{H2\max}$ (a length of a vertical line from the center of a circle to an acting force vector) (m);

r : radius of a slip circle (m).

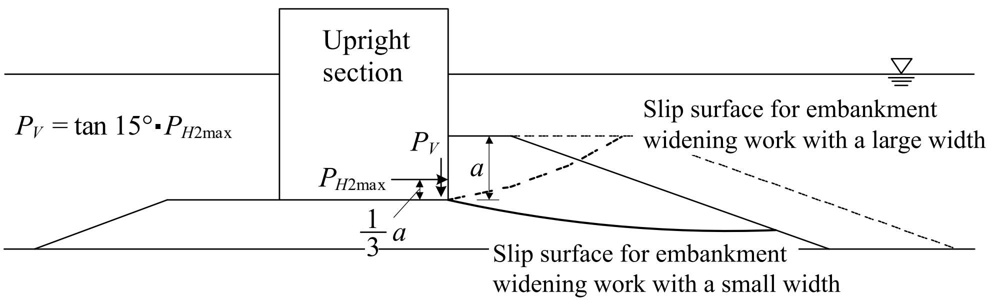


Fig. 3.1.7 Concept of a Shallow Slip Surface When the Harbor Side of Upright Sections Is Reinforced with Rubble Stones

③ When reinforcing breakwaters with embankment widening work made of blocks, the maximum resistance $P_{H2\,max}$ can be obtained by **equation (3.1.11),** in which the friction force P_v is omitted. It is necessary to use a friction coefficient f that is obtained via friction tests.

$$P_{H2\max k} = f_k\, W_{bk} \tag{3.1.11}$$

where

f : friction coefficient between a block and a foundation mound;

W_b : effective weight of a block (kN/m) (atmospheric weight when above water surface or underwater weight when below water surface).

(3) Performance Verification of Overturning

Owing to a low working point, the resistance force from embankment widening work with a height that is 1/3 the height of the upright section does not contribute significantly to the stability of the upright section against overturning. Furthermore, the stone materials used for foundation mounds and embankment widening work exert large shear resistance force after they are subjected to a certain level of shear strain. Therefore, there is a risk that upright sections may have already lost their stability against overturning by the time the resistance force from embankment widening work is exerted. With all these factors, the performance verification of overturning shall be performed without considering the effects of embankment widening work by using **equation (3.1.8**), with partial factors selected from **Table 3.1.8**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification of convenience. However, the resistance force from embankment widening work can be considered only in cases wherein the effectiveness of the resistance force is appropriately verified by model tests.

Table 3.1.8 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies When the Harbor Side of Upright Sections is Reinforced

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

(4) Performance Verification of Bearing Capacity

① The performance verification of bearing capacity when the harbor side of upright sections is reinforced with rubble stones can be performed using **equation (3.1.12)**. In the equation, subscripts k and d indicate the characteristic value and design value, respectively. **Equation (3.1.12)** is the simplified Bishop method, which is expressed in the form of effective stress and assumes deep circular slip surfaces starting from the lower points of upright sections (**Figs. 3.1.8** and **3.1.9**).

When using **equation (3.1.12)**, an auxiliary parameter F_f first needs to be determined via repeated calculations so that F_f satisfies $R_k = F_f \times S_k$ (with attention to the fact that R_k is a function of F_f). Thereafter, the performance verification of bearing capacity can be performed using the R_k and S_k obtained as a result of the repeated calculation. The partial factors and adjustment factors in the equation can be selected from the values in **Table 3.1.9**, in which the symbol "—" in a column means that the value in parentheses in the column can be used for the performance verification for convenience. In the case of embankment widening work made of rubble stones, parameters such shear strength (c' and ϕ') of embankment widening work shall be set in accordance with the values of foundation mounds.

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

$$F_f = \frac{R_k(F_f)}{S_k}$$

$$R_k = \sum \left[\frac{\{c'_k s + (w'_k + q_k)\tan\phi'_k\}\sec\theta}{1 + \tan\theta\tan\phi'_k / F_f} \right] \tag{3.1.12}$$

$$S_k = \sum \{(w_k' + q_k)\sin\theta\} + \frac{a_1 P_{H1_k} + a_2 P_{H2_k}}{r}$$

where

c' : undrained shear strength for cohesive soil ground or apparent adhesion of the ground made of stone materials (kN/m^2);

ϕ' : shear resistance angle under a drained condition for sandy soil or the ground made of stone materials (°);

s : width of a segment (m);

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

q : vertical load acting on a segment (including q_v, P_V and q_b) (kN/m);

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

P_{H1} : friction resistance force on the bottom face of an upright section (kN/m);

P_{H2} : resistance force from reinforcing rubble stones (kN/m) (with the working height set at 1/3 of the height a of embankment widening work. This resistance force cannot be considered in the case of embankment widening work made of blocks);

a_1 : arm length of P_{H1} (a length of a vertical line from the center of a circle to an acting force vector) (m);

a_2 : arm length of P_{H2} (a length of a vertical line from the center of a circle to an acting force vector) (m);

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

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

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.1.9 Partial Factors Used for the Performance Verification of Bearing Capacity When the Harbor Side of Upright Sections is Reinforced

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Bearing capacity of foundation ground (Variable state of waves)	— (1.00)	— (1.00)	1.00

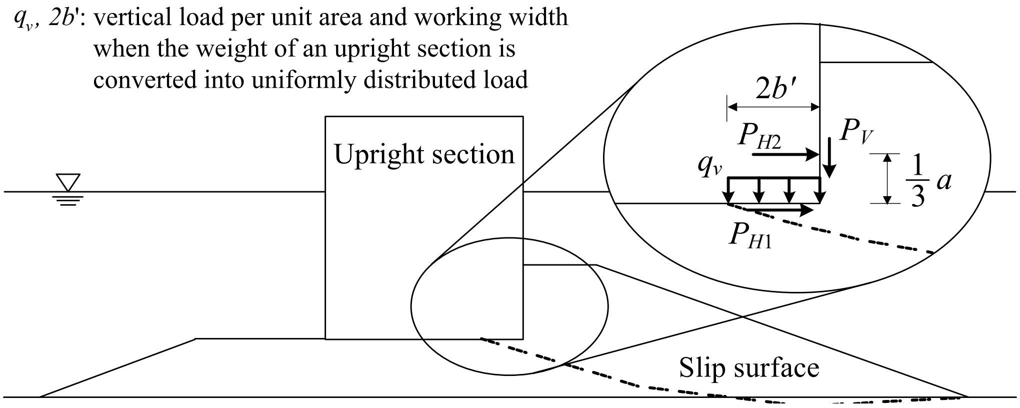


Fig. 3.1.8 Concept of a Deep Slip Surface When the Harbor Side of Upright Sections is Reinforced with Rubble Stones

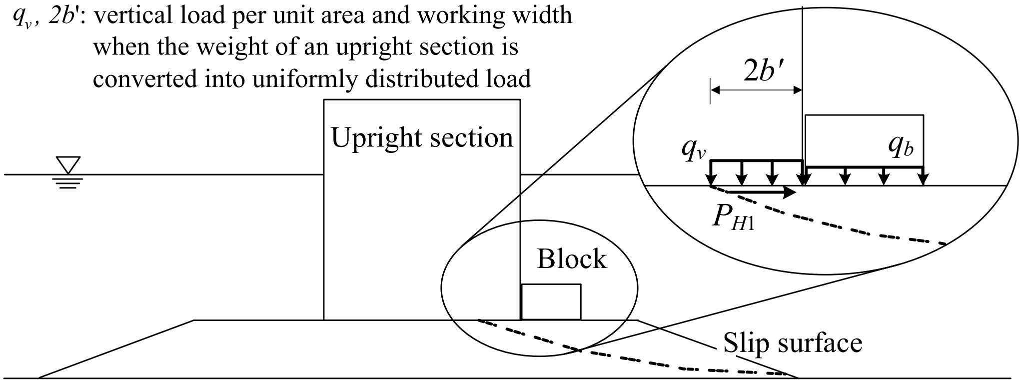


Fig. 3.1.9 Concept of a Deep Slip Surface When the Harbor Side of Upright Sections is Reinforced with Blocks

② When reinforcing breakwaters with embankment widening work made of rubble stones, upright sections are subjected to horizontal wave force P_H, buoyancy P_B, uplift force P_U, reaction force q_v and friction force P_{H1} from foundation mounds, and resistance force P_{H2} and friction force P_V from embankment widening work. The loads acting on a foundation mound and embankment widening work can be calculated using the equation of equilibrium at an upright section and hypothetical conditional equations (i.e., **Equations (3.1.13)** and **(3.1.14])**. Thereafter, the performance verification of bearing capacity can be performed by substituting the calculated loads into **equation (3.1.12)**.

$$P_{Vk} = \tan 15° \cdot P_{H2k} \tag{3.1.13}$$

$$P_{H1_k} = r^* \cdot P_{Hk} \ , \ P_{H2_k} = \left(1 - r^*\right) P_{Hk} \tag{3.1.14}$$

where

r^* : load sharing ratio (a ratio of the portion of horizontal wave force to be resisted by the friction force on the bottom face of an upright section to total horizontal wave force)

③ The load sharing ratios r^* are subjected to cross-sectional shapes, ground materials, and displacement states of upright sections; therefore, it is difficult to set it to a fixed value. By contrast, it has been known that the load sharing ratios r^* have small effects on the stability assessment results.[15] Therefore, the load sharing ratios r^* can be set at 0.5 in the performance verification. However, when P_{H1} and P_{H2} calculated with r^* set at 0.5 exceed the maximum values of $f\,(W-P_B-P_U-P_V)$ and $P_{H2\,\max}$, respectively, the load sharing ratios r^* shall be adjusted to make P_{H1} and P_{H2} less than their maximum values.

④ When reinforcing breakwaters with embankment widening work made of blocks, upright sections are subjected to horizontal wave force P_H, buoyancy P_B, uplift force P_U, and reaction force q_v and friction force P_{H1} from foundation mounds. The load acting on a foundation mound and a working point can be calculated using the equation of equilibrium at an upright section. Furthermore, the effective weight of a block is considered to act on the foundation mound. Thereafter, by using all these loading conditions, the performance verification of bearing capacity can be performed by **equation (3.1.12)**.

3.1.7 Foot Protection Blocks

(1) Breakwaters are preferably provided with foot protection blocks to protect rubble sections from being washed out, except for breakwaters that have very deep rubble sections or are installed in water areas where wave heights are low and rubble mass is sufficient for achieving theoretical stability. It is also preferable that foot protection blocks are brought into tight contact with upright sections.

(2) Armor units for mounds are installed at the front side of foot protection blocks. In terms of ensuring the stability of foot protection blocks, it is preferable to reduce the differences in the levels between armor units and blocks to the greatest extent possible.

(3) Those foot protection blocks provided with holes can reduce the uplift forces acting on them, thereby significantly improving their stability against wave actions.

(4) According to the research by Tanimoto et al.[17)], the aperture ratios of the holes on foot protection blocks are preferably set at approximately 10% because excessively large holes decrease the effect of foot protection blocks to prevent scouring and washing out.

(5) When installing foot protection blocks, it is preferable to arrange them in at least two rows at the seaward side and at least one row at the landward side of upright sections.

(6) The required thicknesses of foot protection blocks can be obtained by **equation (3.1.15)**.[18)]

$$t / H_{1/3} = d_f \left(h'/ h\right)^{-0.787} \tag{3.1.15}$$

where

t : required thickness of a foot protection block (m);

d_f : 0.18 for a trunk section and 0.21 for a head section of a breakwater (m);

h : design water depth (m);

h' : water depth at the top of rubble mound (excluding a block) (m) with the scope of application in the range of h'/h = 0.4 to 1.0.

(7) For the dimensions of foot protection blocks, the required thicknesses can be calculated using **equation (3.1.15)**, and other dimensions can be selected from the values in **Table 3.1.10**. **Fig. 3.1.10** shows examples of the shapes and dimensions of foot protection blocks. In addition to these examples, the shapes of foot protection blocks can be determined with reference to **Reference 19)** and via hydraulic model tests.

Table 3.1.10 Required Thickness and Dimensions of Foot Protection Block (Example)

Required thickness of foot protection blocks t (m)	Dimensions l (m) × b (m) × t (m)	Mass (t/unit)	
		Block with openings	Block without openings
0.8 or less	2.5 × 1.5 × 0.8	6.23	6.90
1.0 or less	3.0 × 2.5 × 1.0	15.64	17.25
1.2 or less	4.0 × 2.5 × 1.2	24.84	27.60
1.4 or less	5.0 × 2.5 × 1.4	37.03	40.25
1.6 or less	5.0 × 2.5 × 1.6	42.32	46.00
1.8 or less	5.0 × 2.5 × 1.8	47.61	51.75
2.0 or less	5.0 × 2.5 × 2.0	52.90	57.50
2.2 or less	5.0 × 2.5 × 2.2	58.19	63.25

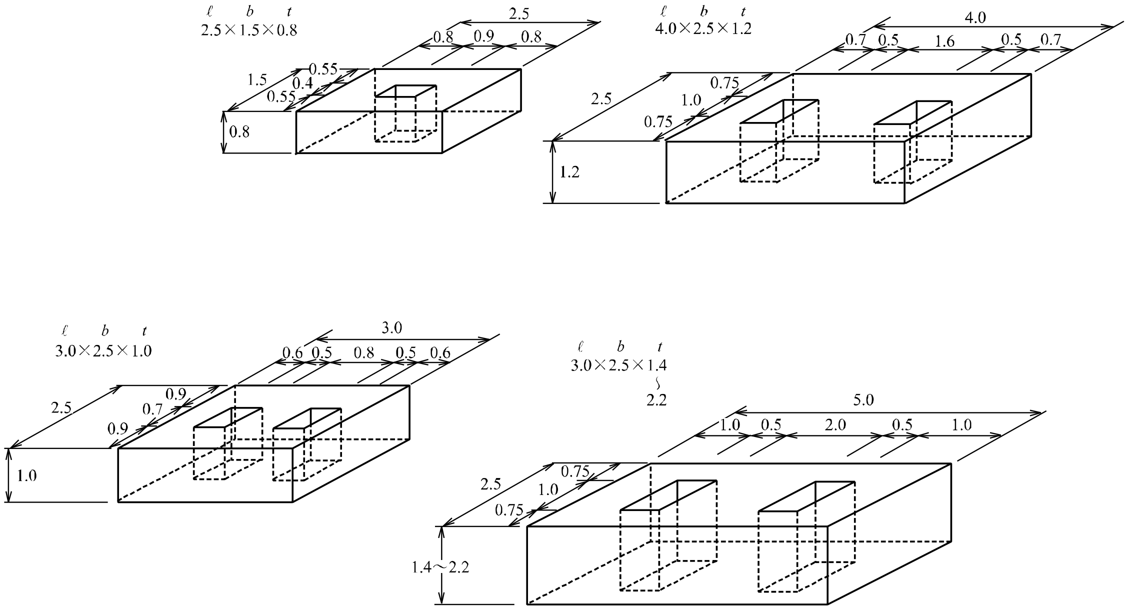


Fig. 3.1.10 Shapes of Foot Protection Blocks (unit in m)

(8) The performance verification of foot protection blocks installed at a harbor side shall be performed by taking into consideration the effects of the waves inside harbors, waves during construction, and overtopping waves as needed.

(9) Given that the occurrences of damage to foot protection blocks at a harbor side are rare, the mass of the foot protection blocks at a harbor side can be smaller than the conventional 1/2 mass at a seaward side provided that the mass shall not be smaller than the required mass determined with consideration to the waves inside harbors and during construction. Particular attention is required in the use of foot protection blocks at the offshore ends of breakwaters as temporary head sections during construction.

3.1.8 Performance Verification of Structural Members

For the performance verification of structural members for caisson type, cellular block type, and hybrid caisson-type breakwaters, refer to **Part III, Chapter 2, 2 Structural Members**.

3.1.9 Structural Details

(1) Items Common to Composite Breakwaters

① The concrete of superstructures shall be constructed with consideration to the integrity of the superstructures with breakwater bodies and shall be provided with joints at proper intervals in the face line directions. The joints are generally installed between caissons for caisson-type breakwaters and at intervals of 10 to 20 m for other types of breakwaters.

② It is preferable to study the necessity of taking measures to curb cracks due to the hydration heat of cement with consideration to the construction conditions of superstructure concrete as needed.

③ Rubble sections preferably undergo heavy weather seasons for the purpose of enhancing their compaction, thereby curbing settlement after the construction of upright sections.

④ To correctly install upright sections, the top surfaces of rubble sections shall be leveled with blinding to ensure flatness with stones that are sufficiently interlocked with one another. Severely uneven rubble surfaces may cause adverse effects on caissons, such as torsional force on caissons and concentrated loads on bottom slabs. The areas of rubble surfaces to be leveled shall consider allowances at both sides of upright sections and include those for foot protection blocks and armor stones.

⑤ Breakwaters with possible risks of scouring and washing out shall be provided with scour and washing-out prevention measures with reference to the descriptions in **Part II, Chapter 2, 7.5 Scouring and Washing Out**. The scour and washing-out prevention measures include small-stepped rubble mounds at slope toes or the protection of slope toes with submerged floor mats, asphalt mats,[20), 21)] or synthetic resin mats. The measures to prevent rubble mounds from settlement due to washing out include the installation of submerged floor mats or the laying of canvas sheets.[22)]

(2) Items for Caisson-type Composite Breakwaters

① The thicknesses of concrete lids of caisson-type composite breakwaters shall be carefully determined by taking into consideration wave and construction conditions.

② The types of materials used as the infill of caissons are concrete, concrete blocks, stones, gravel, sand, slag, etc. It is preferable to determine the types of materials in consideration of construction costs and construction/natural conditions. Sand is the typical material used as the infill of caissons. When using sand and gravel as infill, it is necessary to protect the infill in a manner that completely covers the infill with concrete lids or blocks.

③ Given that some types of slag expand when absorbing water, the quality of the types of slag to be used as infill needs to be carefully studied, including the pretreatment of slag before filling it into caissons.

④ The thicknesses of concrete lids shall be not less than 30 cm for caissons subjected to normal sea conditions or not less than 50 cm for caissons subjected to rough sea conditions. There are even cases wherein concrete lids have thicknesses of not less than 1.0 m for caissons subjected to rough sea conditions for a long period of time with infill protected only by lids without superstructures (refer to **Fig. 3.1.11**). When using precast concrete lids for caissons subjected to rough waves, a rubble layer of 30 to 50 cm thick can be laid beneath each precast concrete lid to prevent possible occurrence of the washing out of filling sand through the gaps between the precast concrete lids and caissons with in situ concrete filled in the gaps washed away by waves.

⑤ There are other cases of caissons that have canvas sheets laid between concrete lids and filling sand as the measure to prevent the filling sand from being washed out through possible cracks in the concrete lids created when the lids are severely hit by rough waves.

⑥ Regarding the wave force acting on superstructure concrete, there are many factors that have not been fully elucidated. Therefore, superstructure concrete shall be constructed so that it can be integrated with breakwater bodies. For the construction joints of superstructure concrete, refer to Standard Specifications for Concrete Structures[23)] by the Japan Society of Civil Engineers. The methods used for enhancing the integration between the superstructure concrete and breakwater bodies include the casting of superstructure concrete with the top sections of caissons embedded into it, provision of concrete lids with indented surfaces (mostly for precast concrete lids), and installation of reinforcing bars or shaped steel between superstructure concrete and caissons (refer to **Fig. 3.1.12**). It is also preferable to provide construction joints between parapets and superstructure concrete with tenons, reinforcing bars, or shaped steel so that they can be integrated.

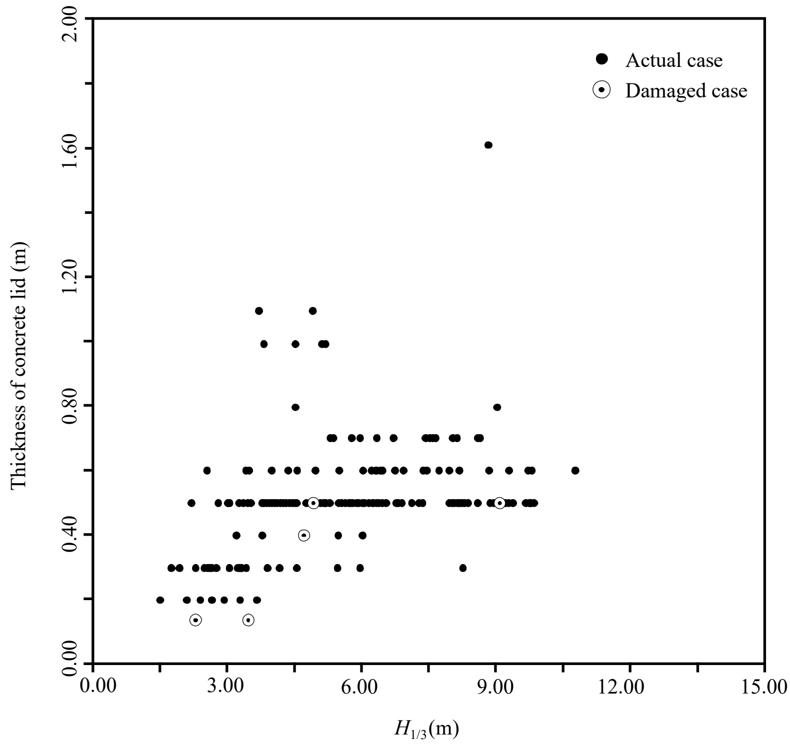


Fig. 3.1.11 Examples of the Construction of Concrete Lids

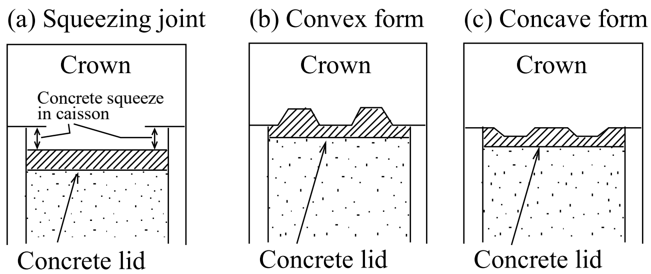


Fig. 3.1.12 Methods for Casting Superstructure Concrete

(3) Items for Block-type Composite Breakwaters

① Block-type composite breakwaters shall have the largest blocks possible. Particularly, it is preferable that the lowermost blocks shall be cast monolithically without joints.

② There are two methods for stacking blocks: one is horizontal stacking, and the other is inclined stacking. Generally, the former can be easily implemented and has been used often. In the case of horizontal stacking, it is preferable that vertical construction joints are not aligned from top to bottom in the cross sections perpendicular to the face lines of breakwaters but are arranged alternately to ensure the integration of breakwater bodies.

③ The inclined stacking method has also been implemented in cases of block-type breakwaters constructed in places with severe erosion or settlement and relatively shallow water depths. In such cases, superstructure concrete is constructed after blocks are sufficiently settled down. It is preferable that each cross section of breakwaters for inclined stacking comprises a single block. The inclination angles are generally 50° to 80° with respect to horizontal planes.

④ It is also preferable that the vertical construction joints are not aligned in the cross sections parallel to the face lines of breakwaters (longitudinal cross sections).

⑤ Blocks are generally provided with tenons and mortises (**Fig. 3.1.13**) to enable them to interlock with each other, thereby preventing them from sliding. Generally, the widths *a* and heights *b* of tenons are approximately

50 and 20 cm, respectively, and the widths *a'* and heights *b'* of mortises are larger than those of tenons by approximately 5 cm.

To prevent blocks from sliding, there is an alternative method in which blocks with prefabricated through-holes are stacked and integrated one above the other by filling the through-holes with concrete or by inserting steel materials into the through-holes with the gaps in them filled with mortar. In such a method, excessively small through-holes reduce the effect of preventing blocks from sliding and excessively large through-holes may cause the destruction of blocks. There are other alternative methods that use interlocking blocks, including deformed blocks with hexagonal or drum shapes. However, in the performance verification, the effects of these deformed interlocking blocks are generally ignored.

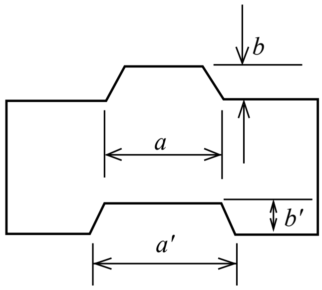


Fig. 3.1.13 Tenon and Mortise of Concrete Blocks

(4) Items for Cellular Block-type Composite Breakwaters

① The thicknesses of concrete lids of cellular block-type composite breakwaters shall be carefully determined by taking into consideration the wave and construction conditions.

② The lowermost cellular blocks should have footings to enhance stability.

③ Concrete or stones can be used as the infill of cellular blocks.

④ When stacking cellular blocks, the integration of cellular blocks shall be enhanced by interlocking effects with the tenons and mortises provided at the top and bottom portions of cellular block walls (**Fig. 3.1.1 [c]**).

⑤ Cellular blocks that are filled with stones may have bottom slabs to prevent the stones from being extruded.

(5) Items for In Situ Concrete Block-type Composite Breakwaters

① In many cases, each concrete block constituting the upright sections of in situ concrete block-type composite breakwaters has a size of 5 to 10 m to prevent cracks due to contraction or uneven settlement.

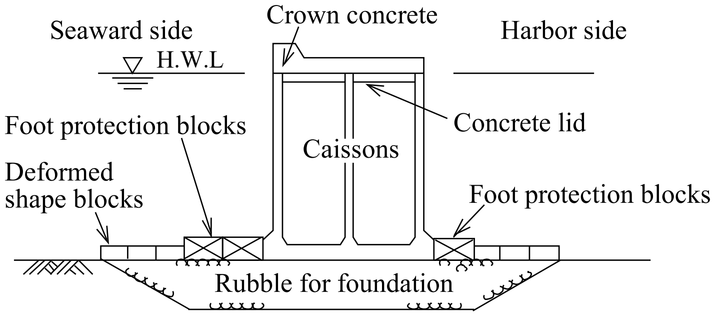
② In situ concrete can be cast by the underwater concrete, prepacked concrete, or dry work method.

3.2 Gravity-type Breakwaters (Upright Breakwaters)

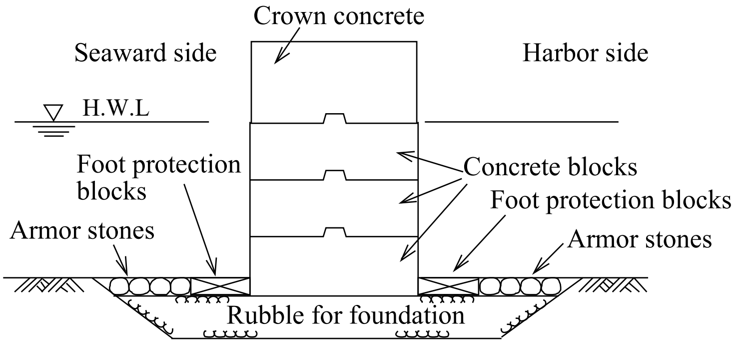
3.2.1 General

(1) Upright breakwaters have structures in which wall bodies with vertical front walls are installed on the seabed to reflect wave energy.

(2) **Fig. 3.2.1** shows examples of the cross sections of upright breakwaters.



1) Caisson type upright breakwater



2) Concrete block type upright breakwater

Fig. 3.2.1 Examples of Upright Breakwaters

3.2.2 Setting of Basic Cross Sections

The cross sections of upright breakwaters shall be appropriately set in conformity with **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

3.2.3 Actions

The actions for upright breakwaters shall be appropriately set in conformity with **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

3.2.4 Performance Verification

The performance verification of upright breakwaters can be performed with reference to **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

3.2.5 Structural Details

(1) The structural details of upright breakwaters shall be appropriately examined with reference to **Part III, Chapter 4, 3.1 Gravity-type Breakwaters (Composite Breakwaters)**.

(2) When constructing breakwater bodies with in situ concrete, unevenness on foundation surfaces is acceptable, but such surfaces shall be free from sand, rock fractions, or seaweeds to ensure the adhesiveness between in situ concrete and foundations. Furthermore, the portions of foundation surfaces that are brought into contact with

formworks shall be leveled to ensure the adhesiveness. In cases wherein there are difficulties in leveling remarkably hard and uneven seabed, it is preferable to ensure the adhesiveness between breakwater bodies and the seabed by flexibly adjusting the shapes of formworks in accordance with the topography of the uneven seabed.

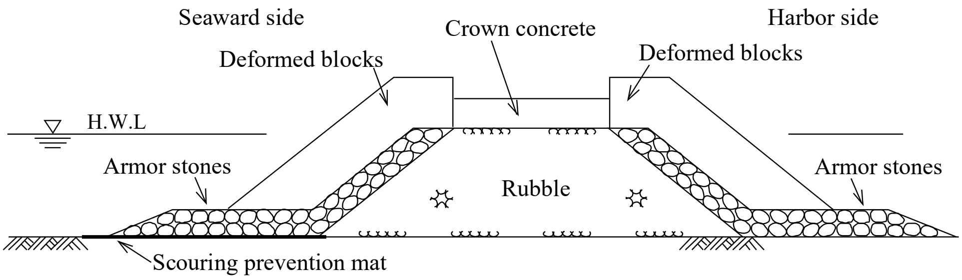
(3) Considering that the lower sections of upright breakwaters are susceptible to scouring, upright breakwaters constructed on the seabed with no bedrock exposed shall be fully provided with foot protection work.

3.3 Gravity-Type Breakwaters (Sloping Breakwaters)

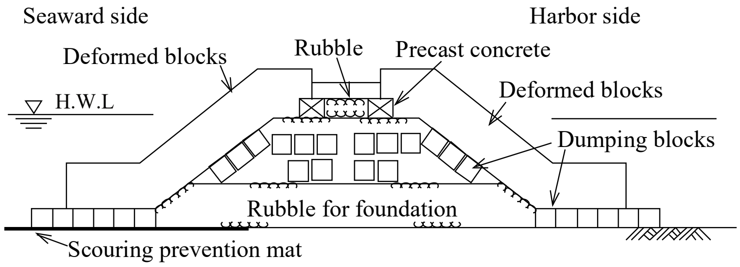
3.3.1 General

(1) Sloping breakwaters have structures where stones or concrete blocks are stacked in trapezoidal shapes in cross sections primarily for dissipating wave force by allowing waves to break on the slopes in front of breakwater bodies.

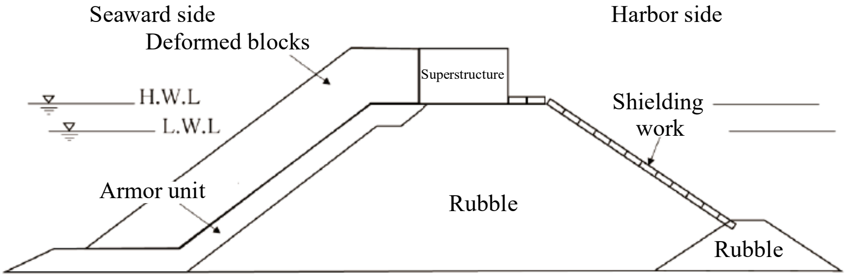
(2) **Fig. 3.3.1** shows examples of the cross sections of sloping breakwaters.



(a) Rubble mound breakwater



(b) Dumping block mound sloping breakwater



(c) Rubble mound breakwater

Fig. 3.3.1 Examples of the Cross Sections of Sloping Breakwaters

3.3.2 Setting of Basic Cross Sections

(1) The crown heights of sloping breakwaters can be set in conformity with composite breakwaters as described in **Chapter 4, 3.1.2 Setting of Basic Cross Sections**. Furthermore, the crown heights can be set in accordance with the intended use of the crowns.

(2) It shall be noted that because sloping breakwaters transmit waves, they may cause wave heights inside harbors to be higher than the cases of upright breakwaters even though the crown heights are identical. For overtopping and

transmitted waves, refer to **Part II**, **Chapter 2, 4.4.7 Wave Run-Up Heights, Overtopping Waves, and Transmitted Waves**.

(3) The crown widths can be set on the basis of the results of appropriate model tests.

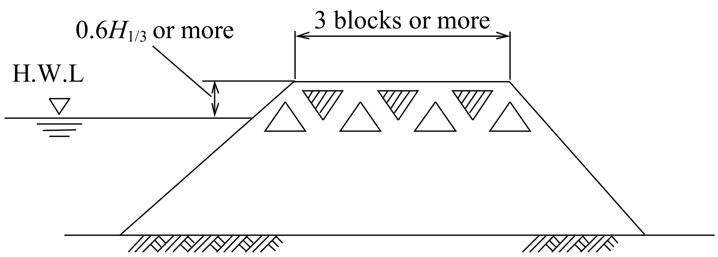
(4) Sloping breakwaters subjected to significant overtopping waves shall have sufficiently wide crown widths to prevent armor units at the top sections from being placed into an unstable state.

(5) The crown widths of rubble mound breakwaters to be constructed in a manner that extends rubble mounds from beaches out to the sea are preferably determined to satisfy the performance verification and to facilitate construction work.

(6) The crown heights and construction methods for sloping breakwaters to be constructed on soft ground can be set with reference to those for composite breakwaters as described in **Chapter 4, 3.1.2 Setting of Basic Cross Sections**.

(7) When deformed blocks are used for sloping breakwaters with crown heights of approximately $0.6H_{1/3}$ above the mean-monthly highest water levels, the crown widths can be equivalent to three rows of the deformed blocks or more (**Fig. 3.3.2**). However, it is preferable that the crown widths are determined via an appropriate model tests because the stability of the top sections of sloping breakwaters varies depending on the characteristics of armor units and wave conditions.

(8) According to the actual cases, the slope gradients are generally set at approximately 1:2 and 1:1.5 for the seaward and landward sides of rubble mound breakwaters, respectively, and approximately 1:1.3 to 1:1.5 for the rubble mound breakwaters covered by concrete blocks. In cases of seaward side slopes with different gradients and mass of armor units between upper and lower sections, the boundaries of the different gradients and mass shall be positioned at least $1.5H_{1//3}$ deeper than still water levels in general.



The number of pieces listed above are the number of hatched blocks in the upper layer of the crown.

Fig. 3.3.2 Crown Width of Sloping Breakwater

(9) The sloping breakwaters in the Europe and the U.S. tend to have high crown heights as a result of using the 2% exceedance run-up heights $R_{2\%}$ of random waves as target crown heights.

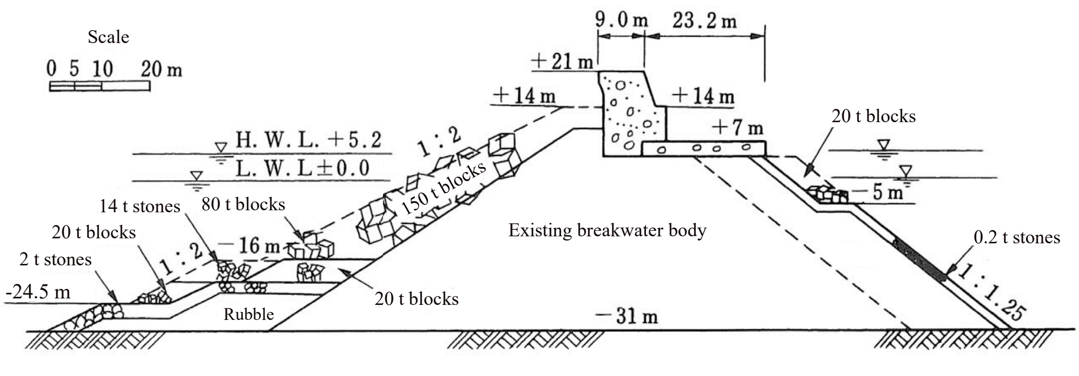


Fig. 3.3.3 Rubble Mound Breakwater (Typical Cross Section of the Head Section of Outer Breakwater at the Port of Bilbao[24])

3.3.3 Performance Verification

(1) Performance verification items for sloping breakwaters

① Sloping breakwaters have problems with overtopping and transmitted waves and are subjected to the following failure modes: scouring and breakages of armor units; breakages, sliding, and overturning of superstructures; slip failures of front slopes; scouring of mounds below armor units; settlement of core materials; scouring of sandy ground at slope toes; washing-out of fine particle components due to internal instability of filtering materials; and ground settlement (**Fig. 3.3.4**). Therefore, the performance verification of sloping breakwaters shall be performed to prevent these failure modes.

② The performance verification items for sloping breakwaters include the stability of superstructures; the stability of armor units (rubble stones, concrete blocks, and deformed concrete blocks) at sloped sections, the required mass of rubble stones and blocks below the armor units at sloped sections and their internal stability as filtering layers, and the bearing capacity of sloped sections and ground.

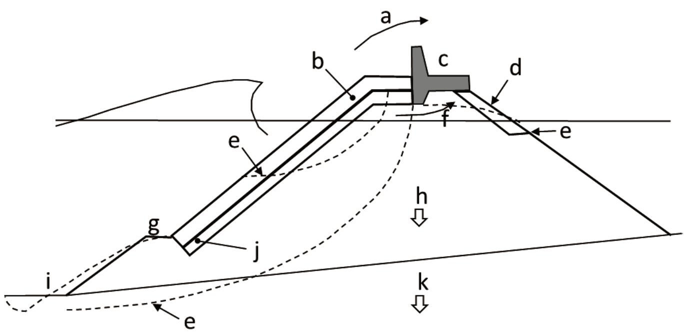


Fig. 3.3.4 Failure Modes of Sloping Breakwater (Refer to ISO 21650)

a: Overtopping waves b: Scouring and breakages of armor units
c: Breakages, sliding, and overturning of superstructures
d: Scouring of armor units e: Slip failures of front slopes
f: Transmitted waves g: Scouring of mounds below armor units;
h: Settlement of core materials i: Scouring of sandy ground at a slope toe
j: Internal instability of filtering materials
k: Ground settlement

(2) Performance verification of the stability of superstructures

① The verification of the stability of superstructures under the variable situation with respect to waves shall be performed for the sliding and overturning of superstructures.

② The verification of the stability of superstructures under the variable situation with respect to waves shall be performed using **Equations** (**3.3.1**) and (**3.3.2**). In these equations, the symbol γ is the partial factor for each subscript. Furthermore, subscripts k and d indicate the characteristic value and design value, respectively. The partial factors in the equation can be selected from the values in **Tables 3.3.1** and **3.3.2**, in which the "—" symbol in a column indicates that the value in parentheses in the column can be used for the performance verification of convenience.

(a) Verification of sliding

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

$$R_k = \{f_k (W_k - P_{B_k} - P_{U_K})\} \qquad \textbf{(3.3.1)}$$

$$S_k = P_{Hk}$$

where

f : a friction coefficient between a superstructure and rubble stones;

W : weight of a superstructure (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift (kN/m);

P_H : horizontal wave force (kN/m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.3.1 Partial Factors Used for the Performance Verification of Sliding of Superstructures

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of superstructure (Variable state of waves)	– (1.00)	– (1.00)	1.20

(b) Verification of overturning

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

$$R_k = a_1 W_k - a_2 P_{B_k} - a_3 P_{U_k} \tag{3.3.2}$$

$$S_k = a_4 P_{H_k}$$

where

W : weight of a superstructure (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift (kN/m);

P_H : horizontal wave force (kN/m);

a_1 to a_4 : arm lengths of respective actions (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : partial factor multiplied by resistance term;

γ_S : partial factor multiplied by load term;

m : adjustment factor.

Table 3.3.2 Partial Factors Used for the Performance Verification of the Overturning of Superstructures

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of superstructure (Variable state of waves)	− (1.00)	− (1.00)	1.20

③ It is necessary to appropriately calculate the characteristic values for the weight W_k and the buoyancy of a superstructure in **equations** (**3.3.1**) and (**3.3.2**).

(3) Performance verification of armor units for sloped sections

① One of the methods for covering sloped sections is to use rubble stones or deformed concrete blocks as armor units, and another method is to cover sloped surfaces with sand mastic.

② The armor units for rubble sections shall have sufficient mass to ensure stability against waves and sufficient thicknesses to prevent infill from being washed out.

③ When calculating the required mass of armor units, refer to **Part II, Chapter 2, 6.6 Stability of Armor Rocks and Blocks against Waves** or **ISO 21650**.

④ The required mass of armor units shall be appropriately set when constructing armor layers not randomly but by orderly arranging armor units or by laying armor stones. The number of layers is generally set at two when constructing armor layers by randomly arranging armor units.

⑤ For the use of sand mastic to cover sloped surfaces, refer to past use cases and the research outcome[7)].

(4) Required mass of rubble stones and blocks below armor units at sloped sections and their internal stability as filter layers

① The required mass of filter layers (rubble stones and blocks) below armor units at sloping breakwaters are preferably approximately 1/10 to 1/15 or more of the mass of armor units. The mass of the stones (core materials) below the filter layers are preferably approximately 1/20 or more of the mass of filter layers.[1)]

② The verification of the stability of the mass of the stones (core materials) below the filter layers can be performed with reference to the following equation (ISO 21650):

$$\frac{d_{15,filter}}{d_{85,core}} < 4 \ to \ 5$$
$$\frac{W_{50,filter}}{W_{50,core}} < 15 \ to \ 20 \qquad (3.3.3)$$

where d and W represent the particle diameter and the mass of a stone or a concrete block, respectively; $d_{15,\mathrm{filter}}$ is the sieve size for 15% passing by mass; $d_{85,\mathrm{core}}$ is the sieve size 85% passing by mass; $W_{50,\mathrm{filter}}$ is the mass of a filter material with a median diameter; and $W_{50,\mathrm{core}}$ is the mass of a core material having a median diameter.

Furthermore, the verification of the internal stability of filter materials can be performed with reference to the following condition.

$$\frac{d_{60}}{d_{10}} < 10 \qquad (3.3.4)$$

(5) Bearing capacity of sloped sections and ground

① The stability of the sloped section of sloping breakwaters can be examined via the verification of the circular slip failures of rubble layers and their sliding failures due to eccentric and inclined loads.

② For the verification of the circular slip failures of rubble layers and the sliding failures due to eccentric and inclined loads, refer to **Part III, Chapter 2, 4.2.1 Stability Analysis by Circular Slip Surfaces** and **Part III, Chapter 2, 3.2.5 Bearing Capacity of Eccentric and Inclined Actions**, respectively.

(6) Performance verification of the stability of head sections

The head sections of sloping breakwaters are preferably constructed in a semicircular form by using armor units with a 1.5 times larger or more mass than those used for trunk sections. When calculating the mass of armor stones and wave dissipating blocks, refer to **Part II, Chapter 2, 6.6 Stability of Armor Stones and Blocks against Waves**. Generally, it is preferable that the stability of head sections be verified via hydraulic model tests.

3.3.4 Performance Verification of Structural Members

For the performance verification of structural members, refer to **Part III, Chapter 2, 2 Structural Members**.

3.3.5 Structural Details

(1) The foundations of sloping breakwaters shall be provided with scouring and washing-out prevention measures as needed.

(2) The scour prevention measures include small stepped rubble mounds at slope toes or the protection of slope toes with rubble blocks, submerged floor mats, asphalt mats, or synthetic resin mats (refer to **Fig. 3.3.1**).

(3) The measures to prevent rubble mounds from settlement due to washing-out include the installation of submerged floor mats or the laying of canvas sheets.

(4) Generally, when constructing superstructures on rubble block and rubble mound breakwaters, the rubble foundations of superstructures shall be blinded with small rubble blocks.

(5) The surface finish work of sloping breakwaters shall be implemented in a manner that ensures the adequate interlocking effects of surface armor unit materials with careful attention to the finishing of crown sections.

(6) In coastal areas affected by littoral drifts, sloping breakwaters are preferably provided with sediment infiltration prevention work to prevent harbors from possible siltation owing to sand passing through sloping breakwaters together with waves.

(7) Sediment infiltration prevention work is normally implemented in a manner that constructs walls with sheet piles or blocks inside breakwaters or dumps stone materials with a wide particle size distribution inside the sloping breakwaters or on the slopes at a harbor side.

(8) It shall be noted that sloping breakwaters are susceptible to wave actions that scatter stones.

(9) For the mixture of materials to be used when covering sloping breakwaters by sand mastic method, refer to **Part II, Chapter 11, 4 Asphalt Materials**.

(10) When constructing sloping breakwaters on soft ground, the settlement and subduction of breakwater bodies generally cause the quantities of rubble stones or blocks required in actual construction to be significantly larger than those based on the cross sections obtained by performance verification. Even in cases of favorable ground conditions, additional quantities of stones are preferably procured in actual construction in anticipation of the scattering and consolidation of stones due to waves.

3.4 Gravity-Type Breakwaters (Breakwaters Covered with Wave-Dissipating Blocks)

3.4.1 General

(1) Breakwaters covered with wave-dissipating blocks have structures wherein wave-dissipating blocks are installed in front of composite or upright type breakwaters so that wave energy can be dissipated by the blocks and transmitted waves can be blocked by upright sections.

(2) **Fig. 3.4.1** shows examples of the cross sections of breakwaters covered with wave-dissipating blocks.

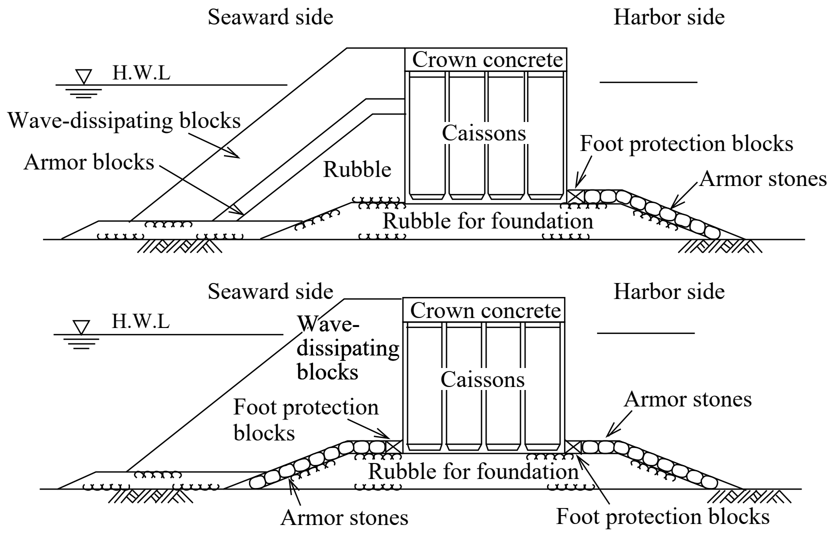


Fig. 3.4.1 Examples of Cross Sections of Breakwaters Covered with Wave-Dissipating Blocks

3.4.2 Setting of Basic Cross Sections

(1) The crown height of breakwaters covered with wave-dissipating blocks can be set in conformity with composite breakwater as described in **Chapter 4, 3.1.2 Setting of Basic Cross Sections**.

(2) Generally, wave-dissipating blocks with excessively lower crown heights than upright sections may allow impulsive breaking wave force to act on the upright sections, and wave-dissipating blocks with excessively higher crown heights than upright sections may destabilize the uppermost wave-dissipating blocks.

(3) Wave-dissipating blocks shall have crown widths equivalent to two rows of wave-dissipating blocks or more to enable them to exert the full wave-dissipating effect.[25), 26)]

(4) The thicknesses of superstructures and the crown heights of installed caissons can be set in compliance with upright breakwaters, and the thicknesses of rubble sections can be set in compliance with composite breakwaters.

(5) If crown heights are identical, breakwaters covered with wave-dissipating blocks can attenuate the degrees of overtopping and transmitted waves compared with upright and composite breakwaters. For overtopping and transmitted waves, refer to **Part II, Chapter 2, 6 Waves**.

(6) Wave-dissipating blocks can reduce wave pressure, attenuate the overtopping and transmitted waves, and curb reflected waves. Model tests should be conducted to accurately assess these effects.

(7) It is necessary to pay attention to the possible impulsive breaking wave force applied to the portions of upright sections where wave-dissipating blocks have not been fully installed during construction.

3.4.3 Actions

(1) For the actions on breakwaters covered with wave-dissipating blocks, refer to **Chapter 4, 3.1.3 Actions.**

(2) Generally, the self-weight of wave-dissipating blocks leaning on caissons is not considered an action on the caissons when breakwaters are subjected to waves. When considering the self-weight of wave-dissipating blocks as the action on caissons, refer to **Part II, Chapter 2, 6.2.5 Wave Force Acting on Vertical Walls Covered by Wave-Dissipating Blocks**.

3.4.4 Performance Verification of the Overall Stability of Breakwater Bodies

(1) For the verification of the stability of breakwaters covered with wave-dissipating blocks, refer to **Chapter 4, 3.1 Gravity-Type Breakwaters (Composite Breakwaters)**.

(2) The performance verification of the sliding and overturning failures of breakwaters covered with wave-dissipating blocks with respect to variable waves shall be performed using **equations (3.1.2)** and **(3.1.4)**. The partial factors in the equation can be selected from the values in **Tables 3.4.1** and **3.4.2**, in which the "—" symbol in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

Table 3.4.1 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of breakwater body (Variable state of waves)	0.79	0.90	– (1.00)

Table 3.4.2 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	0.98	0.99	– (1.00)

(3) The partial factors above were set with reference to the safety levels in past standards.[5)] Furthermore, the partial factors are set under the conditions that the topographies of seabed where breakwaters are installed have gradients of less than 1/30. In cases wherein the topographies of seabed have gradients larger than 1/30, partial factors shall be appropriately set with reference to the descriptions in **Reference 5)**.

(4) In **Reference 5)**, the partial factors were set under the condition that the crown heights of wave-dissipating blocks are not affected by settlement. Given that the crown settlement of wave-dissipating blocks poses a risk of the immediate destabilization of breakwater bodies when the wave force acting on them increases, it is necessary to pay attention to providing wave-dissipation blocks with settlement prevention measures.

3.4.5 Performance Verification of Other Items Related to the Overall Stability of Breakwater Bodies

(1) Performance verification of the stability of sloped sections

① For calculating the required mass of armor units for breakwaters covered with wave-dissipating blocks, refer to **Part II**, **Chapter 2, 6.6.2 Required Mass of Armor Rocks and Blocks of Composite Breakwater Mound against Waves**.

② The required mass of armor units shall be appropriately set when constructing armor layers not randomly but by orderly arranging armor units or by laying armor stones. The number of layers is generally set at two when constructing armor layers by randomly arranging armor units.

(2) Performance verification of the stability of head sections

The head sections of breakwaters covered with wave-dissipating blocks are preferably constructed in a semicircular form by using armor units with a 1.5 times larger or more mass than those used for trunk sections. When calculating the mass of wave-dissipating blocks, refer to **Part II, Chapter 2, 6.6.2 Required Mass of Armor Rocks and Blocks of Composite Breakwater Mound against Waves**.

(3) Performance verification of the stability of wave-dissipating works

For the performance verification of the stability of wave-dissipation works, refer to Part II, Chapter 2, 6.6.2 Required Mass of Armor Rocks and Blocks of Composite Breakwater Mound against Waves.

(4) In the performance verification of the accidental situation with respect to Level 2 earthquake ground motions, refer to Reference **(Part III), Chapter 1, 2 Basic Items Concerning Earthquake Response Analysis**.

3.4.6 Performance Verification of Structural Members

The performance verification of structural members can be performed with reference to **Part III, Chapter 2, 2 Structural Members**.

3.4.7 Structural Details

(1) The dimensions of foot protection blocks at the seaward side of breakwaters covered with wave-dissipating blocks can be calculated by **equation (3.1.15)** in **Chapter 4, 3.1 Gravity-Type Breakwaters (Composite Breakwaters)** for the waves during construction by taking into consideration the periods that breakwaters are left without wave-dissipating blocks during construction.

(2) Generally, because wave-dissipating blocks are susceptible to the scouring and washing out of foundation ground due to waves, they are provided with scouring and washing-out prevention work at the slope toes to prevent settlement, as needed.[27), 28), 29)] For the souring and washing-out prevention work, refer to **Chapter 4, 3.3.5 Structural Details**.

3.5 Gravity-Type Breakwaters (Upright Wave-Absorbing Block-Type Breakwaters)

3.5.1 General

(1) Upright wave-absorbing block-type breakwaters are mass concrete block-type upright breakwaters or composite breakwaters that are constructed by directly stacking special blocks with a wave-absorbing function (upright wave-absorbing blocks).

(2) Upright wave-absorbing block-type breakwaters, except those with large-scale monolithic structures, are normally used as breakwaters in inner bays or inner harbors where wave heights are relatively low.

(3) **Fig. 3.5.1** shows an example of a cross section of an upright wave-absorbing block-type breakwater.

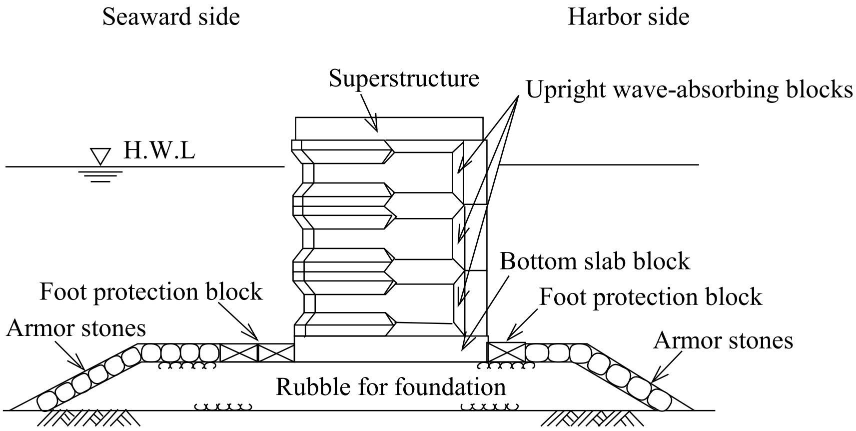


Fig. 3.5.1 Example of a Cross Section of an Upright Wave-Absorbing Block-Type Breakwater

3.5.2 Setting of Basic Cross Sections

(1) It is necessary that the structural dimensions of upright wave-absorbing block-type breakwaters are determined to enable them to deliver the required wave-absorbing performance.

(2) The crown heights of the upright wave-absorbing block-type breakwaters can be decided by considering the heights that satisfy the performance requirements and the heights of the wave-absorbing sections and by referring to **Chapter 4, 3.1.4 Performance Verification of Overall Stability of Breakwater Bodies**. The crown heights of the wave-absorbing sections shall be determined by considering the wave-absorbing performance. In cases of structures with permeability, the dimensions of the opening sections should be determined by considering the transmission characteristics.

(3) The wave-absorbing performance of the upright wave-absorbing block-type breakwaters vary depending on the crown heights and bottom elevations of the wave-absorbing block sections.

(4) Several types of wave-absorbing blocks have been developed. Appropriate types of blocks are preferably selected after sufficiently studying their wave-absorbing performance.

(5) In the upright wave-absorbing block-type breakwaters, wave over-topping and transmitted waves are small in comparison with those with composite breakwaters but tend to be larger than those with breakwaters covered with

wave-absorbing blocks. Accordingly, it is preferable that the crown heights be determined by giving adequate consideration to the conditions of use behind breakwaters. Furthermore, in determining the crown heights, the thickness required for constructing the crown concrete should be secured.

(6) It is preferable that the crown heights h_c' be at least 0.5 times or higher the significant wave heights used in the stability examination of the facilities above the mean monthly high-water levels. The bottom heights h_u should be at least two times or higher the significant wave height used in the stability examination of the facilities below the mean monthly high-water levels (see **Fig. 3.5.2**).

(7) For the run-up heights of the upright wave-absorbing block-type breakwaters and the characteristics of several types of wave-absorbing blocks, refer to **References 30)** and **31)**, respectively.

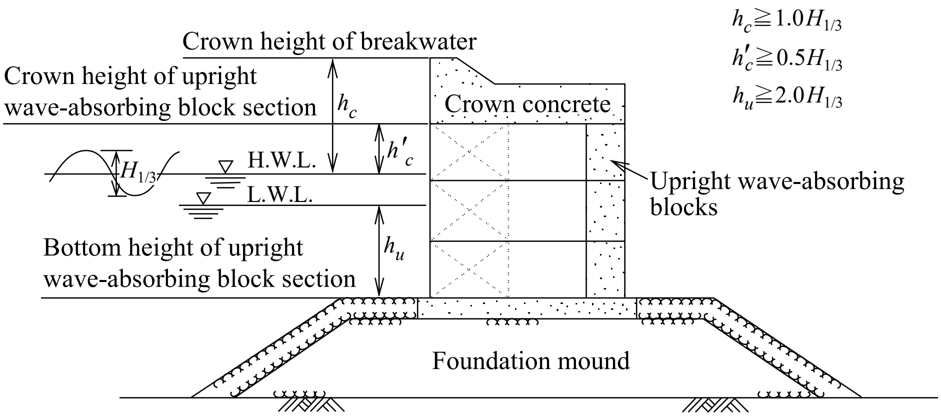


Fig. 3.5.2 Explanatory Diagram for the Crown Height of Upright Wave-Absorbing Block-Type Breakwater

3.5.3 Actions

(1) Depending on the purpose of absorbing waves and wave conditions, the characteristics of waves used for the verification of wave-absorbing performance can be determined separately from those used for the performance verification of the stability of facilities and structural members.

(2) Wave force shall be determined by the equation appropriate for upright wave-absorbing block-type breakwaters or via hydraulic model tests simulating actual wave conditions. For breakwaters with complex structures, the wave force acting on structural members should be studied in addition to the wave force used for verifying the stability of entire upright sections. For the wave force acting on upright wave-absorbing block-type breakwaters, refer to **Part II, Chapter 2, 6.2.7 Wave Force Acting on Upright Wave-Absorbing Caissons**.

(3) Considering that the reflection coefficients of upright wave-absorbing block-type breakwaters significantly vary depending on wave periods, the reflection coefficients shall be determined with due consideration to the influences of reflection waves. The reflection coefficients are preferably determined by hydraulic model tests that simulate actual conditions or may be determined by referring to existing test results.

3.5.4 Performance Verification of Overall Stability of Breakwater Bodies

(1) Given that the hydraulic characteristics of upright wave-absorbing block-type breakwaters such as the transmittance of waves and water permeability have not been fully elucidated, the performance verification with respect to wave actions are preferably performed on the basis of hydraulic model tests simulating actual conditions.

(2) The verification of the stability of upright wave-absorbing block-type breakwaters can be performed in conformity with that of composite breakwaters. The standard partial factors to be used for the verification of sliding and overturning failures are shown in **Tables 3.5.1 and 3.5.2**, in which the "—" symbol in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

Table 3.5.1 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

Table 3.5.2 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

3.5.5 Performance Verification of Structural Members

The performance verification of structural members can be performed with reference to **Part III, Chapter 2, 2 Structural Members**.

3.6 Gravity-Type Breakwaters (Wave-Absorbing Caisson-Type Breakwaters)

3.6.1 General

(1) Wave-absorbing caisson-type breakwaters are classified as deformed caisson breakwaters that use caissons with special shapes. They are provided with porous walls and wave chambers at their front sections to deliver the wave-absorbing effect.[32)]

(2) Compared with composite breakwaters, wave-absorbing caisson-type breakwaters have the following features:

① They can curb reflected waves.

② They can attenuate wave overtopping and transmitted waves.

③ They can reduce wave force. In particular, they can curb significant increases in wave force even under severe wave conditions, whereas conventional caissons on high foundation mounds undergo strong impulsive breaking wave force.

④ They possess a seawater aeration function with porous walls and wave chambers that enhance the mixing of air bubbles with seawater. Furthermore, wave chambers are effective as fishing banks.[33), 34)]

(3) **Fig. 3.6.1** shows an example of the cross section of a wave-absorbing caisson-type breakwater. Depending on the shapes of the respective elements and the combination of elements, various types of structures are conceivable, including vertical slit-wall caissons, horizontal slit-wall caissons, curved-slit caissons, perforated-wall caissons, and others. Regarding the structural type for wave-absorbing caisson-type breakwaters, an appropriate structure should be selected by considering the design conditions, use conditions, economy, etc., on the basis of a careful investigation of the wave-absorbing performance and wave resistance of each structure.

(4) For the structures and the features of various types of wave-absorbing caisson-type breakwaters, the **Technical Manual of New Type Breakwaters**[35)] can be used as a reference.

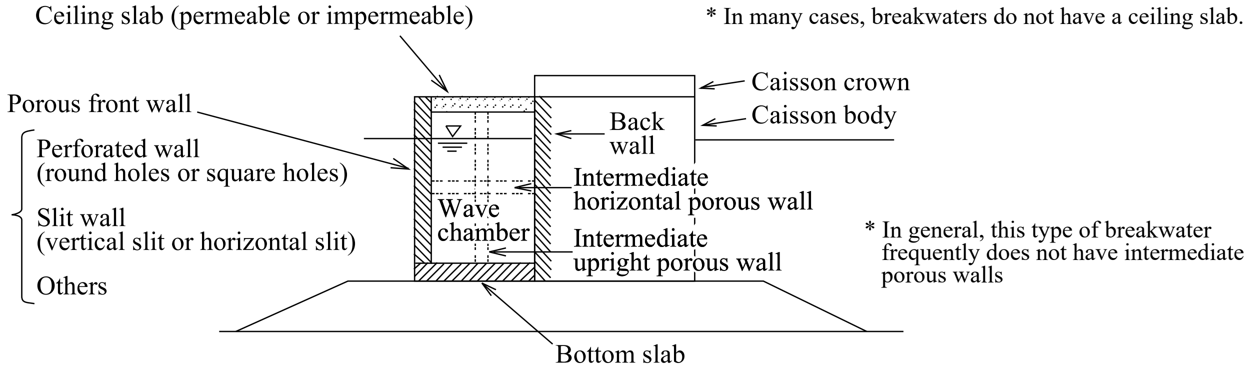


Fig. 3.6.1 Example of a Cross Section of Wave-Absorbing Caisson-Type Breakwater

3.6.2 Setting of Basic Cross Sections

(1) The structures of wave-absorbing caisson-type breakwaters shall be appropriately selected with due consideration to their wave-absorbing performance.

(2) In wave-absorbing caisson-type breakwaters, the required dimensions should be determined appropriately by considering the shapes of the structures. Given that the transmission coefficients differ depending on the structures or wave conditions, it is preferable that the crown heights that correspond to the transmission characteristics of the objective structures should be determined appropriately. In cases wherein the structures have permeability, it is preferable that the dimensions of the opening sections should be determined appropriately.

(3) In addition to wave-absorbing performance, the structure and dimensions of the wave-absorbing section are also related to wave overtopping, transmitted waves, and wave force. Therefore, it is preferable to determine the dimensions and structure by considering these characteristics.

(4) The reflection coefficients of wave-absorbing caisson-type breakwaters vary depending on the factors, including wave characteristics, water depths, structures of frontal porous walls, widths of wave chambers, presence or absence of ceiling slabs and their heights, heights of mounds, and others. Therefore, the structural dimensions of wave-absorbing sections shall be appropriately determined so that the reflection coefficients of object waves do not exceed the target reflection coefficients with due consideration to the effects of the above factors. In terms of enhancing the wave-absorbing performance, it is preferable that wave chambers have sufficiently high crown heights or have no ceiling slabs (permeable).

(5) For the reflection characteristics of vertical slit-wall caissons without ceiling slabs, the research by Tanimoto and Yoshimoto[36)] can be used as a reference.

3.6.3 Actions

(1) Depending on the purpose of absorbing waves and wave conditions, the conditions of waves used for the verification of wave-absorbing performance can be determined separately from those used for the performance verification of the stability of facilities and structural members.

(2) In many cases, wave-absorbing caissons are generally adopted to reduce reflected waves. Consequently, it is preferable to determine the conditions of the waves to be objects of wave absorption and target reflection coefficients corresponding to the required wave-absorbing performance. In particular, because the reflection coefficients of wave-absorbing caissons differ remarkably depending on the wave periods, the conditions of waves as objects of wave absorption shall be determined on the basis of the investigations of the characteristics of wave heights and wave periods.

(3) The wave force shall be determined by the equation appropriate for wave-absorbing caisson-type breakwaters or through hydraulic model tests simulating actual wave conditions. For breakwaters with complex structures, it is preferable to sufficiently study the wave force acting on structural members, in addition to the wave force used for verifying the stability of entire upright sections. For the wave force acting on wave-absorbing caisson-type breakwaters, refer to **Part II, Chapter 2, 6.2.7 Wave Force Acting on Upright Wave-Absorbing Caissons**.

(4) The performance verification of structural members shall be performed with the most severe wave force for respective members. For the wave force acting on the structural members of wave-absorbing caisson-type

breakwaters, refer to **Part II, Chapter 2, 6.2.7 Wave Force Acting on Upright Wave-Absorbing Caissons** and **[Facilities], Chapter 4, 3.5.3 Actions**.

3.6.4 Performance Verification

(1) Given that the hydraulic characteristics of wave-absorbing caisson-type breakwaters such as the transmittance of waves, reflection rates and water permeability have not been fully elucidated, the performance verification with respect to wave actions are preferably performed on the basis of hydraulic model tests as needed.

(2) The performance verification of the stability of wave-absorbing caisson-type breakwaters can be performed in conformity with that of composite breakwaters. The standard partial factors to be used for the verification of sliding and overturning failures are shown in **Tables 3.6.1** and **3.6.2**, in which the "—" symbol in a column means that the value in parentheses in the column can be used for the performance verification for convenience.

Table 3.6.1 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

Table 3.6.2 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of breakwater body (Variable state of waves)	— (1.00)	— (1.00)	1.20

3.6.5 Performance Verification of Structural Members

The performance verification of structural members can be performed with reference to **Part III, Chapter 2, 2 Structural Members**.

3.7 Gravity-Type Breakwaters (Sloping-Top Caisson Breakwaters)

3.7.1 General

(1) Sloping-top caisson breakwaters are classified as deformed caisson breakwaters that use caissons with special shapes. They are configured to reduce horizontal wave force while simultaneously using part of the wave force acting on the sloping walls of the superstructures to stabilize breakwater bodies. They are generally called sloping-top breakwaters.

(2) In sloping-top caisson breakwaters, the sloping walls of the superstructures are normally positioned above still water levels; however, semi-submerged sloping walls with their lower ends positioned below still water levels can further reduce wave force.[37)]

(3) **Fig. 3.7.1** shows an example of the cross section of a sloping-top caisson breakwater.

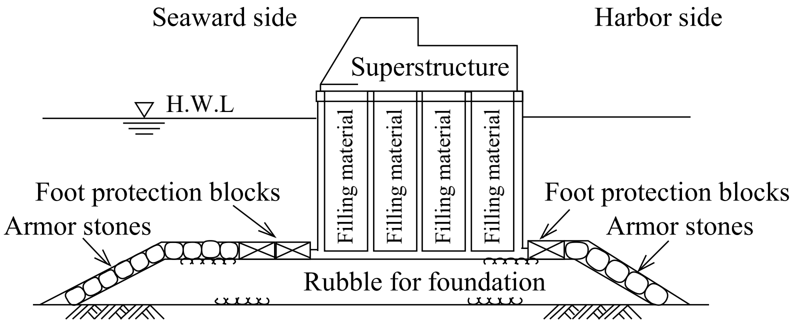


Fig. 3.7.1 Example of a Cross Section of Sloping-Top Caisson Breakwater

3.7.2 Setting of Basic Cross Sections

(1) The required dimensions of sloping-top caisson breakwaters shall be appropriately determined with consideration to their shapes. In particular, crown heights shall be appropriately determined in accordance with the transmission characteristics of structures because the transmission coefficients vary depending on structural types and wave conditions.

(2) The crown heights of sloping-top caisson breakwaters are preferably determined in consideration of harbor calmness because they increase the transmitted wave heights compared with conventional upright breakwaters.

(3) When crown heights are identical, sloping-top caisson breakwaters have wave height transmission coefficients that are approximately two times those of upright breakwaters[38)] (**Fig. 3.7.2**). Therefore, sloping-top caisson breakwaters with crown heights set to the similar level of significant wave heights $H_{1/3}$ can reduce transmitted wave heights to the same level, similar to the case for those upright breakwaters with crown heights set to 0.6 times the significant wave heights.

(4) With the increase in the gradients of sloping walls, sloping-top caisson breakwaters can be more effective in curbing the waves transmitted into harbors but receive a larger wave pressure, thereby reducing the effects of sloping breakwaters. According to the hydraulic model tests that observed the transmission coefficients by changing the gradients of sloping walls, no remarkable differences were identified in the transmission coefficients measured with the gradients of 30°, 45°, and 60°. Therefore, the gradient of sloping walls is preferably set at 45° by taking into consideration the wave pressure reduction effects and facilitation of construction work.

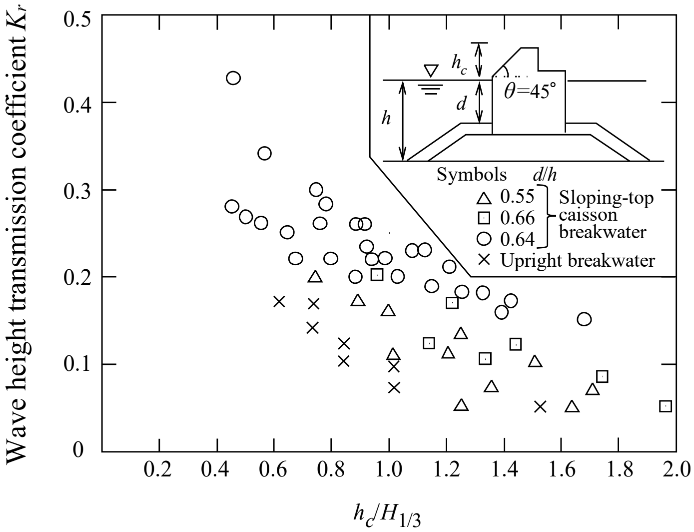


Fig. 3.7.2 Wave Height Transmission Coefficients and Relative Crown Heights

(5) When covering the upright front sections of caissons with wave-dissipating blocks, sloping-top caisson breakwaters may be subjected to impulsive breaking wave pressure depending on the crown heights of wave-dissipating blocks. Furthermore, in such a case, it is necessary to pay attention to the stability of wave-dissipating blocks because they are stacked up to still water levels.[39)]

3.7.3 Actions

(1) Although the wave force acting on sloping-top caisson breakwaters is preferably determined by hydraulic model tests, the provisions in **Part II, Chapter 2, 6.2.6 Wave Force Acting on Sloping-Top Caisson Breakwaters** can be used as a reference when the implementation of hydraulic model tests is difficult.

(2) Depending on the purpose of dissipating waves and wave conditions, the characteristics of waves used for the verification of wave-absorbing performance can be determined separately from those used for the performance verification of the stability of facilities and structural members.

(3) For the wave force acting on sloping-top caisson breakwaters covered with wave-dissipating blocks, the study result by Sato et al.[39)] can be used as a reference.

3.7.4 Performance Verification

(1) The performance verification of sloping-top caisson breakwaters should be performed by hydraulic model tests as needed with appropriate breakwater shapes selected on the basis of sufficient research on wave transmission characteristics.

(2) The verification of the stability of sloping-top caisson breakwaters can be performed in conformity with that of composite breakwaters. **Tables 3.7.1** and **3.7.2** show the standard partial factors to be used for the verification of sliding and overturning failures. For cases wherein the front faces of sloping-top caisson breakwaters are covered with wave-dissipating blocks, the standard partial factors can be selected from the values in the lower columns of the respective tables.

Table 3.7.1 Partial Factors Used for the Performance Verification of the Sliding of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Sliding of sloping-top caisson breakwater (Variable state of waves)	0.84	1.11	— (1.00)
Sliding of sloping-top caisson breakwater covered with wave-dissipating blocks (Variable state of waves)	0.76	0.95	— (1.00)

Table 3.7.2 Partial Factors Used for the Performance Verification of the Overturning of Breakwater Bodies

Verification object	Partial factor multiplied by resistance term γ_R	Partial factor multiplied by load term γ_S	Adjustment factor m
Overturning of sloping-top caisson breakwater (Variable sate of waves)	0.98	1.17	— (1.00)
Overturning of sloping-top caisson breakwater covered with wave-dissipating blocks (Variable state of waves)	0.98	1.06	— (1.00)

(3) The partial factors above have been set with reference to the safety levels in the past standards.[40)] Furthermore, the partial factors are set under the conditions that the topographies of seabed where breakwaters are installed have gradients of less than 1/30. In cases of the topographies of seabed with gradients larger than 1/30, partial factors shall be appropriately set with reference to the descriptions in **Reference 40)**.

3.7.5 Performance Verification of Structural Members

The performance verification of structural members can be performed with reference to **Part III, Chapter 2, 2 Structural Members**.

3.8 Pile-Type Breakwaters

[Public Notice] (Performance Criteria of Pile-Type Breakwaters)

Article 36

The performance criteria for pile-type breakwaters under variable situations, in which the dominating actions are variable waves and Level 1 earthquake ground motion are prescribed in the following items:

(1) The risk that the axial force acting on the piles may exceed the resistance based on the failure of the ground shall be equal to or less than the threshold level.

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

[Interpretation]

10. Protective Facilities for Harbors

(4) **Performance Criteria of Pile-Type Breakwaters** (Article 14, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 36 of the Public Notice)

① The required performance of pile-type breakwaters under the variable action situation in which the dominating actions are variable waves and Level 1 earthquake ground motions shall focus on Serviceability. **Attached Table 10-5** shows the performance verification items and standard indexes for determining the limit values with respect to the actions.

Attached Table 10-5 Performance Verification Items and Standard Indexes for Determining the Limit Values of Pile-Type Breakwaters

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		
14	1	2	36	-	1	Serviceability	Variable	Variable waves [Level 1 earthquake ground motion]	Self-weight, water pressure	Axial force acting on the pile	Action–resistance ratio with respect to the bearing capacity of the pile (Pushing and pulling out)
					2					Yield stress of the pile	Design yield stress

* [ ] means alternative dominating action to be studied as design situations.

② In addition to the requirements above, the superstructures and curtains of pile-type breakwaters shall conform to the requirements and commentaries in Articles 23 to 27 of the Public Notice depending on the types of members constituting the pile-type breakwaters.

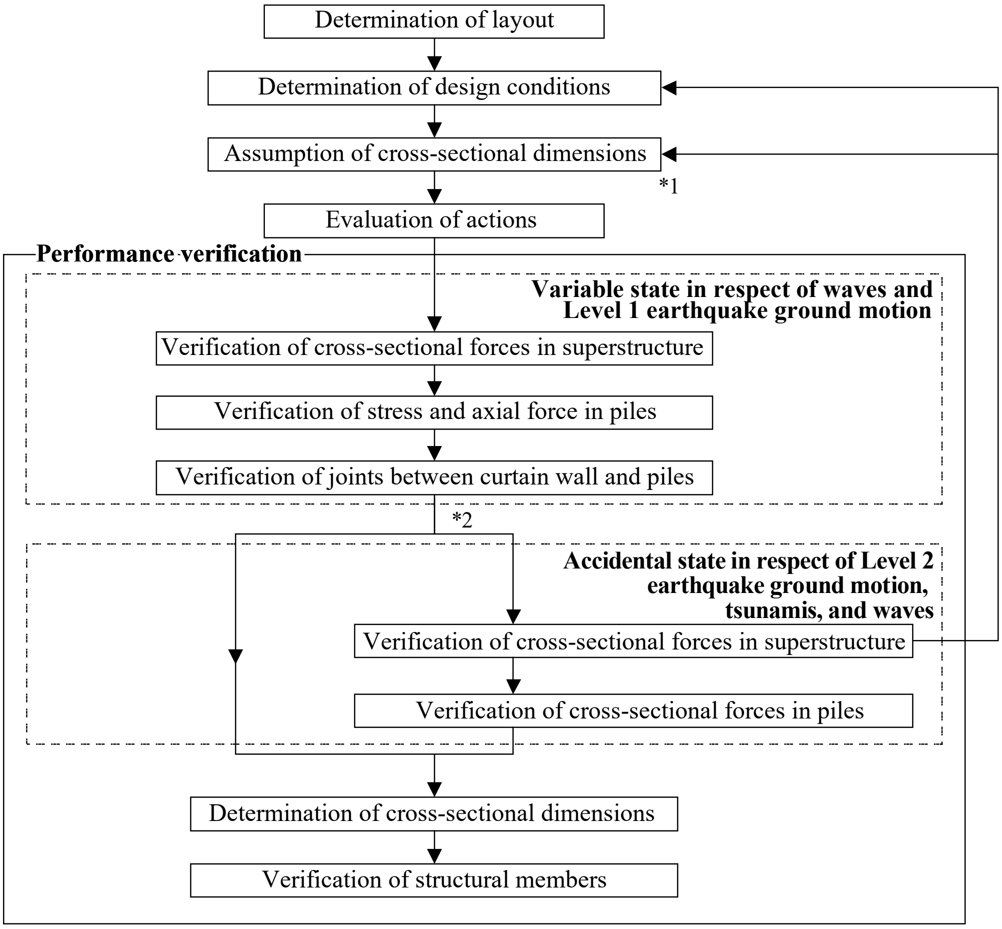
3.8.1 General

(1) Pile-type breakwaters can be broadly divided into curtain wall breakwaters and steel pipe breakwaters. Curtain wall breakwaters are permeable breakwaters of pile structure developed for use in water areas with comparatively low wave heights, such as enclosed bays, or locations with soft sea bottom ground, whereas steel pipe breakwaters are breakwaters that stop waves only by using steel pipe piles.

(2) The performance verification of curtain wall breakwaters should be performed on the basis of hydraulic model tests as needed, with their structures appropriately selected with consideration to reflection and transmission coefficients.

(3) **Fig. 3.8.1** shows an example of the performance verification procedure for curtain wall breakwaters. However, because **Fig. 3.8.1** does not show the evaluation of the effects of liquefaction due to earthquake ground motions, it

is necessary to appropriately deliberate the possibility of and the measures against liquefaction by referring to **Part II, Chapter 7 Liquefaction of Ground**.



*1: Because assessment of the effects of liquefaction is not shown, separate consideration is necessary.
*2: For facilities where damage to the facilities can be assumed to have a serious impact on life, property, and social activity, it is preferable to conduct verification for accidental situations when necessary. Verification for accidental situations in respect of waves shall be conducted in cases where facilities handling hazardous cargoes are located directly behind the breakwater and damage to the objective facilities would have a catastrophic impact.

Fig. 3.8.1 Example of the Performance Verification Procedure for Pile-Type Breakwaters

(4) Curtain wall breakwaters can be broadly divided into single-curtain-wall breakwaters and double-curtain-wall breakwaters depending on how the so-called curtain walls, such as concrete plates, are arranged relative to the directions of wave propagation. Furthermore, a variety of derived types can be conceived depending on the shapes of the pile structures supporting the curtain walls or the shapes of the slits provided in the curtain walls. **Fig. 3.8.2** shows examples of the cross sections of pile-type breakwaters.

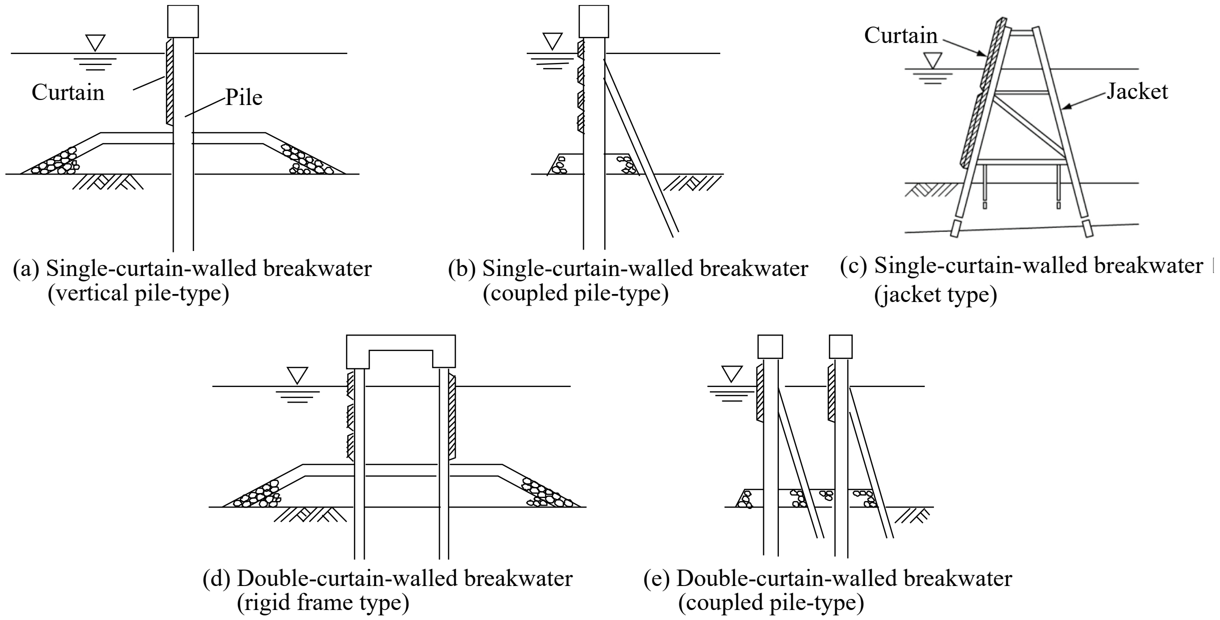


Fig. 3.8.2 Examples of Cross Sections of Pile-Type Breakwaters

(5) Curtain wall breakwaters generally have the following features:

① The reflection coefficients can be reduced to the level equal to or less than those of breakwaters covered with wave-dissipating blocks.

② The exchange of sea water can be expected by tides and waves passing through the slits provided in the curtain walls or the gaps between the lower edges of the curtain walls and the seabed.

③ The construction work such as pile driving, curtain fixing bracket installation, and curtain installation work, needs to be implemented with a certain level of accuracy.

④ Given the expected wave energy attenuation effect between front and back curtain walls, double-curtain-wall breakwaters can reduce reflected and transmitted waves more effectively than single-curtain-wall breakwater.

⑤ Given that the velocities of flows passing under the curtain walls are quite high, it is necessary to take appropriate countermeasures to prevent or suppress the washing out of sand.

(6) Steel pipe breakwaters are breakwaters that use steel pipe piles or steel pipe sheet piles. Steel pipe breakwaters have structures that are lighter than gravity-type breakwaters. Therefore, steel pipe breakwaters are suitable for locations with soft ground and relatively low wave heights.

(7) The performance verification of steel pipe breakwaters can be performed with reference to the concepts of curtain wall breakwaters.

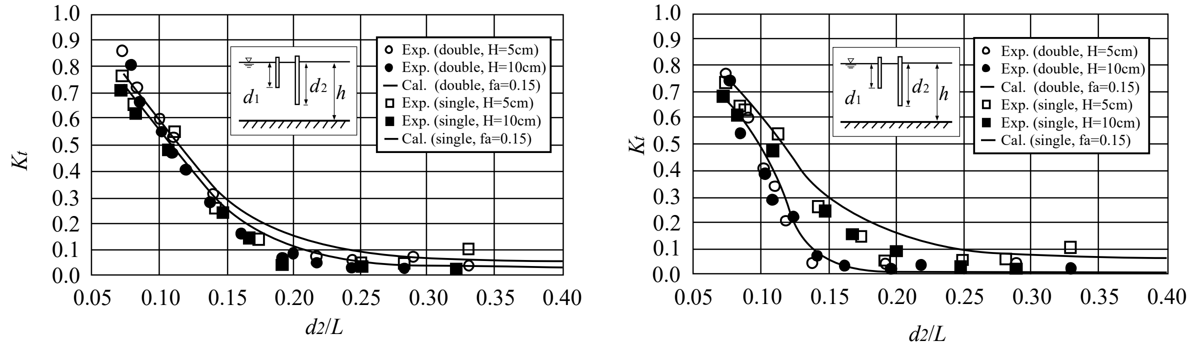
3.8.2 Setting of the Basic Cross Sections

(1) The structural types and shapes of curtain wall breakwaters shall be determined by considering the hydrographic conditions of object water areas, the target reflection and transmission coefficients, and workability.

(2) The cross sections of curtain wall breakwaters, including the crown heights, depths of the lower ends of curtains, sizes of the slits provided in the curtains, and spacing between curtain walls in the cases of double-curtain-wall breakwaters, are preferably set on the basis of model tests that simulate actual conditions. It is preferable that the dimensions of members such as curtains and piles be determined appropriately by considering the spacing between the piles in the face line directions of breakwaters.

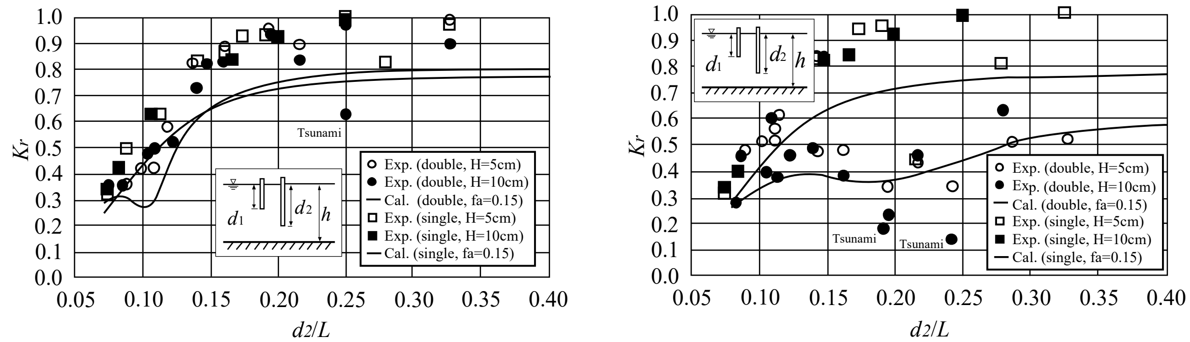
(3) Studies have been conducted on the obtainment of the reflection and transmission coefficients of curtain wall breakwaters by using model tests or numerical analyses. For example, as shown in **Fig. 3.8.3**, Nakamura et al.[41] conducted a model test to observe the reflection and transmission coefficients of a single-curtain-wall breakwater and a double-curtain-wall breakwater with the curtain spacing similar to the width of gravity-type upright

breakwaters by changing drafts and showed that the observation results of the reflection and transmission coefficients agreed well with the wave attenuation theory. Furthermore, Kyono et al.[42] conducted model tests and a numerical analysis based on the Volume of Fluid Method to deliberate the wave pressure distribution and wave force characteristics with respect to curtain wall breakwaters.



d_1/d_2=0.218, d_2/h=0.353 d_1/d_2=1.000, d_2/h=0.353

(a) Comparison of transmission coefficients between double-curtain-wall and single-curtain-wall breakwaters



d_1/d_2=0.218, d_2/h=0.353 d_1/d_2=1.000, d_2/h=0.353

(b) Comparison of reflection coefficients between double-curtain-wall and single-curtain-wall breakwaters

Fig. 3.8.3 Reflection and Transmission Coefficients of Curtain Wall Breakwaters

(4) Examples of model tests for single-curtain-wall breakwaters include the model tests by Morihira et al.[43] According to the model test result, the depths of the lower ends and the crown heights of curtain walls can be obtained in relation to the wave height transmission coefficients by using **Fig. 3.8.4** and **Fig. 3.8.5**, respectively, provided that the crown heights of the curtain walls in **Fig. 3.8.5** are corrected so that *R/H* = 1.25 at *d/h* = 1.0, that is, the crown heights are not those that are capable of completely preventing overtopping waves. In the figure, *d* is the depth of the lower end of a curtain, *h* is the water depth, *L* is the wave length, *R* is the crown height of a curtain, and *H* is the wave height. **Fig. 3.8.6** shows the relationship between wave reflection coefficients and d/h in single-curtain-wall breakwaters.

(5) With steel pipe piles driven at intervals, steel pipe breakwaters can be used as permeable-type breakwaters. Hayashi et al.[44] studied the relationship between the ratios of pile intervals to pile diameters *b/D* and wave transmission coefficients γ_T (**Fig. 3.8.7**).

Furthermore, the moment in piles due to waves decreases as the space between piles increases. However, this moment reduction effect reaches its limits at approximately *d/D* = 0.1. Note that the breakwaters of this type undergo the scouring of the ground between piles.

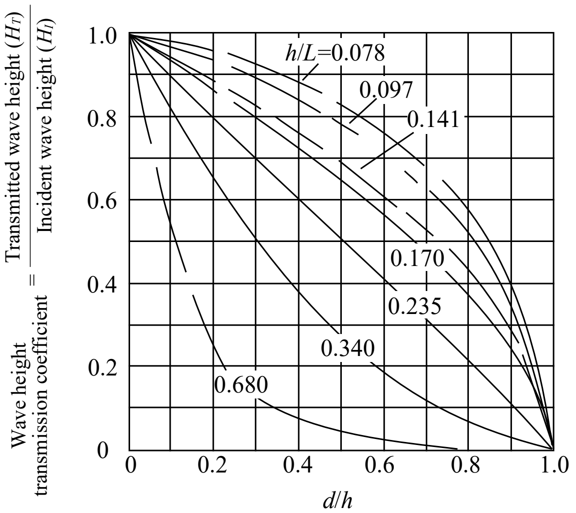


Fig. 3.8.4 Relationship between *d/h* and Wave Transmission Coefficients (Single Curtain Wall)

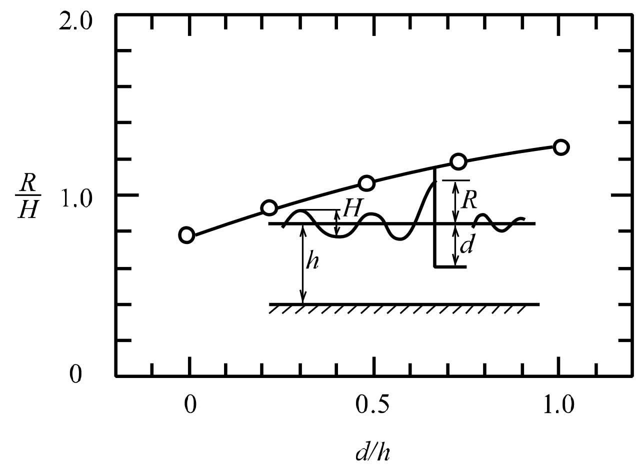


Fig. 3.8.5 Crown Height Calculation Curve (Single Curtain Wall)

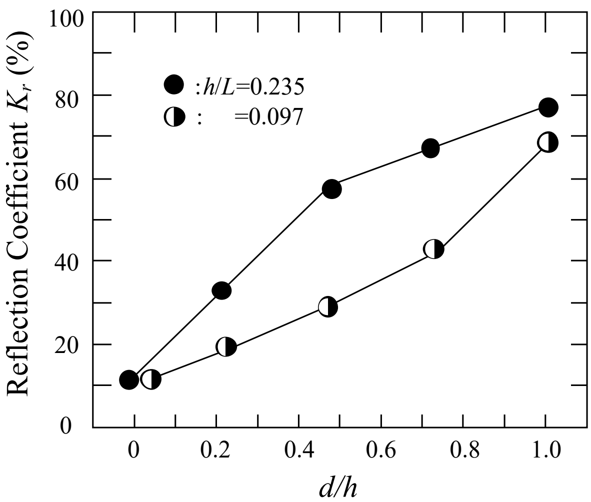


Fig. 3.8.6 Relationship between *d/h* and Wave Reflection Coefficients (Single Curtain Wall)

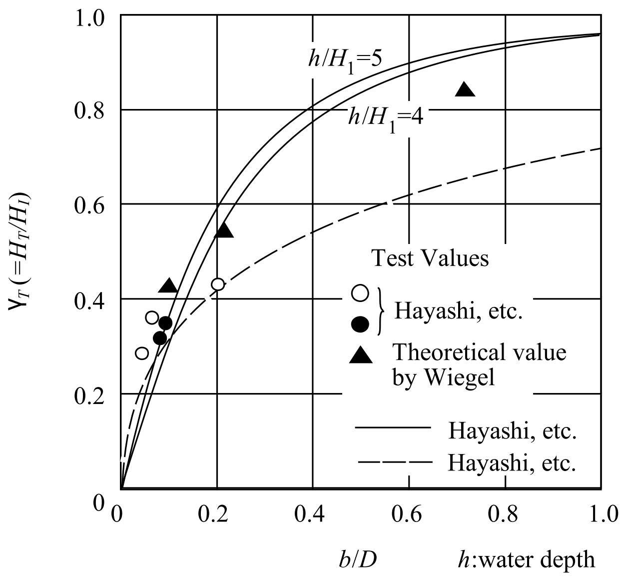


Fig. 3.8.7 Relationship between the Ratio of Pile Interval to Pile Diameter and Wave Transmission Coefficient[43)]

3.8.3 Actions

(1) The actions to be considered in the performance verification of pile-type breakwaters can be set in conformity with composite breakwaters with reference to **Part II, Chapter 4, 3.1.3 Actions**. However, because the reflection and transmission coefficients of pile-type breakwaters vary depending on wave steepness, the deliberation of the reflection and transmission coefficients can be based on the types of waves that have relatively high frequencies and possible risks of interference with the use of ports in general.

(2) Considering that pile-type breakwaters have structures that dissipate waves with slits in curtain sections or the chambers between front and rear curtain sections, the wave force acting on these curtain sections vary depending on their shapes and the characteristics of incoming waves.

(3) The wave force acting on curtain wall breakwaters shall be set on the basis of the results of hydraulic model tests, numerical analyses, or appropriate calculation equations. In the case of single curtain walls, the wave force acting on them can be calculated by subtracting the wave pressure distributed in the sections below the lower edges of curtains from the wave pressure distribution shown in **Part II, Chapter 2, 6 Wave Force**.

(4) Studies on wave force acting on steel pipe breakwaters include the studies conducted by Hayashi et al.[44)] and Nagai et al.[45)] According to their test results, it is considered that **Part II, Chapter 2, 6.3 Wave Force Acting on Submerged Members and Independent Structures** can be used as a reference when setting the wave force acting on steel pipe breakwaters. However, it is preferable not to use steel pipe breakwaters in water areas that are subjected to breaking waves.

(5) Although the performance verification of pile-type breakwaters with respect to Level 1 earthquake ground motions can be performed in a manner that directly calculates the deformation amounts via detailed analysis methods such as dynamic analyses and evaluates whether the required performance is ensured, simplified methods, such as the seismic coefficient method, can also be used for performance verification. When using simplified methods, the characteristic values of the seismic coefficients to be used in the performance verification shall be appropriately set by taking into consideration the structural characteristics of pile-type breakwaters. Depending on the structural types, the characteristic values of the seismic coefficients for pile-type breakwaters can be calculated in conformity with **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles** for convenience.

3.8.4 Performance Verification

(1) For the verification of the stresses in steel pipe piles of pile-type breakwaters, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles**. The adjustment factors to be used in the performance verification shall be appropriately set with reference to the allowable stresses on the basis of past design methods.

(2) In the performance verification of stresses in piles, flexural moment and shear force can be calculated on the basis of hinged head piles in the case of pile structures comprising vertical piles or rigid frames having fixed head piles

with virtual fixed points $1/\beta$ below seafloor surfaces in the case of pile structures comprising a group of piles or front and rear piles with respective pile heads rigidly connected through superstructures. For the calculation of β, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles**.

(3) Respective piles shall have sufficient embedded lengths that ensure lateral resistance when subjected to wave actions and bearing capacity against pushing and pulling force. The embedded lengths of pile-type breakwaters can be calculated with reference to **Part III, Chapter 2, 3.4 Pile Foundations**.

3.8.5 Performance Verification of Structural Members

(1) For the performance verification of concrete curtain members, refer to **Part III, Chapter 2, 2 Structural Members**.

(2) For the performance verification of the superstructures of breakwaters, refer to **Part III, Chapter 5, 5.2 Open-Type Wharves on Vertical Piles**.

3.8.6 Structural Details

The structural details of pile-type breakwaters can be set in conformity with composite breakwaters with reference to **Chapter 4, 3.1.9 Structural Details** or the structural details of other similar structural types.

3.9 Breakwaters with Wide Footings on Soft Ground

[Interpretation]

10. Protective Facilities of Harbors

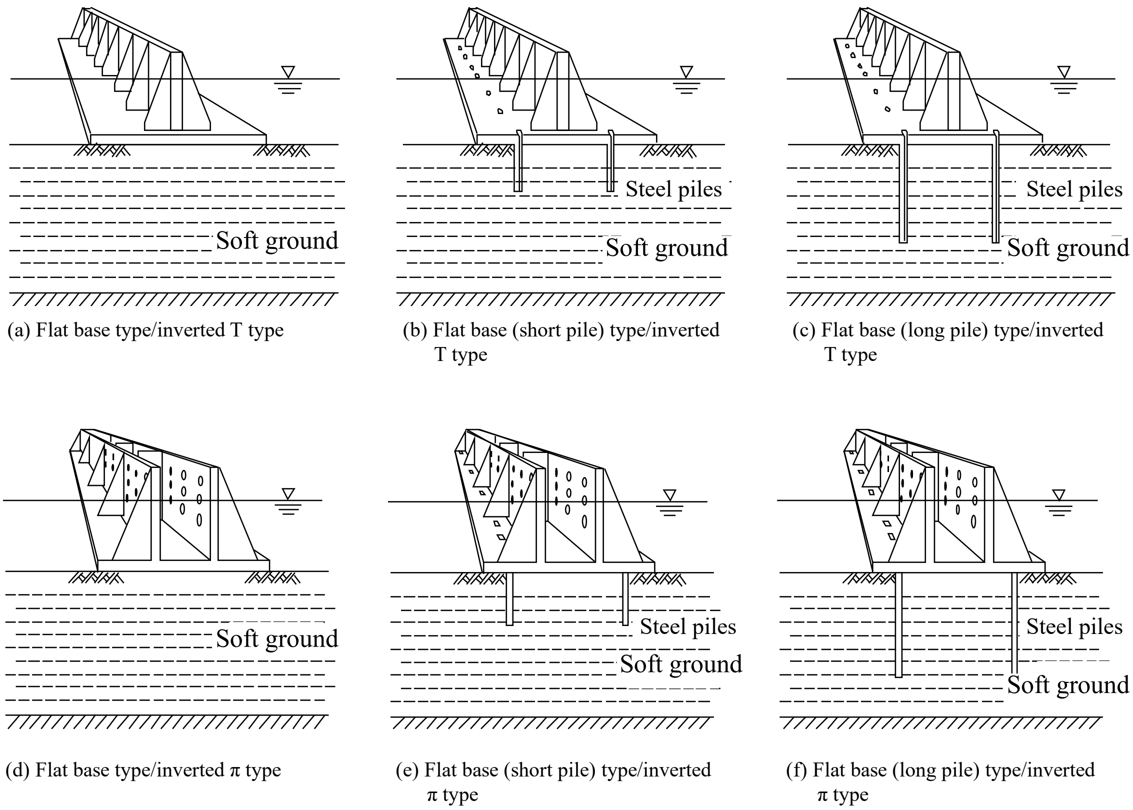
(4) Performance Criteria of Pile-Type Breakwaters (Item 2, Paragraph 1, Article 14 of the Ministerial Ordinance and the Interpretation related to Article 36 of the Public Notice)

③ Because breakwaters with wide footings on soft ground with pile foundations are a structural type which has the respective structural features of the gravity-type breakwater and the pile-type breakwater, the performance criteria for breakwaters with wide footings on soft ground are equivalent to the respective settings in the Public Notice, Article 35 Performance Criteria for Gravity-type Breakwaters and Article 36 Performance Criteria for Pile-type Breakwaters.

3.9.1 Fundamentals of Performance Verification

(1) Breakwaters with wide footings on soft ground (hereafter, soft landing breakwaters) resist against the horizontal wave force by the piles and the cohesion between the bottoms of the breakwater bodies and the surface layers of cohesive soil. On the other hand, the bottom slabs and footings resist against the vertical force. In general, because this type of structure is developed for construction of breakwaters on soft cohesive soil, there are cases where this type is economically advantageous because the weights of the breakwater bodies can be reduced and soil improvement is not required.

(2) **Fig. 3.9.1** shows examples of cross sections of soft landing breakwaters. Although structural types can be broadly divided into the "flat base type" and the "flat base type with piles," the flat base type with piles is generally used.



(a) Flat base type/inverted T type

(b) Flat base (short pile) type/inverted T type

(c) Flat base (long pile) type/inverted T type

(d) Flat base type/inverted π type

(e) Flat base (short pile) type/inverted π type

(f) Flat base (long pile) type/inverted π type

Fig. 3.9.1 Examples of Cross Sections of Wide Footing Breakwaters on Soft Ground

(3) Wide footing breakwaters on soft ground cannot achieve the minimization of both pile dimensions and the widths of breakwater bodies. Therefore, appropriate cross sections shall be selected after the comparative studies of the combinations of pile dimensions and the widths of breakwater bodies.

(4) Constructed directly on soft ground, wide footing breakwaters on soft ground are affected by scouring by waves and water currents in the areas around the breakwater bodies. Therefore, appropriate countermeasures shall be taken, as necessary.

3.9.2 Actions

(1) Actions shall be appropriately set considering natural, use and construction conditions as well as water quality environment. For the types of actions to be considered in the performance verification, reference can be made to **Part III, Chapter 4, 3.1.3 Actions** in accordance with the type of composite breakwaters.

(2) While the performance verification of this type of breakwaters regarding Level 1 earthquake ground motions can be carried out through can be carried out through detailed analysis methods such as dynamic analyses which directly calculate deformation amounts and evaluate performance, simplified methods such as the seismic coefficient method can also be used for the performance verification. When using the simplified methods, the characteristic values of the seismic coefficients to be used in the performance verification can be calculated by the following **equation (3.9.1)** using the maximum values of the time history of acceleration at the bottoms of breakwater bodies obtained through the one-dimensional earthquake response analyses with Level 1 earthquake ground motions in engineering bedrock as input earthquake ground motions. For the calculation of the time history of acceleration of the bottoms of breakwater bodies, reference can be made to **Reference (Part III), Chapter 1, 1.2.2, Group 2 Procedures for the calculation of seismic coefficients for verification (2)** and **(3)**.

$$k_h = \frac{\alpha_{\max}}{g} \tag{3.9.1}$$

where

k_h : a seismic coefficient for verification;

$\alpha_{\max}$: the maximum value of acceleration at the bottom of a breakwater body (cm/s^2); and

g : gravitational acceleration (cm/s^2)

3.9.3 Performance Verification

The performance verification of soft landing breakwaters can be carried out with reference to **References 46)** to **49)**.

3.10 Floating Breakwaters

[Public Notice] (Performance Criteria of Floating Breakwaters)

Article 37

The performance criteria of floating breakwaters under the variable situation, in which the dominating action is variable waves, shall be as prescribed respectively in the following items:

(1) The risk of capsizing of the floating body shall be equal to or less than the threshold level.

(2) The risk of impairing the integrity of the members of the floating body shall be equal to or less than the threshold level.

(3) The risk that the stress generated in mooring ropes may exceed the yield stress shall be equal to or less than the threshold level.

(4) The risk of losing the stability due to tractive forces acting on mooring anchors shall be equal to or less than the threshold level.

[Interpretation]

10. Protective Facilities for Harbors

(5) Performance Criteria of Floating Breakwaters (Article 14, Paragraph 1, Item 2 of the Ministerial Ordinance and the interpretation related to Article 37 of the Public Notice)

① Serviceability shall be the performance requirement of floating breakwaters under the variable situation in which the dominating action is variable waves. The performance verification items and standard indexes to determine the limit values regarding the actions shall be as shown in **Attached Table 10-6**.

Attached Table 10-6 Performance Verification Items and Standard Indexes to Determine Limit Values of Floating Breakwaters

Ministerial Ordinance			Public Notice			Performance requirement	Design state			Verification item	Standard index to determine limit value
Article	Paragraph	Item	Article	Paragraph	Item		State	Dominating action	Non–dominating action		
14	1	2	37	1	1	Serviceability	Variable	Variable waves	Self-weight, wind, water pressure, water currents	Capsizing of floating body	-
					2					Integrity of members	-
					3					Yielding of mooring ropes	Design yield stress
					4					Stability of mooring anchors, etc.	Resistance force of mooring anchors, etc. (horizontal, vertical)

② In **Attached Table 10-6**, the standard indexes to determine limit values shall be appropriately set for the performance verification of the capsizing of floating bodies and the integrity of members.

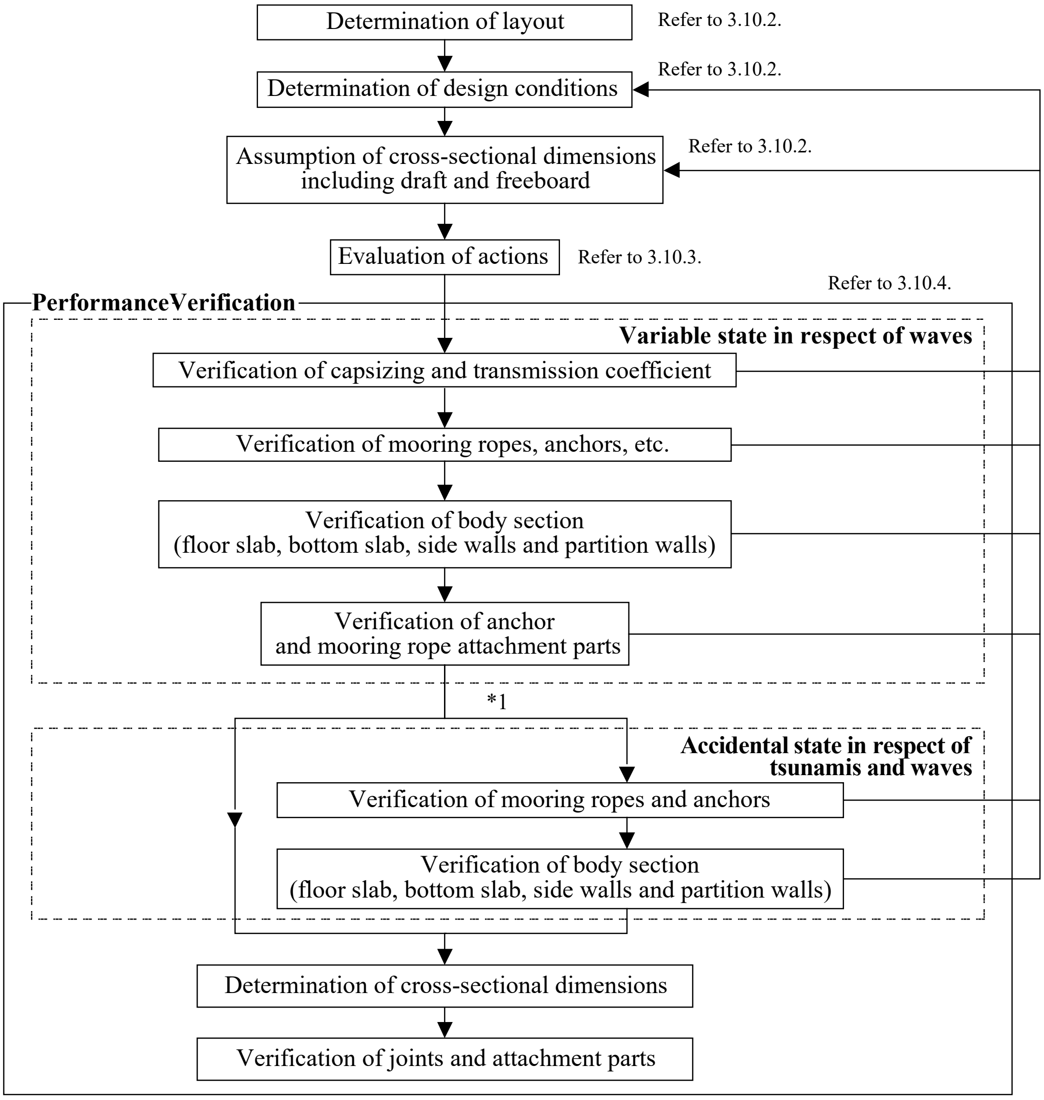
③ Mooring anchor is a collective term for equipment placed on the surface of the sea bottom to retain the floating body. Concretely, in addition to the mooring anchors, sinkers are also included.

3.10.1 Fundamentals of Performance Verification

(1) Floating breakwaters are breakwaters in which transmitted waves are reduced by moored floating bodies. Although the shapes of the floating bodies include many types, the pontoon type is widely used.

(2) It is preferable to carry out the performance verification of floating breakwaters on the basis of hydraulic model tests and theoretical analyses, as needed, with their structures appropriately selected in consideration of wave dissipating effects and stability.

(3) **Fig. 3.10.1** shows an example of the performance verification procedure for floating breakwaters. However, because **Fig. 3.10.1** does not show the evaluation of the effects of liquefaction due to earthquake ground motions on mooring anchors, it is necessary to appropriately deliberate the possibility of and the measures against liquefaction with reference to **Part II, Chapter 7 Liquefaction of Ground**.



*1: For facilities where damage to the facilities can be assumed to have a serious impact on life, property, and social activity, it is preferable to conduct verification for accidental situations when necessary. Verification for accidental situation in respect of waves shall be conducted in cases where facilities handling hazardous cargoes are located directly behind the breakwater and damage to the objective facilities would have a catastrophic impact.

Fig. 3.10.1 Example of Performance Verification Procedure for Floating Breakwaters

(4) The floating breakwaters have various advantages, including the fact that they do not prevent movement of sea water and littoral drift, they are not affected by tidal level changes or ground conditions, and they are moveable. However, they also have some problems in that they generate transmitted waves, their effects differ remarkably depending on the characteristics of waves, they can only be used in locations with small waves due to their limited wave resistance, and the mechanism of the anchor systems' resisting against repeated impulsive actions has not been understood fully. Furthermore, because there is a danger of secondary damage due to drifting of floating

bodies if the mooring ropes break, appropriate measures should be taken including the arrangement of mooring ropes with due considerations to safety.

(5) When determining the quality of materials used for the structures of floating breakwaters, due consideration must be paid to the characteristics, durability, and economic performance of the materials.

3.10.2 Setting of Basic Cross Sections

(1) The layouts and shapes of floating breakwaters shall be set to ensure predetermined harbor calmness. When setting the layouts and shapes, it is preferable to measure transmission coefficients of floating breakwaters through hydraulic model tests. The theoretical analysis methods which can be used as references include the approximate computation method for the motions of two-dimensional rectangular floating bodies by Ito et al.[50)] and the theory pertaining to unmoored free floating bodies by Ijima.[51)]

(2) The shapes of floating breakwaters range widely and the materials used for them include reinforced concrete, pre-stressed concrete, steel, etc. The layout patterns of floating breakwaters are largely divided into serial and alternating parallel patterns.[52)] **Fig. 3.10.2** shows examples of the layouts of floating breakwaters. Floating breakwaters are generally moored with mooring anchors made of steel or concrete and mooring ropes consisting of chains or synthetic fiber ropes.

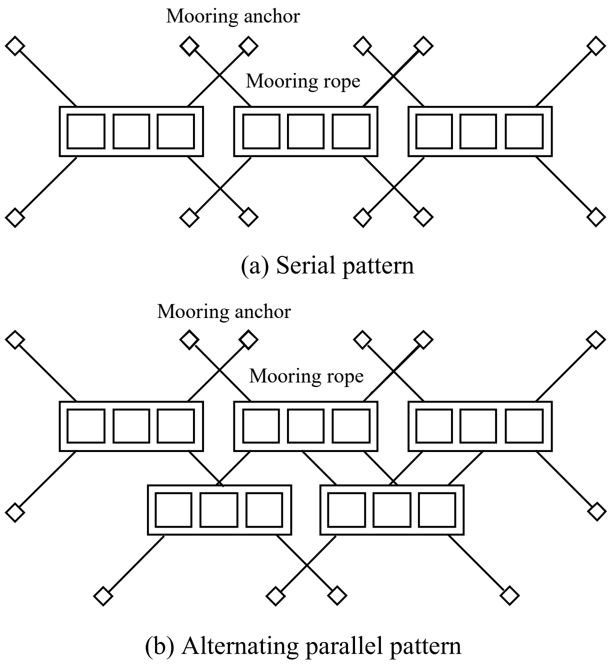


Fig. 3.10.2 Examples of the Layouts of Floating Breakwaters

(3) Mooring methods shall be selected with due consideration given to the actions (waves, currents, etc.) on floating bodies, water depths, tidal levels, seafloor topography, sea bottom soil property, and the lengths of mooring ropes.

3.10.3 Actions

For the actions on floating breakwaters, reference shall be made to **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions**.

3.10.4 Performance Verification

(1) The performance verification of floating breakwaters shall be carried out with reference to **Part II, Chapter 2, 4.8 Actions on Floating Body and its Motions** and **Part III, Chapter 5, 6 Floating Piers**. Also, reference can be made to the **Manual for Design and Construction of Floating Breakwaters (Draft)**.[53)]

(2) The performance verification of the stability of floating bodies shall be carried out with reference to **Part III, Chapter 5, 6.4 Performance Verification** in compliance with the provisions for floating piers, however, that floating breakwaters are not subjected to the performance verification of the stability in accordance with their use conditions. Also, **Reference 54)** can be used as a reference for the alternative idea of studying the stability of floating bodies when inundated.

(3) The performance verification shall also be carried out for the stability of floating bodies when towed using counter ballast during construction.

(4) The performance verification of the mooring systems comprising mooring ropes and anchors can be carried out in the following manner: obtain tensile and other forces acting on mooring ropes and anchors through static and/or dynamic analyses by assuming various conditions concerning mooring systems such as mooring methods, the lengths of mooring ropes, etc. and confirm the stability of the mooring systems by carrying out the performance verification using the obtained tensile and other forces.

(5) The dynamic analyses of mooring ropes are generally to obtain variable tensions and variable displacements generated by the motions of floating bodies and largely divided into two types: one using static mooring characteristics[55)] and the other using dynamic response characteristics of mooring ropes.[56)]

(6) The performance verification of mooring anchors can be carried out in compliance with the provisions for floating piers in **Part III, Chapter 5, 6.4 Performance Verification** and with **Reference 57)**.

(7) The structures of the floating bodies of floating breakwaters shall sufficiently ensure overall safety and local strength. For those structures like floating breakwaters which have substantially large lengths compared to the widths and depths, it is generally preferable to examine the following points (see **Fig. 3.10.3)**.

Longitudinal strength:
Sectional force (longitudinal flexural moment, shear force and torsional moment) on entire floating bodies on static water or when subjected to wave actions

Lateral strength:
Sectional force (flexural moment and shear force) in the directions perpendicular to face lines on entire floating bodies when subjected to wave actions

Local strength:
Sectional force (flexural moment and shear force) on respective members such as wall and beam materials

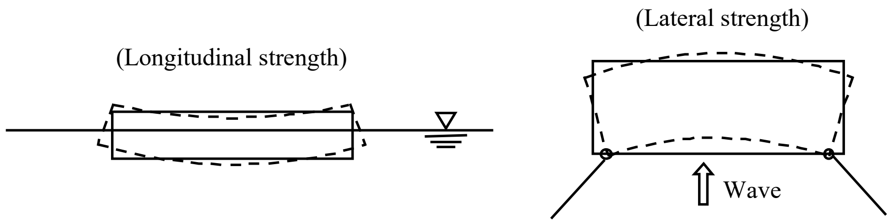


Fig. 3.10.3 Images of the Longitudinal and Lateral Strength of Floating Structure

(8) There are two types of longitudinal strength calculation methods depending on whether or not the motions of floating bodies are considered. Muller's equation is one of the popular methods which does not consider the motions of floating bodies. In contrast, Ueda's formula[58)] is an example of a method which considers the motions of floating bodies. **Reference 58)** introduces the comparisons of the calculation results of the two types of methods which can be used as a reference when selecting the methods.

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4 Green Breakwaters

Green breakwaters shall comply with the requirements of **Part III, Chapter4, 3 Breakwaters with Basic Functions** and may be checked for performance verification as follows:

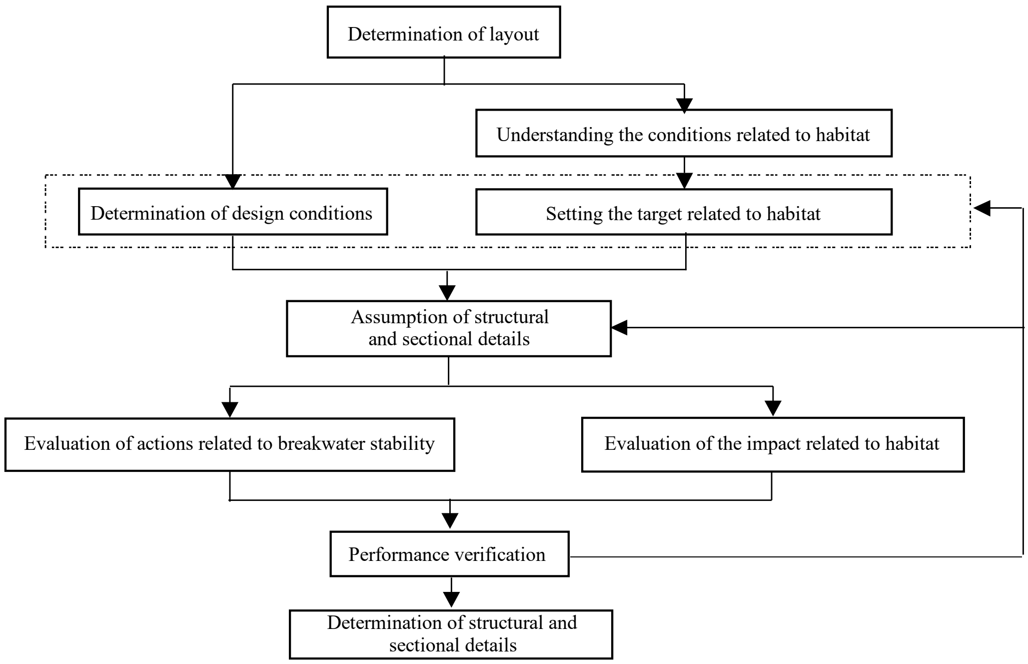
(1) Breakwaters contributing to the development of a good environment in port include green breakwaters[1)] that are intended to allow inhabitation of marine organisms in tidal flats or shore reefs depending on the natural environment where the facilities are located (**Reference (Part I), Chapter 3, 2 Green Port Structures**). Existing breakwaters may be remodeled into green ones by attaching the habitat function during their improvement.

(2) The influence on the targets related to inhabitation of marine organisms (**Reference (Part I), Chapter 3, 2 Green Port Structures**) should be determined via environmental surveys and numerical models. The performance verification of the green breakwater is conducted by checking whether the structure and cross section or the ancillary facility is logically expected to achieve the target.

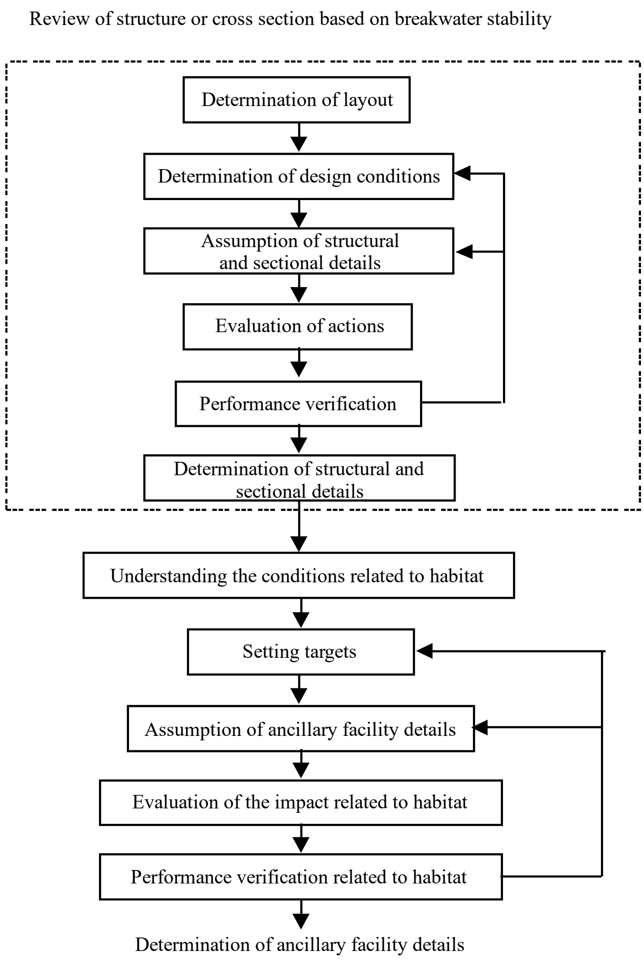
(3) Performance requirement of a green breakwater is that the breakwater should have a habitat function, and its impacts include the presence or absence of the ground related to the inhabitation of marine organisms, external forces such as waves or currents, and the environment necessary for the inhabitation of organisms. The environment necessary for the inhabitation of marine organisms includes, for example, the water depth and water transparency that affects the light intensity necessary for photosynthesis and the water temperature that affects marine organism activity. In particular, when aiming at the cultivation of a seaweed bed, the structure and cross section of a breakwater and the texture and gradient of the ancillary facility need to allow rooting of the target seaweed or seagrass. In addition, it is necessary that the sunlight shielding by them does not affect the light intensity required for the growth of seaweed or seagrass.

(4) Performance verification of a green breakwater should be conducted by ensuring based on the known finding that the environment of a location where one intends to allow marine organisms to inhabit is within the inhabitable range of the target marine organism. For example, during the performance verification of a breakwater used to grow a seaweed bed, the light intensity and water temperature that affect photosynthesis and respiration are assumed to be the environment that should be considered; such verification should be conducted by ensuring that these environmental conditions are within the range wherein a seaweed bed can be established. The location wherein changes in the environmental conditions after a green breakwater is constructed or its future environmental changes can be estimated, a verification method that uses a numerical model related to growth to ensure the environment is within the inhabitable range of marine organisms may be considered.

(5) The procedure for reviewing a green breakwater varies depending on whether the habitat function is provided to the structure and cross section of the breakwater or to its ancillary facility. An example of the review procedure is shown in **Fig. 4.1.1**.



(a) When the habitat function is provided to the structure and cross section of the breakwater



(b) When the habitat function is provided to the ancillary facility

Fig. 4.1.1 Example of a Review Procedure of Green Breakwater

(6) During the performance verification of a green breakwater, **Reference (Part I), Chapter 3, 2 Green Port Structures** and the **Guideline for Development and Maintenance of Green Port Structures**[1)] may be used as reference.

(7) It may be feasible to develop breakwaters that can consider the environment with these synergetic effects by adding the amenity function to the symbiotic function.

[Reference]

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5 Amenity-oriented Breakwaters

Amenity-oriented breakwaters shall comply with **Part III, Chapter 4, 3 Breakwaters Having Basic Functions** with necessary modifications made in consideration of the structural type, and their performance verification may be performed as follows:

(1) For the performance verification of amenity-oriented breakwaters, refer to the **Technical Manual for the Improvement of Port Environment**[1].

(2) Breakwaters may be provided with amenity-oriented functions, such as fishing facilities, so that they can be used for multiple purposes.[2]

(3) Amenity-oriented breakwaters shall be equipped with fall prevention fences and other ancillary facilities as needed to prevent users from falling into the sea.

(4) The crown height of an amenity-oriented breakwater must be examined from the viewpoint of public use and safety considering splash, the extent of wave overtopping, and other factors.[3]

(5) Walkways and slopes of breakwaters shall have widths, pitches, and other dimensions that allow elderly users and physically disabled users, including those in wheelchairs, to move safely.[4] [5] [6]

(6) Amenity-oriented functions can be enhanced by considering the inhabitation of marine organisms (**Reference (Part I), Chapter 3, 2 Green Port Structures**).

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6 Storm Surge Protection Breakwaters

6.1 General

(1) The layout, crown height, and other structural details of a storm surge protection breakwater shall be appropriately set considering its effectiveness in reducing the impact of storm surges.

(2) The stability of a storm surge protection breakwater shall be secured not only against the action of waves but also against storm surges considering a rise in the water level and other characteristics of the port or harbor when it is attacked by storm surges.

(3) Storm surge protection breakwaters shall be designed in accordance with this document and by referring to the **Technical Standards and Commentary for Shore Protection Facilities**[1)], **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors**[2)], **Design Concept for Parapets against Tsunamis (Provisional Edition)**[3)], **Guidelines for Tsunami-Resistant Design of Breakwaters**[4)], **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores**[5)], and **JSCE Design Handbook for Shore Protection Facilities**[6)].

6.2 Setting of Basic Cross Section

(1) The basic cross section of a storm surge protection breakwater shall conform to **Part III, Chapter 4, 3 Breakwaters Having Basic Functions** with modifications made as necessary in consideration of the structural type.

(2) The crown height required for a storm surge protection breakwater shall be determined by giving appropriate consideration to waves and the tidal level in the place where it will be constructed. For waves and the tidal level, refer to **Part II, Chapter 2, 4 Waves** and **Part II, Chapter 2, 3 Tidal Level**, respectively.

6.3 Actions and Performance Verification

(1) For the performance verification of a storm surge protection breakwater, the design high water level shall be set appropriately by considering the largest possible storm surges. For setting the design high water level, refer to **Part II, Chapter 2, 3 Tidal Level** and **Part II, Chapter 2, 4 Waves**.

(2) For the performance verification of a storm surge protection breakwater, the rise in the water level shall be considered due to the entry of storm surges into the port or harbor as well as waves that simultaneously occur with storm surges. For setting design conditions and verifying performance in consideration of the simultaneous occurrence of storm surges and waves, refer to **Part II, Chapter 2, 3 Tidal Level** and **Part II, Chapter 2, 4 Waves**.

6.4 Structural Details

(1) Note that when the foundation of a storm surge protection breakwater has high permeability, water flows through the foundation and this decreases the effectiveness of the breakwater in reducing the rise of the water level behind the breakwater. Storm surge protection breakwaters should be provided with water sealing work as needed.

(2) There are cases wherein water flows through the rubble mound of a breakwater because of a difference in the water level between the inside and outside of the breakwater and results in scouring of the foundation ground. In these cases, small-sized rubble, scouring protection mats, or other materials should be laid as needed. For details regarding seepage flows through rubble mound, refer to **Part II, Chapter 3, 3 Underground Water Level and Seepage**.

[References]

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7 Tsunami Protection Breakwaters

7.1 General

(1) Tsunami protection breakwaters are mainly located at the mouths of bays and are intended to reduce a rise in water level behind them in the event of a tsunami. Tsunami protection breakwaters also work together with posterior facilities, such as revetments and dikes, to protect the lives of people and prevent damage to properties that are present behind the breakwaters. Similar to ordinary breakwaters, tsunami protection breakwaters also serve an important purpose, namely, to maintain the calmness of the harbor behind them against oncoming ocean waves.

As a type of tsunami protection facility, a high dike may be constructed along a coastline to protect the land area behind it. However, this approach might result in the loss of a waterfront area that otherwise could be utilized, for example, as a place for social and economic activities. In some cases, there is an insufficient amount of area available for constructing a large facility like a dike. A tsunami protection breakwater may be constructed instead of a dike to make better use of the land and the water surface behind it, with no concerns about the matters mentioned above.

(2) Tsunami protection breakwaters serve to reduce a rise in the water level behind them in the event of a tsunami. By reducing a rise in the water level behind the breakwaters, facilities such as revetments, dikes, and other posterior facilities do not need to have high crowns. In the event that the port or harbor is struck by a tsunami larger than the design tsunami, the tsunami protection breakwater works together with revetments, dikes, and other facilities as an integrated protection system to reduce the extent and depth of inundation on the coast and provide people a longer time to evacuate.

Tsunami protection breakwaters are capable of not only reducing a rise in water level in the event of a tsunami but also reducing the tsunami flow velocity behind them. Ito et al.[1)] made numerical calculations and proved that the breakwater at the mouth of Ofunato Bay is capable of reducing the flow velocity over the entire bay, except the gap. Furthermore, Goto and Sato[2)] proved that the breakwater at the mouth of Kamaishi Bay is effective for reducing the extent of land inundation and the flow velocity behind the breakwater.

(3) In the event of a tsunami, tsunami protection breakwaters serve to reduce a rise in water level and an increase in flow velocity behind them. However, to achieve the required level of protection, a disaster prevention plan that does not rely only on a tsunami protection breakwater but uses it in combination with seawalls and other facilities located behind it is commonly developed. Therefore, the details of a tsunami protection breakwater, including the type, <u>face line</u>, water depths at its gaps, and widths of the gaps, shall be determined to ensure that the water level behind it will not exceed the water level that was determined in consideration of the crown heights and other characteristics of seawalls and other facilities located on the seacoast behind the breakwater when the breakwater is struck by the design tsunami.

When determining the details of the tsunami protection breakwater, it is important to verify the safety and protection performance of the facilities against the design tsunami and to evaluate the safety and protection performance of the facilities against the largest possible tsunami (postulated tsunami) that can occur in the area of interest.[3)]

(4) A safe and economical type of tsunami protection breakwater should be selected on the basis of a comprehensive consideration of many factors, such as the hydraulic conditions of the design tsunami, the design tide level, the design waves <u>and the like</u>, the conditions of the foundation ground, the availability of materials, the constructability of the breakwater, its influences on the surrounding sea areas and adjacent seacoast, its influences on the ecological system, the landscape <u>and the like</u>, the conditions of use of areas behind the breakwater, and the level of difficulty in its maintenance and repair. Similar to ordinary breakwaters, tsunami protection breakwaters can be mainly classified as composite breakwaters, sloping breakwaters, upright breakwaters, and breakwaters covered with wave-dissipating blocks.

A composite breakwater consists of an upright wall body that is constructed on a rubble mound. It works similar to a sloping breakwater or an upright breakwater when the crown of the rubble mound is at a shallow or deep level relative to the wave height, respectively. A sloping breakwater consists of stones or concrete blocks that are piled in a trapezoidal shape, and its main purpose is to dissipate wave energy by making waves break on the slope. An upright breakwater consists of a wall body that is constructed on the seabed and has a vertical front surface. Its main purpose is to reflect wave energy. A breakwater covered with wave-dissipating blocks consists of an upright or composite breakwater and wave-dissipating blocks piled in front of the breakwater, and its purpose is to enable wave-dissipating blocks to dissipate wave energy and to enable the upright wall to prevent waves from passing through the breakwater.

(5) The presence of a tsunami protection breakwater might change the tsunami propagation characteristics and adversely affect the adjacent seacoast. Therefore, the effects of the breakwater should be considered when determining its layout. The effects of the reflection of ordinary waves on the surrounding sea areas should also be considered. When determining the position to construct a tsunami protection breakwater, it is necessary to avoid the positions of the nodes of the natural frequency of a bay and positions where resonance occurs when the design tsunami acts on the bay. If the tsunami protection breakwater is placed in any of these positions, water resonation will occur behind the breakwater because of the action of a tsunami and will result in a significant change in water level.

(6) Reducing the cross-sectional area of a gap of the breakwater is one of the ways to increase the effectiveness of a tsunami protection breakwater in mitigating the impact of a tsunami. However, the gap must have a width and a water depth that will not interfere with the sailing of ships and must face a direction that allows ships to navigate through it easily. Reducing the cross-sectional area of the gap will increase the effectiveness of obstructing the flow of a tsunami, thus preventing tsunami damage; however, this approach may impede the exchange of inshore and offshore waters at normal times. In view of this disadvantage, measures should be taken to prevent seawater from stagnating behind the breakwater and to avoid water quality deterioration.

(7) Tsunami protection breakwaters are constructed to protect areas that are used for <u>material transport</u>, <u>production</u>, fishery, recreational activities, and other purposes. Therefore, the face line of a tsunami protection breakwater shall be determined to minimize the limitation to the current and future use of the areas.

(8) The structural safety of tsunami protection breakwaters shall be secured against the design tsunami. In most cases, a tsunami is caused by an undersea earthquake, and a significant seismic force acts on a tsunami protection breakwater before it is struck by the tsunami. Therefore, the structural safety of facilities against earthquakes should be secured.

Tsunami waves in offshore deepwater areas are smaller than those on the coast, i.e., high waves in stormy weather may exert a larger force on a breakwater than the force of a tsunami. Therefore, the structural safety of facilities against high waves should be secured.

7.2 Basic Concept of Tsunami-Resistant Design

7.2.1 Design Tsunami

The tsunami-resistant design of a breakwater shall be developed to achieve a highly durable structure that will not be collapsed by tsunamis to ensure that its required capabilities remain active against the design tsunami and remain active for the longest extent possible even when it is struck by a tsunami larger than the design tsunami.[4)]

(1) Basic concept of the design tsunami

The tsunami-resistant design of a breakwater against the design tsunami shall be developed with consideration to the functions of a port or harbor behind it and the importance of the facility to ensure that the breakwater can secure the calmness of the harbor immediately after a tsunami and can work effectively to mitigate the possible damage from the tsunami, thus allowing for the possibility of over flowing. Therefore, it is basically necessary to set cross-sectional dimensions so that the capabilities of the breakwater will not be impaired by the action of waves and by the action of the design tsunami.

(2) Demonstration of high durability against a tsunami larger than the design tsunami

To ensure that a breakwater can demonstrate the effectiveness of mitigating possible damage to the greatest extent possible even when it is struck by a tsunami larger than the design tsunami, a highly durable structure that may get deformed but will be hardly collapsed by a tsunami larger than the design tsunami should be developed by taking additional measures for the breakwater on the basis of its cross-sectional dimensions set as described above in (1) and in consideration of the importance, cost-effectiveness, and other characteristics of the facility.

To ensure that a breakwater has a highly durable structure, additional measures shall be developed by [1] assuming that the scale of tsunamis will increase in stages and will ultimately exceed the scale of the design tsunami; [2] thoroughly examining the forms of breakwater failures that can occur owing to tsunamis with different scales, such as the scouring of the foundation mound and seabed and the sliding of the upright wall; [3] identifying the structural weaknesses of the breakwater against tsunamis with different scales; and [4] improving the structure to compensate for weaknesses.

When examining the highly durable structure, its effectiveness should be verified by performing hydraulic model tests and numerical analyses.

7.2.2 Design and Performance Verification Procedure

Fig. 7.2.1 shows an example of a procedure for a comprehensive verification of the overall stability of a breakwater when a tsunami and its preceding earthquake ground motions act on the breakwater. This procedure begins with the setting of the initial cross section against actions due to factors other than tsunamis and proceeds to the setting of cross-sectional dimensions in accordance with **Part III, Chapter 4, 7.4 Actions of Design Tsunami** to ensure that the overall stability of the breakwater will not be impaired by the design tsunami.

The procedure ends with the setting of the cross section of a highly durable structure by making a comprehensive judgment on the basis of the importance, cost-effectiveness, and other characteristics of the facility in accordance with **Part III, Chapter 4, 7.5 Examination of a Highly Durable Structure against Tsunamis Larger than the Design Tsunami**.

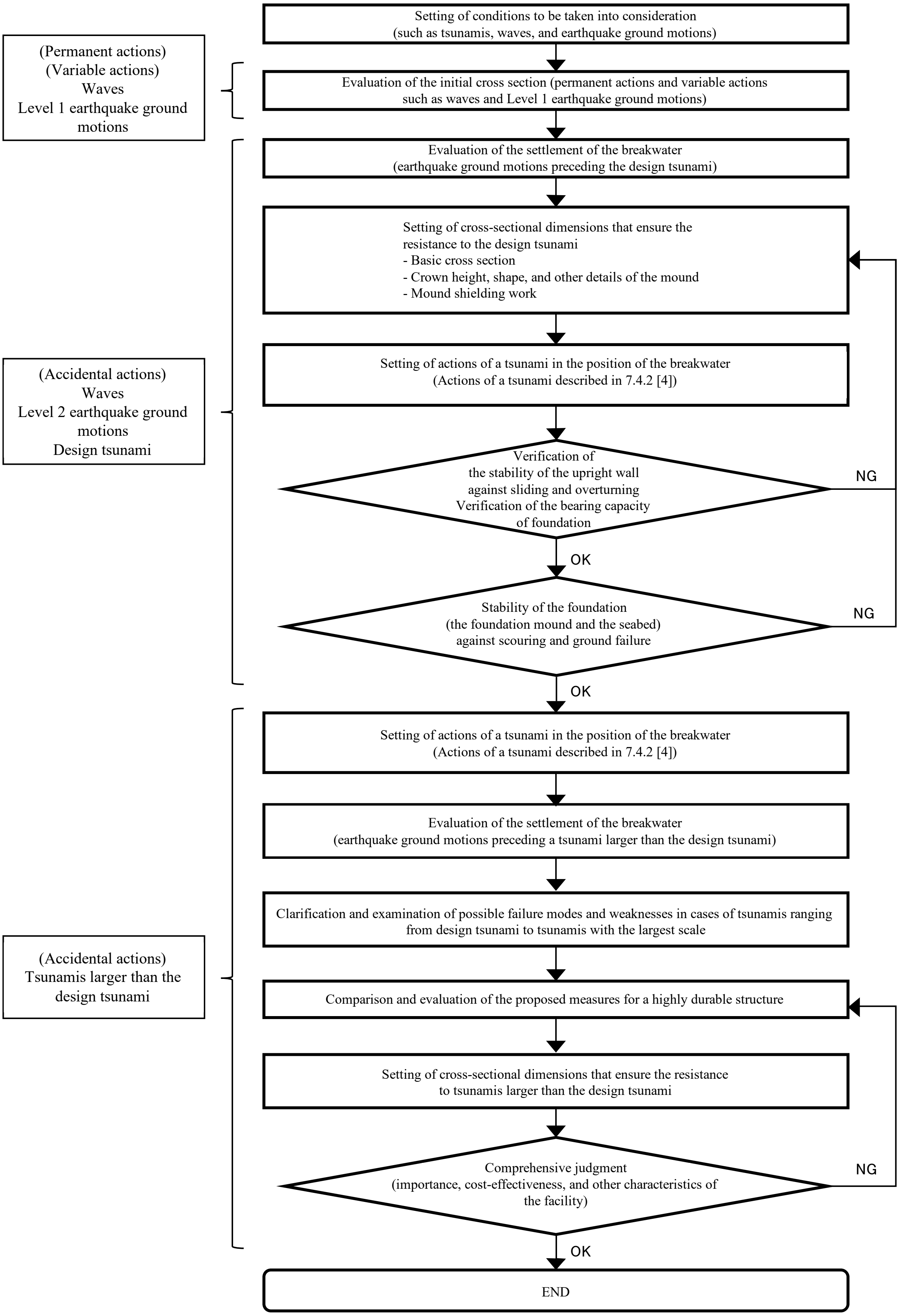


Fig. 7.2.1 Example of the Procedure for the Comprehensive Verification of the Overall Stability of a Breakwater

7.3 Setting of the Basic Cross Section and Required Considerations

7.3.1 Setting of the Basic Cross Section

(1) The crown height of a tsunami protection breakwater should be set to the height necessary for preventing wave overflowing when the design tsunami acts on the breakwater with the tide level set appropriately.

(2) Tsunami protection breakwaters shall conform to **Part III, Chapter 4, 3 Breakwaters Having Basic Functions** with necessary modifications in consideration of the structural type.

7.3.2 Considerations Required in Setting the Basic Cross Section

(1) In the performance verification of a breakwater, its overall stability against possible tsunamis and their preceding earthquake ground motions shall be verified comprehensively on the basis of the forms of breakwater failures caused by tsunamis and by giving due consideration to the various characteristics of the port or harbor, including its geographical features and facility layout.

(2) Factors that contributed to damage to breakwaters that were struck by tsunamis triggered by the earthquake off the Pacific coast of Tohoku[5)]

In the 2011 earthquake off the Pacific coast of Tohoku, a large tsunami struck the Pacific coast of eastern Japan and caused serious damage to ports and harbors on the Pacific coast, including breakwaters. The affected breakwaters included the north breakwater at the Port of Hachinohe, the breakwater at the mouth of Kamaishi Bay, the breakwater at the mouth of Ofunato Bay, and the offshore breakwater of the Port of Soma. According to the analysis results of the damage, the main factors that contributed to breakwater damage include the sliding of upright walls caused by the tsunami wave force, the loss of bearing capacity of their rubble mounds and the seabed inside the ports due to tsunami overflowing and resultant scouring, or the combinations of these actions. The results also indicated that the foundation may have become unstable owing to seepage flow through the foundation mound.

(3) Identification of failure factors and limiting conditions

In the performance verification of a breakwater, the weaknesses that can constitute the factors of failures should be identified on the basis of the forms of possible breakwater failures caused by tsunamis, propose two or more countermeasures, and compare and examine the countermeasures by taking into account the geographic features of the port or harbor, the layout of facilities, and other conditions.

For example, if a waterway (sea route) runs immediately behind the breakwater, this situation imposes a limitation to the area available for constructing countermeasure work inside the port or the harbor. Furthermore, the head of the breakwater is exposed to a high risk of scouring because of concentrated flow through the gap. In the event that the head of the breakwater is displaced, the resultant scouring will cause the successive displacement of adjacent caissons. Therefore, a highly durable structure for the breakwater should be developed, particularly for the head of the breakwater.

It is necessary to study the results of the latest researches and technology developments, including those achieved by private companies; identify the effective countermeasures proposed to cope with failure factors and weaknesses; and set the optimum cross-sectional dimensions on the basis of a comprehensive judgment.

For the identification of effective measures, refer to **Table 7.3.1**.

Table 7.3.1 Main Failure Factors of Breakwaters and Effective Countermeasures

Row: Countermeasure work Column: Failure factor	Countermeasure [1]* Increasing the weight	Countermeasure [2]* Increasing the resistance	Countermeasure [3]* Changing the structure	Countermeasure [4]* Supporting the breakwater from behind	Countermeasure [5]* Covering the mound	Countermeasure [6]* Controlling over flowing water (for example, diverting the flow)
Tsunami wave force	Changing the shape of the superstructure and/or adjusting the specific gravity of filling materials	Laying friction enhancement mats	Changing the shape of the main body**	Additional rubble mound	—	—
Scouring due to over flowing	—	—	—	Levee widening work or the like	Placing armor or foot protection blocks or the like	Changing the shape of the superstructure
Ground seepage flow	—	—	Changing the shape of the main body** Reducing the water permeability of the mound core	Levee widening or the like	—	—

*: It should be noted that the numbers in brackets do not show the order of priority among the countermeasures.

**: It must also be noted that the effectiveness of changing the shape of the main body varies depending on the structure. For example, for a breakwater with an embedded structure, changing the shape of its main body is effective for preventing seepage flow in the ground.

7.3.3 Setting of Cross-Sectional Dimensions

For the tsunami-resistant design of a breakwater, appropriate cross-sectional dimensions shall be determined to ensure that the overall stability of the breakwater will not be impaired by actions of the design tsunami and its preceding earthquake ground motions. The main cross-sectional dimensions that should be determined in the tsunami-resistant design of breakwaters are listed below:

Rubble mound : mound shape, crown height and width, and rubble specifications (weight)

Upright wall : caisson width, crown height, and crown shape (such as parapet)

Foot protection : specifications of foot protection work, including the quantity and layout of foot protection blocks

Armor units : specifications of armor units, including the layout and other details of armor stones and armor blocks

Wave-dissipating work : specifications of wave-dissipating work

Additional rubble mound : height, width, and shape of additional rubble mound and specifications of armor units for additional rubble mound

Scouring prevention work for seabed : specifications of scouring prevention work, including the layout of scouring prevention mats and the like

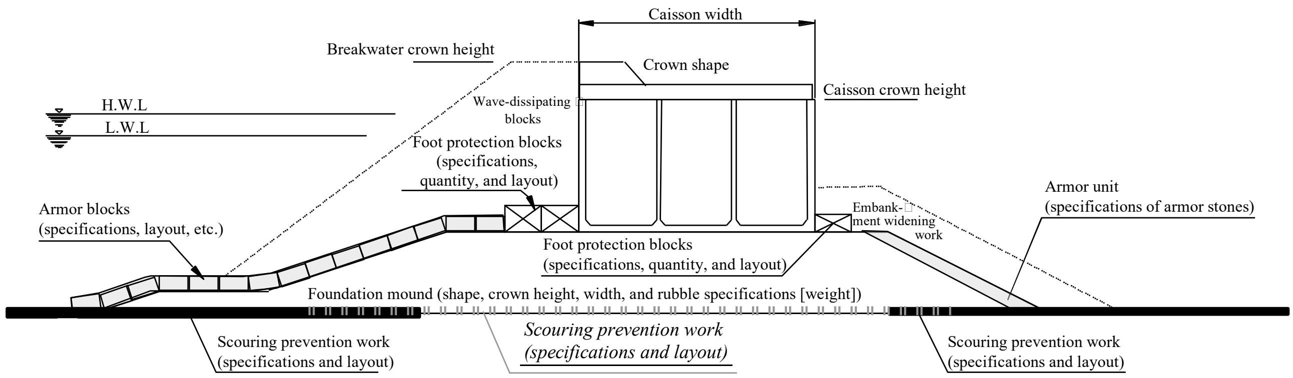


Fig. 7.3.1 Example of the Setting of Cross-Sectional Dimensions

7.4 Actions of Design Tsunami

7.4.1 Setting of Earthquake Ground Motions Preceding a Tsunami and the Evaluation of Their Effects

(1) Setting of earthquake ground motions preceding a tsunami

In the performance verification of a breakwater, earthquake ground motions preceding a tsunami should be appropriately set, and their effects should be appropriately evaluated.

① **Setting of earthquake ground motions preceding the design tsunami**

Earthquake ground motions preceding the design tsunami shall be set by making a model of faults in relation to an earthquake that triggers the design tsunami and by setting a time-history waveform on the engineering bedrock surface by using the technique described in **Part II, Chapter 6, 1.3 Level 2 Earthquake Ground Motions Used in Performance Verification of Facilities**.

② **Setting of earthquake ground motions preceding a tsunami larger than the design tsunami**

In some cases, it is possible to set earthquake ground motions preceding a tsunami larger than the design tsunami by using the same technique as that described above in ①. However, for convenience, appropriate earthquake ground motions can be set by taking into consideration the following earthquake ground motions:

(a) Level 2 earthquake ground motions set in the seismic design (only when subduction zone earthquakes are of interest)

(b) Other earthquake ground motions (including those based on a model of faults that take into account the strong motion pulse generation area with a non-exceedance probability of 50% or more)

(2) Setting of crustal movements preceding a tsunami

Crustal movements shall be set on the basis of an appropriate evaluation that takes into account the characteristics of earthquakes of interest and the amounts of crustal movements caused by past earthquakes.

(3) Evaluation of effects on the overall stability of breakwaters

The foundation ground immediately below a breakwater may become soft owing to liquefaction or other ground failure caused by earthquake ground motions preceding a tsunami, and this situation may result in the settlement of the breakwater. Crustal movements may cause the settlement of a breakwater. When a breakwater settles down, it becomes more likely to be overtopped by tsunami waves, thus increasing the possibility that the foundation mound or the seabed behind the breakwater might be scoured by tsunami waves and increasing the risk that the overall stability of the breakwater might deteriorate. Furthermore, the buoyancy and wave forces acting on the breakwater might become larger and cause a decrease in the stability of the upright wall against sliding and other failures.

Therefore, in the verification of the overall stability of a breakwater, it is necessary to appropriately evaluate the effects of the settlement of the breakwater due to earthquake ground motions and crustal movements preceding a tsunami.

The settlement of the breakwater due to the liquefaction of the seabed shall be estimated by allowing for the settlement associated with shearing deformation and the volume compression of the foundation mound and the seabed and by taking into consideration past cases of breakwater settlement and research studies. Furthermore, the preliminary inclusion of the margin for settlement in the crown height of a breakwater shall be determined as needed. However, it must be noted that increasing the crown height of the breakwater will result in increases in the wave force and other effects of variable waves on the breakwater.

7.4.2 Points to Remember regarding the Actions of Tsunamis

(1) For tsunamis, refer to **Part II, Chapter 2, 5 Tsunamis**.

(2) In the verification of performance of a breakwater against tsunamis, the difference in the water level between the inside and outside of the breakwater should be appropriately evaluated when it is exposed to the action of a tsunami on the basis of a numerical simulation. It should be noted that the water level behind the breakwater will not always be equivalent to the still water level owing to the incoming and outgoing waves of the tsunami.

(3) For the calculation of tsunami wave force, refer to **Part II, Chapter 2, 6 Wave Forces**. However, because there are many points that still need to be clarified, the wave force should be confirmed by using an appropriate method such as a hydraulic model test. Furthermore, it is necessary to consider a decrease in the bearing capacity of the foundation mound and the scouring of the sandy ground below it because of seepage flow.

(4) Points to remember about the actions of tsunamis

① **Various characteristics that affect actions of tsunamis**

Damage to a breakwater due to the actions of tsunamis varies significantly in terms of the affected area, the severity and form of damage, the characteristics of tsunamis of interest, and the characteristics of the port or harbor. Therefore, it is necessary to appropriately set the wave force, flow velocity, and other characteristics of actions of tsunamis that should be considered in the performance verification of the breakwater by giving due consideration to the results of numerical analyses (tsunami simulations), hydraulic model tests, and other similar methods.

- Characteristics of tsunamis: heights, flow velocities, directions, period characteristics, time-variation characteristics, durations, and other characteristics of tsunamis of interest
- Characteristics of the sea area: geographic features, water depth, facility layout (such as positions and widths of gaps), crown height (possibility of over flowing) of the breakwater, and other characteristics

② **Consideration of the time-variation characteristics of the actions of tsunamis**

A tsunami has a long duration and generates landward and seaward motions of waves repeatedly. Furthermore, the direction at which a tsunami wave hits a breakwater and the period characteristics of the tsunami vary significantly with time. In the verification of the stability of the breakwater against the tsunami wave force and in the verification of countermeasures against the scouring of the rubble mound and the seabed, it is necessary to verify the stability of the whole breakwater and to give due consideration to the maximum tsunami height and maximum flow velocity, as well as the duration and time-variation characteristics.

(5) Performance verification utilizing hydraulic model tests and numerical analyses

At present, sufficient knowledge is not always available about the methods for verifying performance against tsunamis. When conducting the performance verification of a breakwater, its overall stability should be confirmed by fully using hydraulic model tests and numerical analyses.

7.4.3 Performance Verification of the Stability of Breakwater Body

(1) When verifying the stability of a breakwater against the wave force of the design tsunami in terms of the stability of the upright wall against sliding and overturning and the bearing capacity of the rubble mound, it is necessary to appropriately evaluate the tsunami wave force acting on the upright wall and the effects of tsunami flow on the rubble mound and seabed.

(2) Setting of the tsunami wave force acting on the upright wall

When a tsunami strikes a breakwater, water pressures act on the following surfaces of the upright wall: the front and rear surfaces hit by the landward and seaward motions of waves respectively; the bottom surface; and the crown surface (as shown in **Fig. 7.4.1**). When verifying the stability of the upright wall, it is necessary to appropriately evaluate the magnitudes of these water pressures and their distribution characteristics and to set the tsunami wave force acting on the upright wall.

When a tsunami strikes and overtops a breakwater, its wave force acting on the breakwater varies depending on the shape of the parapet and other factors. Therefore, the tsunami wave force should be appropriately evaluated on the basis of the results of hydraulic model tests, numerical analyses, and other similar methods.

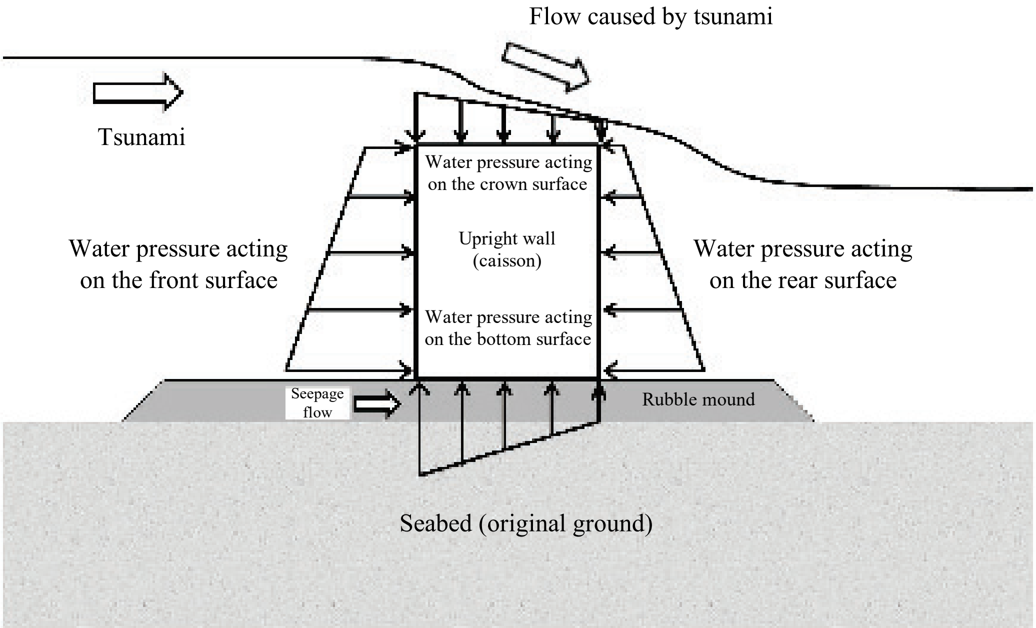


Fig. 7.4.1 Water Pressures Caused by a Tsunami and Acting on the Front and Rear Surfaces, the Crown Surface, and the Bottom Surface of an Upright Wall

(3) Stability of a breakwater against the tsunami wave force in terms of the stability of the vertical wall against sliding and overturning and the bearing capacity of the rubble mound

In general, the stability of a breakwater against the tsunami wave force should be evaluated in terms of the stability of the upright wall against sliding and overturning and the bearing capacity of the rubble mound in accordance with **Part III, Chapter 4, 3 Breakwaters Having Basic Functions**.

When applying the equations given below to the verification of the stability of a breakwater that is being designed, the characteristic values should be set by appropriately evaluating the tsunami wave force acting on the breakwater with consideration to various characteristics affecting the actions of a tsunami. It is also necessary to appropriately evaluate the effects of the seepage flow through the rubble mound because it decreases the bearing capacity of the rubble mound. Furthermore, the characteristic values should be set by thoroughly examining other factors that affect the stability of the upright wall, including the softening of the ground due to the liquefaction of the seabed or other ground failure, and by appropriately evaluating the friction coefficient and the shear strength of ground materials.

Equations (7.4.1), **(7.4.2)**, and **(7.4.3)** may be used to verify the stability against sliding, the stability against overturning, and the bearing capacity of the rubble ground, respectively. In the following equations, subscripts k and d indicate the characteristic value and the design value, respectively. The numerical values given in **Tables 7.4.1**, **7.4.2**, and **7.4.3** may be used for the partial factors in the equations. The "—" symbol shown in each table indicates that the numerical value in parentheses may be used to simplify verification calculations.

① **Verification of stability against sliding**

$$m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k$$
$$R_k = f_k (W_k - P_{B_k} - P_{U_k}) \tag{7.4.1}$$
$$S_k = P_{H\,k}$$

where

f : friction coefficient between caisson and rubble mound;

W : weight of the breakwater body (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

P_H : Horizontal wave force of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : resistance factor;

γ_S : load factor;

m : adjustment factor.

Table 7.4.1 Partial Factors to be Used for the Verification of the Stability of Breakwater Body against Sliding

Object of verification	Partial factor by which the resistance term is multiplied, γ_R	Partial factor by which the load term is multiplied, γ_S	Adjustment factor, m
Sliding of breakwater body (Accidental situation due to design tsunami)	— (1.00)	— (1.00)	1.20

② **Verification of stability against overturning**

$$
m \cdot \frac{S_d}{R_d} \leq 1.0 \quad R_d = \gamma_R R_k \quad S_d = \gamma_S S_k
$$
$$
R_k = \{a_1 W_k - a_2 P_{B_k} - a_3 P_{U_k}\} \qquad (7.4.2)
$$
$$
S_k = a_4 P_{H_k}
$$

where

W : weight of breakwater body (kN/m);

P_B : buoyancy (kN/m);

P_U : uplift of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

P_H : horizontal wave force of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

a_1 to a_4 :arm length of each action (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : resistance factor;

γ_S : load factor;

m : adjustment factor.

Table 7.4.2 Partial Factors to Be Used for the Verification of the Stability of Breakwater Body against Overturning

Object of verification	Partial factor by which the resistance term is multiplied, γ_R	Partial factor by which the load term is multiplied, γ_S	Adjustment factor, m
Overturning of breakwater body (Accidental situation due to design tsunami)	— (1.00)	— (1.00)	1.20

③ Verification of the bearing capacity of the rubble ground

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

$$F_f = \frac{R_k(F_f)}{S_k}$$

$$R_k = \sum \left[\frac{\{c'_k s + (w'_k + q_k)\tan\phi'_k\}\sec\theta}{1 + \tan\theta\tan\phi'_k / F_f} \right] \tag{7.4.3}$$

$$S_k = \sum\{(w_k' + q_k)\sin\theta\} + \frac{a_1 P_{H_k}}{r}$$

where

P_H : horizontal wave force of tsunami (kN/m) (refer to **Part II, Chapter 2, 6 Wave Forces**);

a_1 : arm length of the horizontal wave force of a tsunami (m);

c' : undrained shear strength for clayey ground or apparent cohesion under drained condition for sandy ground or stone (kN/m^2);

s : width of the slice segment (m);

w' : effective weight of the slice segment (kN/m) (weight in the air for the part above the water surface or weight in water for the part below the water surface);

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

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

θ : angle formed by slice segment with bottom (°).

F_f : supplementary parameter showing ratio of design value of resistance and design value of effect of action;

r : radius of slip circle (m);

R : resistance term (kN/m);

S : load term (kN/m);

γ_R : resistance factor;

γ_S : load factor;

m : adjustment factor.

Table 7.4.3 Partial Factors to Be Used for the Verification of the Bearing Capacity of Foundation Ground

Object of verification	Partial factor by which the resistance term is multiplied, γ_R	Partial factor by which the load term is multiplied, γ_S	Adjustment factor, m
Failure of bearing capacity of foundation ground (Accidental situation due to design tsunami)	— (1.00)	— (1.00)	1.00

Equation (7.4.3) is the effective stress expression based on the simplified Bishop's method. When **equation (7.4.3)** is used, the supplementary parameter F_f shall be first determined by repeated calculations so that $R_k = F_f \times S_k$ is satisfied. Note that F_f is contained in the equation for R_k. Thereafter, the R_k and S_k obtained from these calculations shall be used for the verification of the stability in terms of bearing capacity.

Considering that the seepage flow in the rubble mound causes a decrease in the bearing capacity of the rubble, the effects of the seepage flow in the verification of bearing capacity should be appropriately evaluated. It was indicated that the bearing capacity may decrease by approximately 10 to 17% at a breakwater of ordinary shape and scale when there is a difference of approximately 9 to 10 m in the water level between the inside and outside of the port or harbor.[6), 7)] However, the effects of the seepage flow have not been fully understood yet; therefore, it is preferable to include a margin of approximately 20% in the design of a breakwater for a port or harbor where the difference in the water level is 10 m.[8)] The rate of decrease in the bearing capacity is proportional to the difference in water level. For example, when the difference in the water level between the inside and outside of the port or harbor is 5 m, the rate of decrease should be considered as 10%, and **equation (7.4.3)** should be satisfied by using the adjustment factor of 1.1. It is also possible to verify the bearing capacity in consideration of the effects of the seepage flow by using a finite element analysis in which the effects of the seepage flow can be considered or by using a full-scale centrifuge model test in which the bearing capacity characteristics can be simulated. When using the finite element method, it is necessary to use the apparent cohesion and the angle of shear resistance that allows an appropriate evaluation of the shear strength of the ground under the confining pressure that decreased due to the effects of seepage flow.[9)]

(4) Examples of damage to breakwaters due to the actions of the tsunami caused by the 2011 earthquake off the Pacific coast of Tohoku

Fig. 7.4.2 shows the results of a research on the frontline breakwaters of ports* that were exposed to the actions of the largest-scale tsunami triggered by the 2011 earthquake off the Pacific coast of Tohoku. To determine whether the breakwaters were significantly damaged by the tsunami**, data were analyzed using two indices: one is the safety factor against the sliding of the upright wall of each breakwater, and the other is the over flowing water depth, which is defined as the water depth from the crown surface of the breakwater to the tsunami water level on the seaward side of the breakwater.

*: Port of Hachinohe, Port of Ofunato, Port of Kamaishi, Port of Soma, Port of Kuji, Port of Sendai Shiogama, and Port of Onahama

**: Breakwaters that slid inside their design construction site are counted as breakwaters that were damaged by the tsunami.

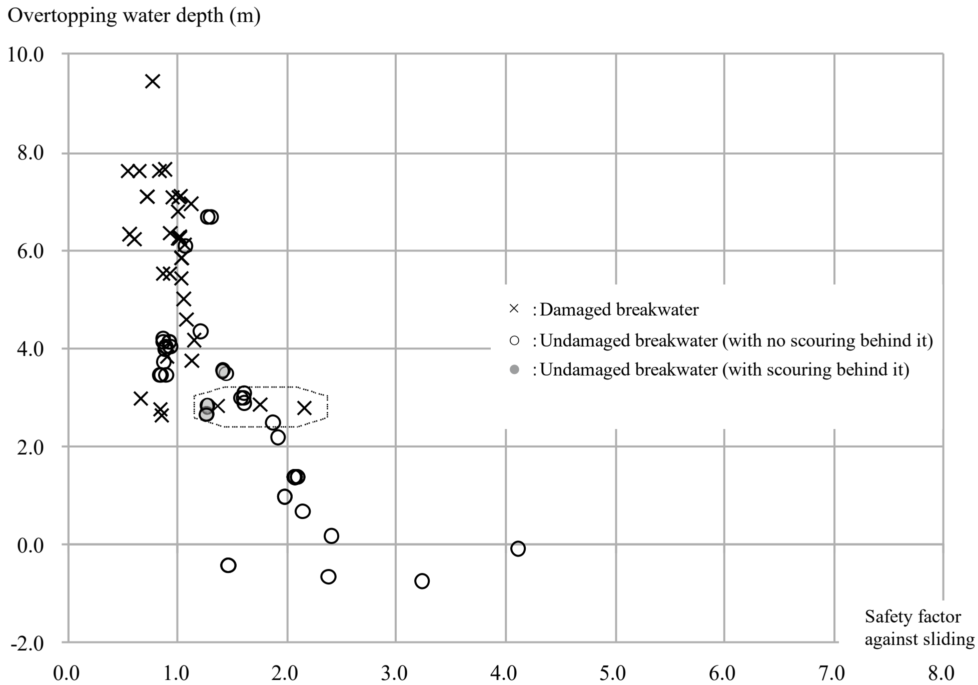


Fig. 7.4.2 (a) Relationship between the Over flowing Water Depth and the Safety Factor against Sliding for Damaged and Undamaged Breakwaters (Case Examples of Damage to Breakwaters Due to the 2011 Earthquake Off the Pacific Coast of Tohoku)

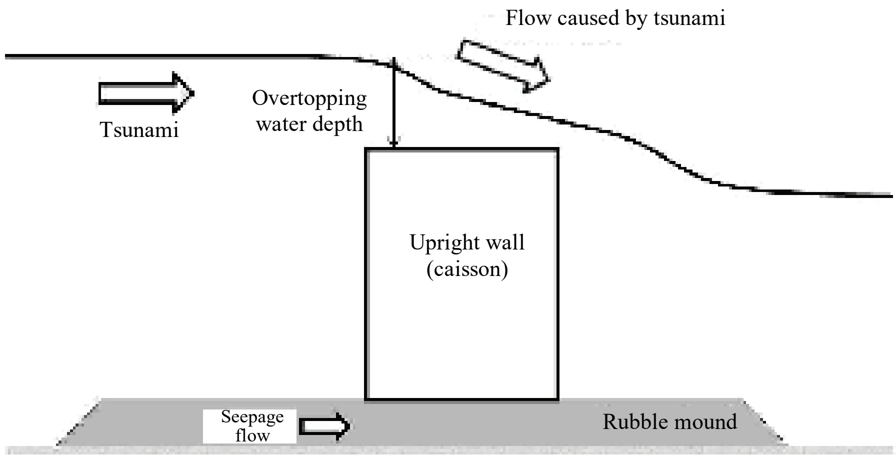


Fig. 7.4.2 (b) Definition of Over flowing Water Depth

This figure indicates that there were many damaged breakwaters in cases where the safety factor against sliding defined by conventional standards was lower than approximately 1.2. It also indicates that breakwaters were damaged from scouring in cases wherein the over flowing water depth exceeded approximately 2 m. Some breakwaters were damaged even in cases wherein the safety factor against sliding was higher than 1.2 (the three × marks enclosed by the dotted-line in the chart indicate these breakwaters). It is considered that these breakwaters were damaged from over flowing and the resultant scouring of the mound or seabed behind them (refer to **Part II, Chapter 4, 7.4.4 Stability of the Rubble Mound and Seabed against Tsunami Flow**). Some other breakwaters showed signs of scouring of the mound and the seabed behind them as a result of over flowing even though there was no displacement of caissons (the gray circles in the chart indicate these breakwaters).

7.4.4 Stability of the Rubble Mound and Seabed against Tsunami Flow

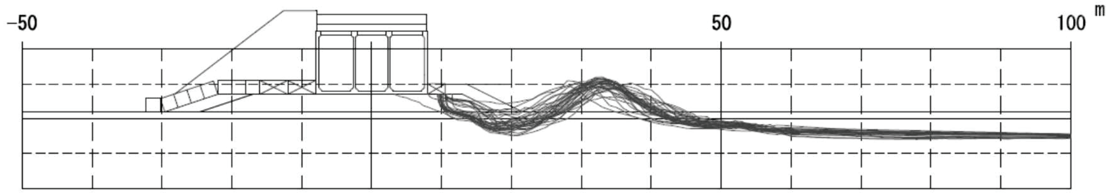
(1) In the evaluation of the stability of the rubble mound and the seabed against the tsunami flow caused by the design tsunami, it is necessary to appropriately evaluate over flowing, seepage flow, and other actions of the tsunami flow on the rubble mound and the seabed and to take appropriate measures to prevent the occurrences of scouring and rubble mound failures that may impair the stability of the upright wall.

(2) Significance of countermeasures against scouring

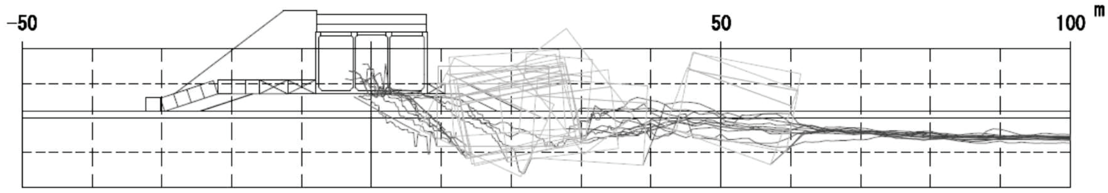
A large-scale tsunami has a large wave force and generates a long-lasting, strong, one-way current. When the rubble mound (including foot protection work and shielding work) of a breakwater and the seabed are scoured owing to the tsunami flow, including over flowing and seepage flow, this might cause the settlement of caissons and a decrease in the bearing capacity of the rubble mound, thus resulting in the collapse of the upright wall.

When a tsunami overtops the breakwater, it becomes more likely that the rubble mound and the seabed behind it will be scoured and results in the collapse of the upright wall. **Fig. 7.4.3** shows a specific case of scouring behind a breakwater. In the 2011 earthquake off the Pacific coast of Tohoku, the largest-scale tsunami struck the north breakwater at the Port of Hachinohe and caused scouring on the rubble mound and the seabed behind it, thus resulting in damage to the breakwater (in the 11th construction site of the breakwater). **Fig. 7.4.3(a)** shows the superposed cross sections along the survey lines for the remaining part of the damaged upright wall. **Fig. 7.4.3(b)** shows the superposed cross sections along the survey line for the slid part of the damaged upright wall in the said construction site.

As seen in these figures, the rubble mound and seabed behind the breakwater were significantly scoured even though the upright wall was not displaced. It can also be seen that scoured soil was deposited on the landward side of the scoured area. Considering that the seabed behind a breakwater is increasingly scoured, the rubble mound and seabed will become unable to bear the upright wall. The upright wall will ultimately fall down on the scoured area and will get damaged.



(a) Remaining part of the upright wall



(b) Slid part of the upright wall

Fig. 7.4.3 State of Damage Due to Over flowing in the Eleventh Construction Site of the North Breakwater at the Port of Hachinohe

The foundation mound and seabed are also likely to be scoured in areas around the head and gaps of a breakwater owing to the strong concentrated flow of a tsunami current. This increases the risk of the collapse of the upright wall of the breakwater. As a specific case of scouring around the head or gap of a breakwater, **Fig. 7.4.4** shows the location where the Port of Hachinohe scouring occurred due to the tsunami caused by the 2011 earthquake off the Pacific coast of Tohoku. Scouring down to a depth of more than 10 m from the original sand-bed was found around the heads and gaps of the breakwaters. Around the central part of the gap between the first central breakwater and the second central breakwater, scouring occurred and resulted in the collapse of caissons at the heads of the breakwaters.

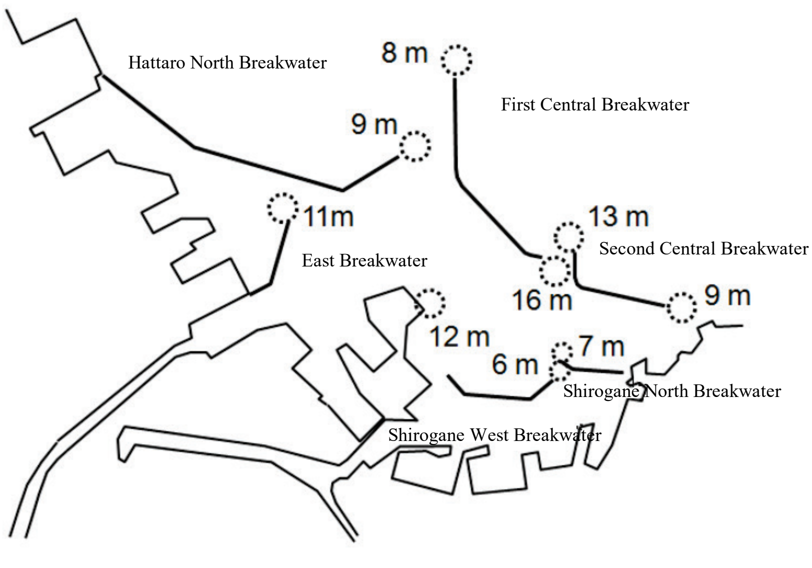


Fig. 7.4.4 Scouring of the Rubble Mounds and the Seabed (Port of Hachinohe)
The dotted-line circles indicate the scoured areas, and the numerical values indicate the depth of scouring.[10)]

In protecting a port or harbor from damage caused by tsunamis, it is more effective in some cases to take small-scale measures to reinforce the main body of a breakwater than to take additional measures, such as countermeasures against scouring. One measure that can be taken in such cases is to increase the width and/or crown height of the upright wall in consideration of the importance and cost-effectiveness of the facility. However, it must be noted that increasing the crown height of a breakwater will cause the breakwater to be subjected to the greater effects of waves.

(3) Points to remember in examining countermeasures against scouring

The tsunami-resistant performance of a breakwater against the design tsunami can be significantly affected by currents flowing at higher velocities in areas around the head and gaps and by over flowing. Therefore, the basic countermeasures against scouring involve ensuring that the rubble mound and seabed will not be scoured by waves and the design tsunami. If it is difficult to completely prevent the scouring of the seabed, measures should be taken to ensure that scouring will not cause the rubble mound to collapse in chain and impair the stability of the upright wall.

With regard to scouring due to over flowing, a situation wherein the tsunami height in front of a breakwater is maximized does not always result in the most severe scouring of the rubble mound and seabed. Therefore, when considering countermeasures against scouring due to over flowing, it is necessary to focus on the water levels and the other time-variation characteristics of tsunamis of interest, which are listed below:

- During the landward motion of waves (when the tsunami flow acts from the outside to the inside of a port or harbor)

  - Range of the tsunami water level on the front side (outside the port or harbor)
  - Range of the still water level on the rear side (inside the port or harbor)
  - Range of the settlement of the breakwater (allowing for rise)
  - Duration of over flowing

- During the seaward motion of waves (when the tsunami flow acts from the inside to the outside of the port or harbor)

  - The same items as those during the landward motion of waves shall be taken into consideration.

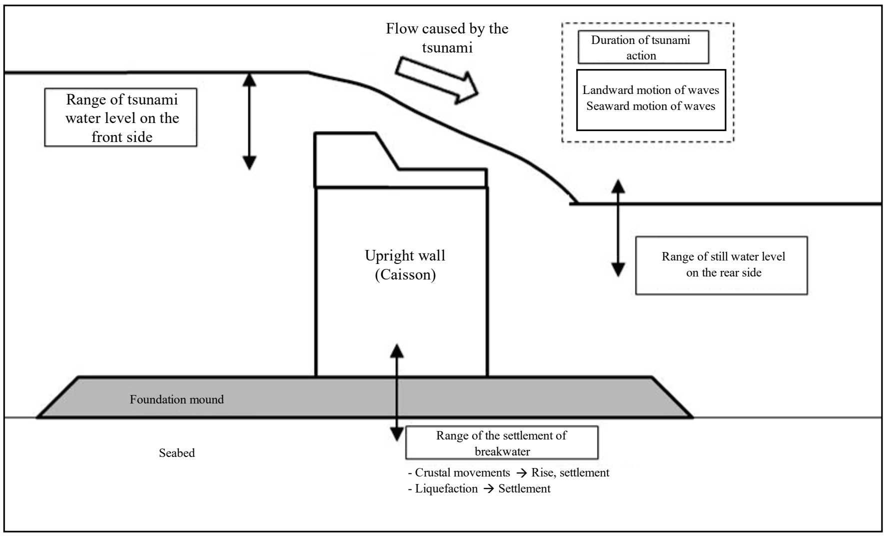


Fig. 7.4.5 Diagram of Conditions to Be Considered in the Examination of Countermeasures against Scouring

Adequate attention must be paid to the durability of armor materials to ensure that shielding work can fulfill its function when a tsunami strikes the port or harbor. For the required weight of shielding work, refer to **Part II, Chapter 2, 6 Wave Forces**. It is possible that the scouring of a rubble mound due to over flowing may be accelerated when the confining pressure of the mound decreases because of the effects of seepage flow.[11), 12)]

(4) Examination of countermeasures against scouring based on hydraulic model tests

Hydraulic model tests and numerical analyses should be used to examine and determine specific measures to protect the rubble mound of a breakwater and seabed from the scouring caused by over flowing or by very swift currents that can occur in areas around the head and gaps of the breakwater.

(5) Examination of piping

When seepage flow occurs in the rubble mound owing to differences in water level between the inside and outside of the port or harbor, piping might occur in the area of the mound that is close to the lowest part on the rear side of a caisson. Therefore, it is necessary to conduct piping verification and to construct the mound without using small stones to mitigate piping.[7)]

7.4.5 Additional rubble mound

Levee widening work can increase the sliding resistance of the upright wall and the bearing capacity of the rubble mound and can also reduce the scouring of the rubble mound and the sandbed behind the upright wall. Therefore, a breakwater with levee widening work is considered to have a structure that will hardly collapse even when it is struck by a large tsunami.

Fig. 7.4.6 shows a schematic diagram of levee widening work. It is possible to increase the stability against over flowing by constructing shielding work, foot protection work, and scouring prevention work. Levee widening work may be examined by using the method described in **Part III, Chapter 4, 3 Breakwaters Having Basic Functions**.

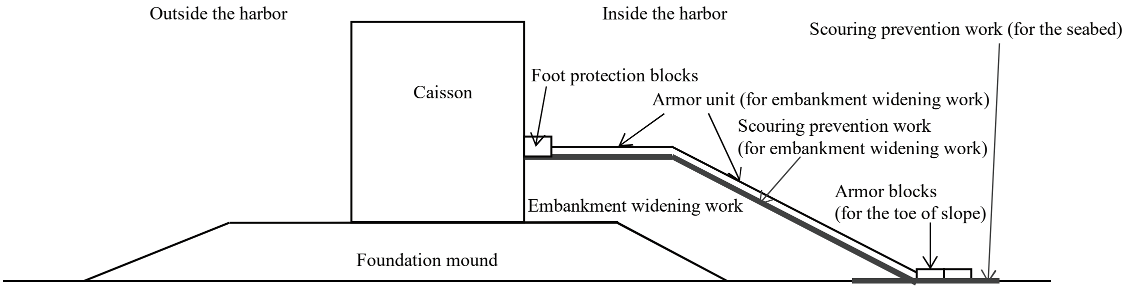


Fig. 7.4.6 Levee Widening Work Combined with Shielding Work, Foot Protection Work, and Scouring Prevention Work

In cases wherein it is difficult to construct extensive countermeasure work for a breakwater (e.g., at the head of the breakwater or in an area behind the breakwater where there is a sea route or anchorage area), it is suggested to use steel pipe piles and concrete block frames to make the cross section of the breakwater smaller than that of a breakwater with ordinary levee widening work on the basis of the tsunami over flowing condition and the flow velocity condition behind the breakwater.[13)]

7.5 Examination of a Highly Durable Structure against Tsunamis Larger than the Design Tsunami

7.5.1 General

When examining the highly durable structure of a breakwater against tsunamis larger than the design tsunami, cross-sectional dimensions that are appropriate for the security objectives of the port or harbor should be set to ensure that the highly durable structure can maintain the overall stability of the breakwater as much as possible even when it is struck by a tsunami larger than the design tsunami. This can be performed by thoroughly examining the forms of failures and structural weaknesses of the breakwater in relation to tsunamis of different scales and by taking additional measures to compensate the weaknesses.

7.5.2 Concept of the Highly Durable Structure of a Breakwater

When examining the resistance of a breakwater against tsunamis larger than the design tsunami, the breakwater should have a highly durable structure that allows it to deform but not to collapse when it is struck by a tsunami larger than the design tsunami, thereby allowing the breakwater to maintain the overall stability of the breakwater as much as possible. This can be accomplished by assuming that there are possible tsunamis with scales that increase in stages and exceed the scale of the design tsunami and that tsunamis work as external forces on the cross section that was set against waves and the design tsunami (refer to **Part III, Chapter 4, 7.4 Actions of Design Tsunami**), by thoroughly examining the forms of failures and structural weaknesses of the breakwater in relation to tsunamis of different scales on the basis of hydraulic model tests and other similar tests, and by taking additional measures to compensate for the weaknesses in consideration of the importance, cost-effectiveness, and other characteristics of the facility.

7.5.3 Examination of Additional Measures against Tsunamis of Scales Increased in Stages

When it is assumed that there are possible tsunamis with scales that increase in stages and exceed the scale of the design tsunami, the extent of over flowing of a breakwater becomes larger in stages, thus increasingly revealing the structural weaknesses of the breakwater and resulting in severe damage to the structure. Therefore, when examining the highly durable structure of the breakwater, it is preferable to assume that there are possible tsunamis with scales that increase in stages and exceed the scale of the design tsunami, to verify the effectiveness of additional measures to compensate structural weaknesses (or means of improving the structure), and to determine the specific cross-sectional dimensions in consideration of the importance, cost-effectiveness, and other characteristics of the facility.

Fig. 7.5.1 shows an example of the examination of measures against tsunamis with scales that increase in stages. The horizontal axis indicates the scales of tsunamis. The vertical axis indicates the cost required for taking additional measures (or means of improving the structure) for a breakwater to ensure that it has a highly durable structure; therefore, it will not collapse when it is struck by a tsunami larger than the design tsunami. The two curves show how the cost increases when measures are taken against tsunamis with scales that increase in stages.

In the figure, Breakwater A represents a breakwater that is designed to serve in an area where it is exposed to gentle waves, and the horizontal wave force generated by the waves and acting on the breakwater is larger than the tsunami wave force generated by the design tsunami but is smaller than the tsunami wave force generated by a tsunami of the largest scale. When it is assumed that there are possible tsunamis with scales that increase in stages and exceed the scale of the design tsunami, the stability of this breakwater against the sliding of the upright wall and other failures will decrease drastically against tsunamis with scales that exceed the stage (marked with a black circle in the figure) where a tsunami is not as large as a tsunami of the largest scale; therefore, a breakwater with a cross section that is designed to resist waves can resist a tsunami. Therefore, it is considered necessary to construct levee widening work behind the breakwater or take other large-scale means of improving the structure to ensure that the breakwater has high durability against tsunamis with scales that exceed the said stage.

By contrast, Breakwater B represents a breakwater that is designed for an area exposed to rough waves and wherein the horizontal wave force generated by the waves and acting on the breakwater is equivalent to the tsunami wave force generated by a tsunami of the largest scale. The stability of this breakwater against the sliding of the upright wall or other failure will not drastically decrease against tsunamis with scales that exceed the stage where a tsunami is not as large as a tsunami of the largest scale. Therefore, it is considered possible to maintain the high durability against tsunamis with scales up to the largest scale by taking small-scale means of improving the structure, that is, by taking countermeasures against scouring caused by over flowing.

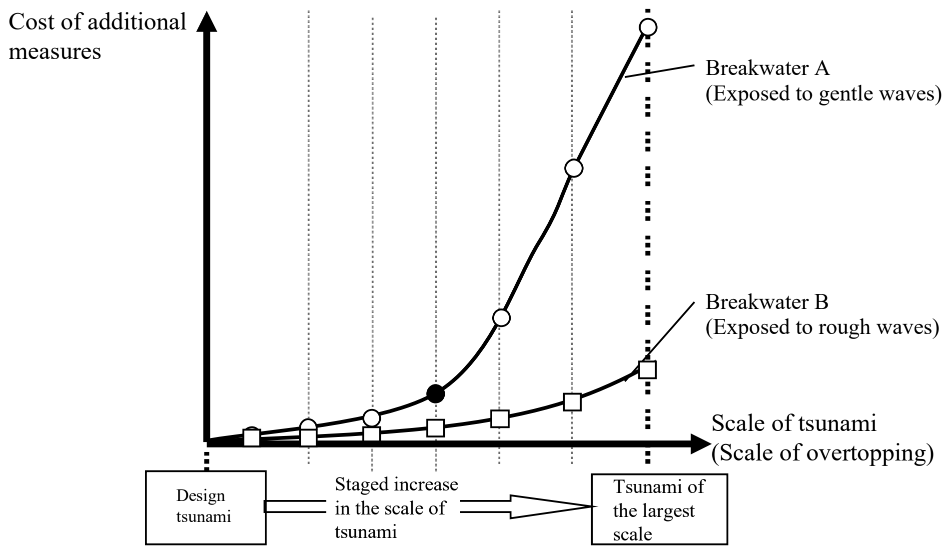


Fig. 7.5.1 Example of the Examination of Additional Measures against Tsunamis with Scales That Increase in Stages

In cases wherein a breakwater is designed to serve in an area where it is exposed to gentle waves, such as Breakwater A, and tsunamis of the largest scale are postulated as tsunamis that can exceed the scale of the design tsunami for the breakwater in consideration of its importance and other characteristics, it will cost a lot of money to construct standard levee widening work or other large-scale work as additional measures for the breakwater. In such a case, it must be noted that introducing a newly developed technology may be more economically efficient than constructing standard levee widening work because the new technology may cost significantly in terms of the initial investment for additional measures against tsunamis that are slightly larger than the design tsunami but may not cost much in terms of the overall investment for additional measures against tsunamis that have the largest scales.

7.5.4 Verification of Effectiveness of the Highly Durable Structure of a Breakwater

To verify the effectiveness of the highly durable structure of a breakwater, it is necessary to appropriately evaluate the actual modes of deformation and verify the stability of the breakwater against deformation. In particular, for the upright wall and the rubble mound, the modes of deformation in various situations should be appropriately evaluated, for example, where levee widening work is constructed, where seepage flow affects deformation, where the structure is reinforced with steel pipe piles or other pipes, and where rubble is covered with friction enhance mats or other mats.

As a convenient and indirect way of evaluating the high durability of a structure, it is possible to examine how much margin for the tsunami wave force is given to the structure in consideration of the passive resistance of levee widening work and other factors by using the equations given for the verification of the stability of the upright wall against sliding or overturning and for the bearing capacity of the rubble mound and calculating the safety factor against sliding as one of the criteria for high durability. If the safety factor against sliding is larger than 1.0, the structure can be considered highly durable. When verifying the high durability in this way, the effectiveness of specific measures should be verified, including countermeasures against scouring, on the basis of hydraulic model tests and numerical analyses.

For the final step, it is necessary to comprehensively evaluate from various points of view the scales of tsunamis that the breakwater can resist without collapsing and while maintaining its highly durable structure. This can be performed by carefully examining the effectiveness of measures by using hydraulic model tests and numerical analyses and by considering the importance and cost-effectiveness of the facility.

7.6 Effectiveness of Tsunami Protection Breakwaters in Mitigating the Impact of Tsunamis and Delaying the Rise in Water Level

The effectiveness of the tsunami protection breakwater in Ofunato Bay in mitigating the impact of a tsunami was evaluated by analyzing the harbor resonance in the bay on the basis of the tide level recorded in the earthquake off the coast of Tokachi (in May 1968). **Fig. 7.6.1** shows the results of the comparison of data measured before and after the construction of the tsunami protection breakwater wherein the wave height amplification ratio M is the ratio of the amplitude in the inner part of the bay to the amplitude of incident waves. For long-period oscillations in low-order modes, the wave height amplification ratio after the construction is significantly lower than that before the construction, thus indicating that the tsunami protection breakwater demonstrated effectiveness in mitigating the impact of the tsunami.[14)] The effectiveness of this tsunami protection breakwater was also verified by Ito et al.[1)] on the basis of numerical calculations.

In the earthquake off the Pacific coast of Tohoku (in March 2011), the breakwater at the mouth of Kamaishi Bay reduced the tsunami height by 40% and the maximum wave run-up height by 50% and delayed tsunami wave overt flowing of the seawall by 6 minutes, thus demonstrating effectiveness in delaying the rise in water level.[15)]

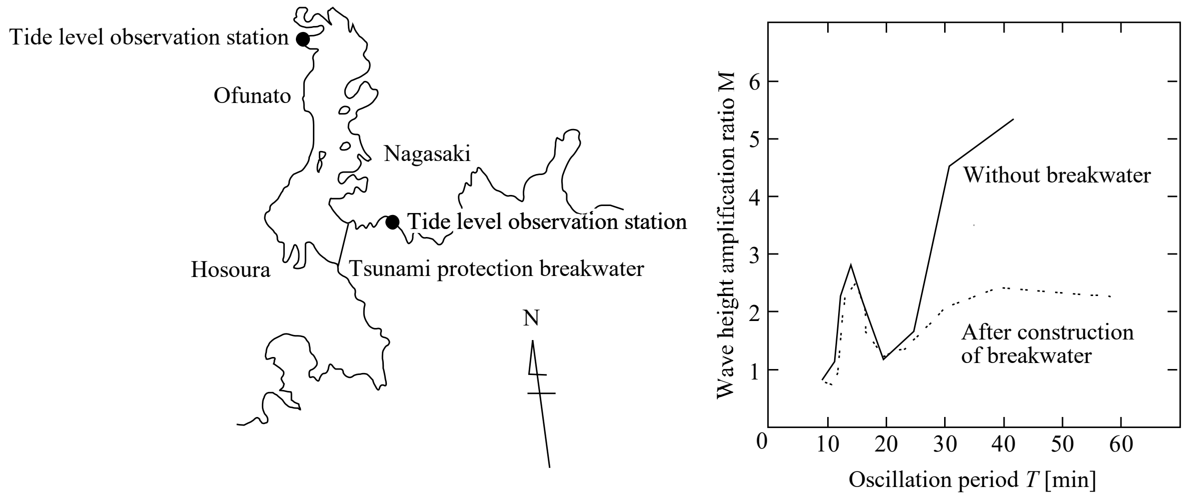


Fig. 7.6.1 Effectiveness of the Tsunami Protection Breakwater in Ofunato Bay

7.7 Others

(1) An experimental study by Tanimoto et al.[16] has verified that in a situation wherein a tsunami flows into a port or harbor through a narrow entrance, the tsunami flows at an increased velocity while generating vortices. This situation significantly affects the stability of the armor material of the mound of a submerged breakwater. Tsunamis also exert strong tractive forces on the bed that are said to be even greater than those by storm surges. Therefore, the upright walls and the foundation ground at the entrance of a port or harbor should be adequately reinforced (e.g., by increasing the stability of the upright walls and preventing scouring of the foundation ground).

(2) Given that the rubble mound becomes thicker as the water becomes deeper, it is necessary to pay careful attention to the stability of the rubble mound against wave forces and to the wave transformation on the slope surface of the rubble mound. It is also necessary to carefully determine dimensions, such as the height of the extra-banking of rubble, with consideration to the increased compression of the mound.

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8 Breakwaters for Timber Handling Facilities

8.1 General

Breakwaters for timber handling facilities shall comply with **Part III, Chapter 4**, **3 Ordinary Breakwaters** with modifications made, as necessary in consideration with the structural type, and their performance verification may be performed as follows:

8.2 Actions

(1) Generally, timber handling facilities, such as timber storage ponds and sorting ponds, are located in the innermost part of a port or harbor; therefore, they are not exposed to high waves. Unlike ordinary breakwaters, breakwaters for such facilities are mainly intended to prevent timbers from drifting out. Therefore, it is advisable to examine not only the force of waves but also the force that occurs when timbers collide with a breakwater under the effect of the wind, tidal current, and/or waves, depending on the situation.

(2) The colliding force of timbers has not been clarified yet. While conducting performance verification, refer to records and data of past cases.

8.3 Setting of Basic Cross Section

(1) The crown height of a breakwater for timber handling facilities should be set to an appropriate height in consideration of the structure of the breakwater, the usage of the water area behind it, and other factors to ensure that timbers do not drift out under conditions such as an abnormal tide level. It is preferable that the crown height is higher than the abnormal tide level by approximately 60% of the design significant wave height.

(2) In most cases, harbor calmness behind a breakwater is controlled to keep the wave height at approximately 50 cm even in rough weather.

8.4 Structural Details

(1) To prevent timbers from drifting out of a sorting pond, it is preferable to construct fences, as needed, to prevent timber drifting or piles to moor timbers.

(2) Fences to prevent timber drifting should have the crown height and pile interval appropriate for preventing timbers from drifting out and should be equipped with a superstructure, as needed.

(3) It is necessary that fences to prevent timber drifting and piles to moor timbers have structures that can resist the colliding or tractive force of timbers caused by winds, tidal currents, and waves. Additionally, it is preferable to consider wave forces and other forces depending on the situation.

9 Sediment Control Groins

[Ministerial Ordinance] (Performance Requirements for Sediment Control Groins)

Article 15

1 The performance requirements for sediment control groins shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied for the purpose of mitigating siltation in waterways and basins due to littoral drift through effective control of littoral drift.

2 The provisions of Article 14, paragraph (1), item (ii) shall apply mutatis mutandis to the performance requirements for sediment control groins.

[Public Notice] (Performance Criteria of Sediment Control Groins)

Article 38

1 The provisions of Article 35 or Article 36 shall apply mutatis mutandis to the performance criteria of sediment control groins in consideration of the structural type.

2 In addition to the provisions of the preceding paragraph, the performance criteria of sediment control groins shall be such that the facilities are located appropriately so as to control littoral drift, in consideration of the environmental conditions, etc. to which the facilities are subjected, and shall have the dimensions necessary for the functions of the sediment control groins.

[Interpretation]

10. Protective Facilities for Harbors

(6) Performance Criteria for Sediment Control Groins (Article 15 of the Ministerial Ordinance and the interpretation related to Article 38 of the Public Notice)

① Performance criteria of gravity-type breakwaters and pile-type breakwaters and their interpretations shall be applied correspondingly to sediment control groins with modifications made as necessary in consideration of the structural type.

② In the performance verification for sediment control groins, appropriate consideration shall be given to the effects of an increase in earth pressure due to sedimentation of littoral drift and the effects of river currents, as needed, in addition to the previous paragraph.

9.1 General

(1) A breakwater in a harbor surrounded by a sand beach also serves as a sediment control groin, and as a result, it is impossible to separate these functions. Therefore, in this section, breakwaters are referred to simply as "breakwaters," except when their sediment control function is especially important.

(2) Layout of Sediment Control Groins

① Sediment control groins shall be appropriately located by considering the characteristics of littoral drift so as to fulfill the expected sediment control function.

② A sediment control groin on the updrift side of longshore sediment transport shall run perpendicularly to the shoreline in shallow water including the surf zone. In deep water beyond the surf zone, the groin shall be located so that it can disperse littoral drift to the side opposite to the harbor entrance.

③ When a sediment control groin is constructed on the downdrift side of longshore sediment transport in order to prevent littoral drift from being carried into the harbor from the shore on the downdrift side, the groin shall run perpendicularly to the shoreline in principle and shall also have an appropriate length considering wave direction and wave transformation. However, in cases where a sediment control groin also functions as a breakwater, it shall be located appropriately considering its required functions as a breakwater.

④ If there is a need to provide a sediment control groin in the vicinity of a waterway inside a harbor, it shall be constructed in an appropriate location in consideration of the environmental conditions.

(3) Layout of Updrift Side Breakwater

It is preferable that the updrift side breakwater is extended perpendicularly to the shoreline beyond the surf zone or across the wave breaking line in order to prevent longshore sediment transport and allow sediment to deposit on the updrift side of the breakwater (refer to **Fig. 9.1.1**). When this part extending from the shore is short or deflected from the line perpendicular to the shoreline toward the downdrift side, the capability of the breakwater to catch sediment on the updrift side is reduced and sediment can easily move along the deflected part of the breakwater toward the harbor entrance. When this part is deflected from the line perpendicular to the shoreline toward the downdrift side, it is likely to become the cause of local scouring on the shore on the updrift side.[1)] In deep water beyond the wave breaking line, the breakwater shall be deflected so that it can block waves as a breakwater, and at the same time, disperse littoral drift toward the opposite side of the harbor entrance with the aid of reflected waves or Mach-stem waves (refer to **Fig. 9.1.1**).

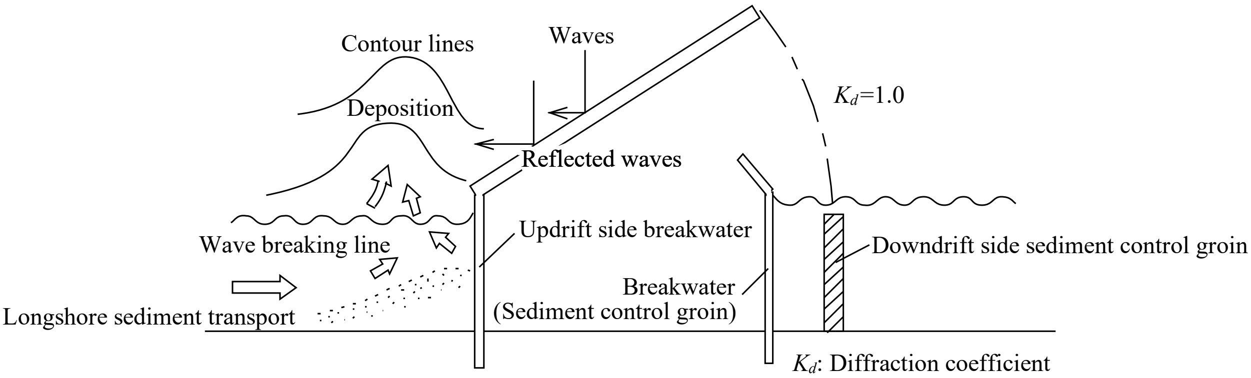
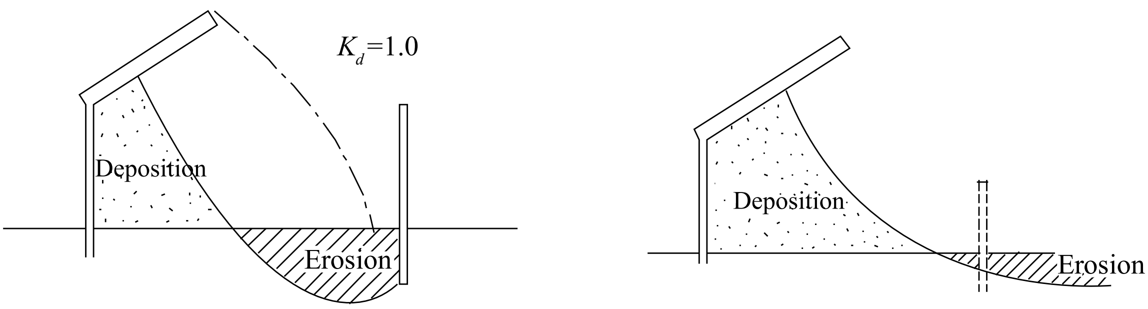


Fig. 9.1.1 Conceptual Diagram of Layout of Breakwater (Sediment Control Groin)

(4) Positioning and Construction Time of Downdrift Side Breakwater

When the updrift side breakwater is extended beyond the extension line of the downdrift side breakwater that runs perpendicular to the shoreline, this will allow sediment to deposit on the downdrift side of the latter breakwater, resulting in formation of a sandbar extending from the shore on the downdrift side toward the harbor entrance and erosion of the shore on the downdrift side.[2)] If the downdrift side breakwater is extended before the deflected part of the updrift side breakwater is extended to a sufficient length, significant local erosion may occur on the harbor side of the downdrift side breakwater, as shown in **Fig. 9.1.2 (a)**. Conversely, if the extension work of the downdrift side breakwater is delayed, this may often cause sedimentation in the harbor and erosion of the shore on the downdrift side as shown in **Fig. 9.1.2 (b)**. The closest attention should therefore be paid to the speeds of extending the updrift and downdrift side breakwaters, and care must be taken to maintain the appropriate balance between the lengths of the breakwaters.



(a) Case with rapid extension of the downdrift side breakwater

(b) Case with slow extension of the downdrift side breakwater

Fig. 9.1.2 Construction Time of Downdrift Side Breakwater

(5) Length of the Breakwater and Water Depth at the Tip

Longshore sediment transport occurs mainly in the surf-zone so it is necessary to extend the breakwater to a point offshore beyond the surf zone. In small ports where the tip of the breakwater is within the surf zone during stormy weather, it is difficult to completely prevent littoral drift from entering the port. In major ports in Japan, it is common for the water depth at the tip of an updrift side breakwater to be approximately equal to the maximum depth of the navigation channels in the port concerned.

Fig. 9.1.3 shows case examples of sediment control groins that work effectively as ancillaries. **Fig. 9.1.3 (a)** shows a case in which sediment control groins serve to prevent sand from entering the waterway from both sides. **Fig. 9.1.3 (b)** shows a case in which a sediment control groin (i) serves to increase the capability of blocking littoral drift on the updrift side, and a sediment control groin (ii) serves to allow incoming sediment to deposit on the natural beach on the right side.3)

Even if a very long breakwater is constructed, it is hard to completely prevent sediment carried by the water flow along the breakwater from going around the tip of the breakwater and into the harbor. When a basin or waterway is located behind a breakwater, it requires some maintenance dredging. Therefore, it is preferable to determine the most economically efficient length of the breakwater in consideration of such maintenance dredging.

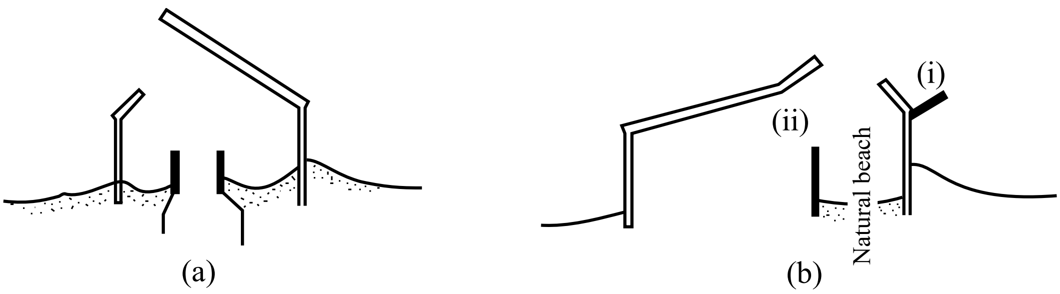


Fig. 9.1.3 Case Examples of Sediment Control Groins Provided as Ancillaries3)

(6) Structural Forms of Sediment Control Groins

Sediment control groins should have impermeable structures because they are expected to stop sediment transport completely. Where a rubble-mound or concrete-block structure is adopted to build the landward end of a sediment control groin, it may be filled with quarry run or rubble of up to 100 to 200 kg; there are also cases where sediment infiltration prevention work is constructed, as needed, on the harbor side of the sediment control groin by using impermeable materials such as sand mastic asphalt. In the following situations, it is preferable to additionally adopt a wave-absorbing structure.

① When there is significant concern about scouring by currents

② When there are concerns about reflected waves that can cause siltation or obstruction to the navigation of ships

9.2 Performance Verification

(1) For the performance verification of sediment control groins, refer to provisions concerning composite breakwaters given in **Part III, Chapter 4, 3.1.4 Performance Verification for Overall Stability of Breakwater Body**," as well as provisions concerning the performance verification for each structural type. Note, however, that it is necessary to appropriately consider the effects of an increase in earth pressure due to sedimentation of littoral drift.

(2) Crown Height of Sediment Control Groins

Although it is preferable that sediment control groins not allow waves to overtop them in order to prevent waves from carrying suspended sediment into the harbor, there are also cases where overtopping is allowed due to structural constraints, economic efficiency and other reasons. In principle, the crown height of each part of a sediment control groin should be determined by taking the following into consideration:

① Part near the landward end of a sediment control groin

It is preferable that the crown height of a sediment control groin at the part near the landward end be high enough to prevent run-up waves from overtopping it. Because sand carried by run-up waves may overtop the crown of the sediment control groin at the landward end, the crown should be sufficiently high. It is preferable to raise the crown height or extend the groin itself to the landward direction, in light of the conditions after construction.

② **Part on the landward side of the wave breaking line**

The crown height of the sediment control groin at the part on the landward side of the wave breaking line may be set to 0.6 $H_{1/3}$ above the mean monthly-highest water level (HWL), where $H_{1/3}$ is the significant wave height around the tip of the sediment control groin.

③ **Part on the seaward side of the wave breaking line**

The crown height of the sediment control groin at the part on the seaward side of the wave breaking line may be determined by adding a certain margin to the mean monthly-highest water level. In deep water offshore beyond the surf zone, the suspended sediment is concentrated near the seabed and the overtopping water contains only small amount of sediment, and therefore overtopping may be allowed in most cases.

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10 Seawalls

[Ministerial Ordinance] (Performance Requirements for Seawalls)

Article 16

1 The performance requirements for seawalls shall be as prescribed respectively in the following items so as to protect the hinterland of the seawall with modifications made as necessary in consideration of the structural type:

(1) Seawalls shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable the protection of the hinterland of the seawalls from waves and storm surges.

(2) Damage to seawalls, etc. due to self-weight, earth pressure, variable waves, Level 1 earthquake ground motions, etc. shall not impair functions of the seawalls and shall not adversely affect the continueous use of the seawalls.

2 In addition to the provisions of the preceding paragraph, the performance requirements for seawalls in the place where there is a risk of serious impact on human lives, property or socioeconomic activities shall be as prescribed in the following items with modifications made as necessary in consideration of the structural type.

(1) The performance requirements for seawalls which are required to protect the hinterland of the seawalls from design tsunami or accidental waves shall be such that the seawalls satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable the protection of hinterland of the seawalls from design tsunami or accidental waves.

(2) Damage to seawalls, etc. due to design tsunamis, accidental waves, Level 2 earthquake ground motions, etc. shall not have a serious impact on the structural stability of the seawalls, even in cases where functions of the seawalls are impaired. Provided, however, that for the performance requirements for a seawall which requires further improvement of the performances due to the environmental, social conditions, etc. to which the seawall is subjected, the damage due to the actions, etc. shall not adversely affect the restoration through minor repair work of the functions of the seawalls.

3 In addition to the provisions of the preceding two paragraphs, the performance requirements for breakwaters in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity, shall be such that a serious impact on the structural stability of the breakwaters caused by damage, etc. due to the actions of the tsunamis, even in cases where tsunami with intensity exceeding the design tsunami occurs at a place at which the breakwaters are installed, shall be delayed as much as possible.

[Public Notice] (Performance Criteria of Seawalls)

Article 39

1 The provisions concerning the structural stability in Article 49 through Article 52 (excluding the provisions concerning ship berthing and traction by ships) shall apply mutatis mutandis to the performance criteria of seawalls with modifications made as necessary in consideration of the structural type.

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

(1) Seawalls shall be located appropriately so as to enable the control of wave overtopping in consideration of the environmental conditions, etc. to which the facilities are subjected, and shall have the dimensions necessary for the function of the seawalls.

(2) Under the variable situation, in which the dominating action is water pressure, the risk of losing stability due to seepage failure of the ground shall be equal to or less than the threshold level.

(3) For structures with parapets, the risk of sliding and overturning of the parapet under the variable situation, in which the dominating actions are variable waves and Level 1 earthquake ground motion, shall be equal to or less than the threshold level.

3 In addition to the provisions of the preceding two paragraphs, the performance criteria of seawalls to which damage might significantly affect human lives, property or socioeconomic activity shall be as prescribed respectively in the following items:

(1) Seawalls which are required to protect the hinterland of the seawalls from design tsunamis or accidental waves shall have the dimensions necessary for the protection of the hinterland from tsunamis or accidental

waves.

(2) The degree of damage under the accidental situation, in which the dominating actions are design tsunamis, accidental waves, or Level 2 earthquake ground motion, shall be equal to or less than the threshold level in consideration of the performance requirements.

[Interpretation]

10. Protective Facilities for Harbors

(7) Performance Criteria for Seawalls

① **Common performance criteria for seawalls** (Article 16, paragraph 1, item 2 of the Ministerial Ordinance and the interpretation related to Article 39, paragraph 2, item 2 and 3 of the Public Notice)

(a) Serviceability shall be the required performance for seawalls under variable situations in which the dominating actions are water pressure, variable waves or level 1 earthquake ground motions. Performance verification items for those actions and standard indexes for setting limit values shall be in accordance with **Attached Table 10-7**.

Attached Table 10-7 Performance Verification Items and Standard Indexes for Setting Limit Values for Seawalls (Excluding Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
16	1	2	39	2	2	Serviceability	Variable	Water pressure	Self-weight	Seepage failure of ground	—
					3			Variable waves [level 1 earthquake ground motion]	Self-weight, earth pressure, water pressure	Sliding or overturning of parapet* 1)	Action-to-resistance ratio for sliding Action-to-resistance ratio for overturning

Note: The action shown in brackets is an alternative dominating action.
*1): Limited to structures having a parapet.

(b) **Attached Table 10-7** shows no particular index for setting the limit value for seepage failure of the ground, so it is necessary to appropriately set the index when conducting the performance verification of a seawall for seepage failure.

(c) In addition to these provisions, the provisions of the Public Notice, Article 22, Paragraph 3 (Scouring and Sand Washing Out) and Article 28 (Performance Criteria of Armor Stones and Blocks) and their interpretation shall be applied to performance criteria of seawalls, as appropriate, and Article 23 through Article 27 shall also be applied, depending on the types of members comprising each seawall.

② **Seawalls serving as facilities prepared for accidental incidents** (the Ministerial Ordinance, Article 16, Paragraph 2, Item 2 and Paragraph 3, and the interpretation related to the Public Notice, Article 39, Paragraph 3, Item 2)

(a) Safety and restorability shall be the required performance for seawalls serving as facilities prepared for accidental incidents under accidental situations in which the dominating actions are level 2 earthquake ground motions, design tsunami or accidental waves, depending on the functions required for such seawalls. Performance verification items for those actions and standard indexes for setting limit values shall be in accordance with **Attached Table 10-8**. This table uses the non-specific term "damage" to describe the verification item because there will be different verification items depending on the structural type. When conducting the performance verification for this verification item, it is necessary to appropriately set a specific index for setting the limit value.

Attached Table 10-8 Performance Verification Items and Standard Indexes for Setting Limit Values for Seawalls Serving as Facilities Prepared for Accidental Incidents (Only Under Accidental Situations)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Standard index for setting limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
16	2	2	39	3	2	Safety, restorability	Accidental	Level 2 earthquake ground motion [Design tsunami] [Accidental waves]	Self-weight, earth pressure, water pressure	Damage	–

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

(b) In the performance verification of a seawall serving as a facility prepared for accidental incidents, the limit value of the degree of damage to the seawall under accidental situations in which the dominating actions are level 2 earthquake ground motions, design tsunami or accidental waves shall be set by giving comprehensive consideration to not only the functions of the seawall, but also the state of construction and maintenance of facilities serving to protect the land area behind the seawall, as well as intangible measures to reduce and prevent possible damage to the area. In cases where a seawall serves as a facility prepared for accidental incidents and its performance requirement is restorability, the limit value of the degree of damage shall be set by giving appropriate consideration to the allowable restoration period.

(c) When conducting the verification of a seawall serving as a facility prepared for accidental incidents in terms of performance for design tsunami, it is necessary to consider the effects of the action of earthquake ground motions preceding the tsunami. When doing so, it should be noted that the earthquake ground motions preceding the postulated design tsunami are not necessarily identical with the level 2 earthquake ground motions.

(d) In addition to these provisions, the provisions of the Public Notice, Article 22 (Common Performance Criteria for Members Comprising Facilities Subject to the Technical Standards) and their interpretation shall be applied to performance criteria of seawalls serving as facilities prepared for accidental incidents concerning their performance under accidental situations.

(e) A seawall serving as a facility prepared for accidental incidents shall have a structure designed to maintain its stability as long as possible in order to ensure that the seawall demonstrates effectiveness in reducing possible damage even when the seawall is exposed to the action of a tsunami of which the intensity, at the site where the seawall is located, is higher than that of the design tsunami.

10.1 General

(1) The purpose of seawalls is to protect the land areas behind them from waves, storm surges and tsunamis.

(2) The provisions of this section may also apply to revetments, coastal dikes, parapets, floodgates, locks and land locks that compose a tide barrier system.

(3) Seawalls based on the technical standards include both public and private facilities, unlike shore protection facilities, which are public facilities under the **Technical Standards for Shore Protection Facilities**. Therefore, it must be noted that the level of protection to be provided by a seawall may differ from the level of protection to be provided by shore protection facilities as specified by the seashore administrator depending on the importance of the posterior facilities that need to be protected by the seawall.

(4) Seawalls shall be designed in accordance with this document and by reference to the **Technical Standards and Commentary for Shore Protection Facilities**[1], the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Ports and Harbors**[2], the **Design Concept for Parapets for Tsunamis (Provisional Edition)**[3], the **Guidelines for Tsunami-Resistant Design of Breakwaters**[4] and the **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores.**[5] If the facility of interest is a shore protection facility, it shall conform to the **Technical Standards for Shore Protection Facilities**.

10.2 Layout

(1) Seawalls shall be located appropriately, in consideration of future plans of the port or harbor concerned, to ensure that they can protect the land areas behind them from waves, storm surges and tsunamis, and will obstruct as little as possible the physical distribution, traffic and other movements inside and outside the port or harbor.

(2) It is possible to choose to build seawalls alone or to build both seawalls and breakwaters as facilities that protect land areas behind them in a port or harbor.

10.3 Setting of Basic Cross Section

(1) The basic cross section of a seawall shall be set in accordance with **Part III, Chapter 4, 14 Revetments**, **Part III, Chapter 4, 15 Coastal Dikes and Part III, Chapter 4, 17 Parapets** with modifications made as necessary in consideration of the structural type.

(2) The required crown height of a seawall shall be determined according to the performance requirements of the facility and by giving appropriate consideration to waves, tidal levels (including during storm surges), tsunamis, settlement after construction (consolidation settlement and settlement due to earthquake ground motions) and other conditions in the location where it will be constructed. For waves, tidal levels and tsunamis, refer to **Part II, Chapter 2, 4 Waves**, **Part II, Chapter 2, 3 Tidal Level** and **Part II, Chapter 2, 5 Tsunamis**, respectively.

10.4 Actions and Performance Verification

(1) Seawalls shall be designed by appropriately setting actions and design situations to be considered as well as performance criteria in accordance with the performance requirements for the facilities.

(2) For the performance verification of seawalls, refer to the following descriptions:

① **Wave overtopping rate and permissible rate of wave overtopping**

When setting the layout and dimensions of a seawall (its structure and cross-sectional dimensions and ancillary facilities) in the performance verification, it is necessary to appropriately verify that the wave overtopping rate will not exceed the permissible rate of wave overtopping. When evaluating the wave overtopping rate in the performance verification of the seawall, it is necessary to appropriately consider environmental conditions to which the seawall will be subjected and its structural characteristics. When setting the permissible rate of wave overtopping in the performance verification of the seawall, it is necessary to appropriately consider the density of houses, public facilities and other buildings in the land area behind the seawall and the conditions of use of those buildings, as well as the capacities of drainage facilities in the land area behind the seawall.

② **Wave force and hydrostatic pressure acting on a seawall**

The wave force acting on a seawall shall be set appropriately by reference to **Part II, Chapter 2, 6 Wave Force**. The water pressure acting on a wall body, such as a parapet, needs to be set appropriately in consideration of the simultaneous actions of the increased water pressure due to a storm surge and the wave pressure caused by waves by referring to **Part II, Chapter 2, 6.2.10 Wave Force and Hydrostatic Pressure during a Storm Surge (When the Tide Level is High)**."

③ **Effects of settlement caused by level 1 earthquake ground motions**

Seawalls are required to fulfill the function of appropriately controlling wave overtopping even after ground settlement. Therefore, when evaluating the wave overtopping rate in the performance verification of a seawall, it is necessary to appropriately consider the effects of ground settlement caused by the action of level 1 earthquake ground motions.

④ **Ancillary facilities**

In the performance verification of a seawall, it is necessary to appropriately consider ancillary facilities, including apron work, drainage ditches, drainage holes and drainage facilities that are provided for protecting the land area behind the seawall from wave overtopping and getting flooded, in order to ensure that the land area behind the seawall can be protected appropriately from waves and storm surges.

⑤ **Measures to prevent washing-out**

In the performance verification of a seawall, it is necessary to pay attention to the prevention of washing-out of backfill soil behind the seawall in consideration of the structural type. It is also necessary to take measures to prevent washing-out of backfill soil, for example, by placing sand invasion prevention sheets or sand invasion prevention plates, as appropriate.

(3) For the performance verification of seawalls serving as facilities prepared for accidental incidents, refer to the following descriptions:

① **Common items**

Performance criteria for specifications common to all seawalls as described above in (2), ① through ⑤, shall be applied to seawalls serving as facilities prepared for accidental incidents, except for postulated environmental conditions such as level 2 earthquake ground motions, tsunamis and accidental waves.

② **Setting of the limit value of the degree of damage to seawalls under accidental situations**

When setting the limit value of the degree of damage to a seawall under accidental situations, it is necessary to give comprehensive consideration to not only the functions of the seawall, but also to the state of construction and maintenance of floodgates and other harbor protective facilities that compose a tide barrier system and other peripheral facilities, as well as intangible (non-structural) measures to reduce and prevent possible damage in the area concerned.

③ **Accidental situation in which the dominating action is a design tsunami**

(a) Stability against design tsunami and tsunamis with intensity higher than that of the design tsunami

For evaluating the stability against design tsunami and tsunamis with intensity higher than that of the design tsunami in order to develop a tsunami-resistant design, refer to the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Ports and Harbors**.[2)]

(b) Consideration of the effects of earthquake ground motions

In the verification of performance of a facility for design tsunami, it is necessary to appropriately consider the possibility that, when the postulated design tsunami is triggered by an earthquake of which the seismic center is close to the facility, the facility may be subjected to the action of earthquake ground motions caused by the earthquake before it is subjected to the action of the design tsunami. In view of this, it is necessary to consider the effects of the action of earthquake ground motions preceding the design tsunami when conducting the verification of performance for the design tsunami. When doing so, it must be noted that the earthquake ground motions preceding the postulated design tsunami are not necessarily identical with the level 2 earthquake ground motions.

④ **Accidental situation in which the dominating action is accidental waves**

(a) Conditions of accidental waves

Conditions of accidental waves shall be set appropriately by reference to **Part II, Chapter 2, 4 Waves** and **part II, Chapter 2, 3 Tidal Level**."

(b) Consideration of the effects of storm surges

In the verification of performance of a facility for accidental waves, it is necessary to appropriately consider a storm surge that occurs at the same time as the postulated waves. For setting the conditions of storm surges, refer to **Part II, Chapter 2, 4.1.1 Setting of Wave Conditions to be Used for Verification of Stability of Facilities and Safety (for Cross-Sectional Failure) of Structural Members**, **Part II, Chapter 2, 3.2 Storm Surges** and **Part II, Chapter 2, 3.6 Design Tide Level Conditions**.

(c) Design storm surges

In the performance verification of facilities prepared for accidental incidents, design storm surges shall be set appropriately in consideration of storm surges up to the largest class. For setting a design storm surge, refer to **Part II, Chapter 2, 4 Waves** and **Part II, Chapter 2, 3 Tidal Level**.

[References]

1) National Association of Agricultural Sea Coast, National Association of Fisheries Infrastructure, National Association of Sea Coast, Ports and Harbours Association of Japan: Technical Standards and Commentary for shore Protection Facilities, 2018 (in Japanese)
2) MLIT, Ports and Harbours Bureau: Guidelines for Earthquake-resistant Design of Storm Surge Barriers (Breast Walls) in Ports, 2013 (in Japanese)
3) Fisheries Agency, Fishery Infrastructure Department, Fishing Communities Promotion and Disaster Prevention Division and MLIT, Ports and Harbours Bureau, Coast Administration and Disaster Prevention Division: Design Concept of Breast Walls for Tsunami (Tentative Edition), 2015 (in Japanese)
4) MLIT, Ports and Harbours Bureau: Guideline for Tsunami-Resistant Design of Breakwaters (Partial Revision), 2015 (in Japanese)
5) Coastal Development Institute of Technology: Technical Manual for Flap-type Land Gates at Ports and Coasts, 2016 (in Japanese)

11 Training Jetties

[Ministerial Ordinance] (Performance Requirements for Training Jetties)

Article 17

1 The performance requirements for training jetties shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied for the purpose of mitigating siltation in waterways and basins and preventing the closure of river mouths due to littoral drift through effective control of littoral drift.

2 The provisions of Article 14, paragraph (1), item (ii) shall apply mutatis mutandis to the performance requirements for training jetties.

[Public Notice] (Performance Criteria of Training Jetties)

Article 40

The provisions of Article 38 shall apply mutatis mutandis to the performance criteria of training jetties.

[Interpretation]

10. Protective Facilities for Harbors

(8) Performance Criteria of Training Jetties (Article 17 of the Ministerial Ordinance and the interpretation related to Article 40 of the Public Notice)

① Performance criteria of sediment control groins and their interpretation shall be applied correspondingly to training jetties.

② In the performance verifications of training jetties, appropriate consideration shall be given to the effects of an increase in earth pressure due to sedimentation of littoral drift and the effects of waves and river currents, as needed, in addition to the previous paragraph.

11.1 General

(1) Layout and Shape of Training Jetties

Examples of the layout of training jetties in relation to the direction of longshore sediment transport are shown in **Fig. 11.1.1**.[1] The most preferable layout for maintaining the water depth at the mouth of a river is to extend two parallel training jetties because a single training jetty alone is not effective. Where two training jetties of different lengths are put in place, it is usually effective to make the training jetty on the downdrift side longer than the other. Bending the updrift side training jetty toward the downdrift side will prevent sediment from moving into the area between the two training jetties and allow smooth longshore sediment transport to the downdrift side. For actual examples of river mouth improvement, refer to **Reference 2)**.

(2) Water Depth at Training Jetty Tips

① The water depth at the tip of a training jetty should be equal to or greater than the water depth of the waterway in the vicinity of the training jetty.

② The tip of the training jetty should be located at a point where the water depth is equal to or greater than the wave breaking limit depth.

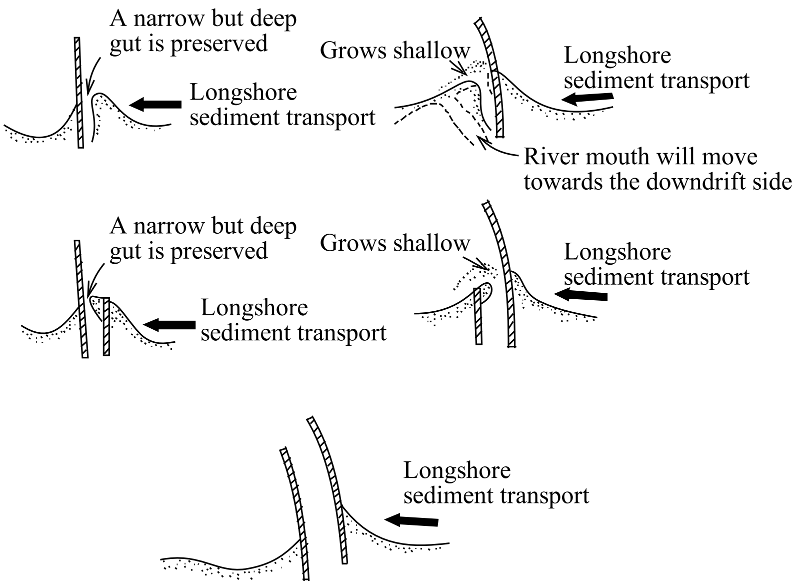


Fig. 11.1.1 Varieties of Training Jetty Layouts[1)]

11.2 Performance Verification

(1) For the performance verification of training jetties, refer to **Part III, Chapter 4, 3 Breakwaters Having Basic Functions**" in consideration of the structural type. Note, however, that it is necessary to give appropriate consideration to the effects of an increase in earth pressure due to sedimentation of littoral drift and the effects of waves and river currents.

(2) Training jetties are generally longer than groins and are exposed to intensive wave actions. Therefore, it is necessary to consider scouring at the tip and lateral sides of the training jetty. It is also necessary to consider that the river side of the training jetty will be subject to scouring actions of the river current.

[References]

1) JSCE: Handbook of Civil Engineering, (Vol. 2), pp.2268-2270, 1974 (in Japanese)

2) Uda, T., A. Takahashi and H. Matsuda: Nationwide classification of River Mouth Morphology and River Mouth Improvement, Technical Note of PWRI No. 3281, 123p., 1994. (in Japanese)

12 Floodgates

(English translation of this section from Japanese version is currently being prepared.)

12.1 General

(English translation of this section from Japanese version is currently being prepared.)

12.2 Setting of Layout and Dimensions of Floodgates

(English translation of this section from Japanese version is currently being prepared.)

12.3 Performance Verification of Floodgates

(English translation of this section from Japanese version is currently being prepared.)

13 Locks

(English translation of this section from Japanese version is currently being prepared.)

13.1 General

(English translation of this section from Japanese version is currently being prepared.)

13.2 Setting of Layout and Dimensions of Locks

(English translation of this section from Japanese version is currently being prepared.)

13.3 Structures and Performance Verification of Locks

(English translation of this section from Japanese version is currently being prepared.)

14 Revetments

[Ministerial Ordinance] (Performance Requirements for Revetments)

Article 20

1 The provisions of Article 16 shall apply mutatis mutandis to the performance requirements for revetments.

2 In addition to the provisions of the preceding paragraph, the revetments specified in the following items shall satisfy the performance requirements prescribed respectively in those items:

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

(2) "Performance requirements for revetments to be utilized by an unspecified large number of people" shall be such thatrevetments satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to secure the safety of users of the ports.

[Public Notice] (Performance Criteria of Revetments)

Article 43

1 The provisions of Article 39 shall apply mutatis mutandis to the performance criteria for revetments.

2 In addition to the provisions of the preceding paragraph, the revetments specified in the following items shall satisfy the performance criteria prescribed respectively in those items:

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

(2) "Performance criteria of revetments to be utilized by an unspecified large number of people" shall be such that revetments shall have the dimensions necessary to secure the safety of users of the ports and harbors depending on the environmental conditions, usage conditions, etc. to which the revetments are subjected.

[Interpretation]

10. Protective Facilities for Harbors

(11) Performance criteria of revetments

① **Symbiotic revetments** (Article 20, Paragraph 2, Item 1 of the Ministerial Ordinance and the interpretation related to Article 43, Paragraph 2, Item 1 of the Public Notice)

(a) Performance criteria of seawalls and their interpretation shall be applied correspondingly to revetments.

(b) Revetments intended for environmental conservation are referred to as "symbiotic revetments." In addition to the provisions applicable to revetments, the following items shall be applied to symbiotic revetments.

(c) Usability shall be the performance requirement for symbiotic revetments. The "usability" here denotes the performance of revetments in contributing to the conservation of environments of ports and harbors, including living things and ecological systems, without impairing the original functions of the revetments.

(d) The dimensions of revetments intended for environmental conservation denote their structures, cross-sectional dimensions, and ancillary facilities. When setting the structures and cross-sectional dimensions of revetments intended for environmental conservation in their performance verification and when installing their ancillary facilities, appropriate consideration shall be given to the factors that affect the capability of the revetments to fulfill the purpose to conserve environments of ports and harbors, including living things and ecological systems, without impairing the original functions of the revetments.

② **Amenity-oriented revetments** (Article 20, Paragraph 2, Item 2 of the Ministerial Ordinance and the interpretation related to Article 43, Paragraph 2, Item 2 of the Public Notice)

(a) Revetments to be utilized by an unspecified large number of people are referred to as “amenity-oriented revetments.” In addition to the provisions applicable to revetments, the following items shall be applied to amenity-oriented revetments.

(b) Usability shall be the performance requirement for amenity-oriented revetments. The “usability” here denotes the performance of revetments in securing the safety of their users depending on conditions, including the environmental conditions to which the revetments are subjected and the conditions of use of the revetments.

(c) The dimensions of amenity-oriented revetments denote their structures, cross-sectional dimensions, and ancillary facilities. When setting the structures and cross-sectional dimensions of amenity-oriented revetments in the performance verification and when installing their ancillary facilities, consideration shall be given to the effects of wave overtopping and spray, prevention of their users from slipping, overturning or falling into the water, and smooth rescue of users who have fallen into the water. Ancillary facilities such as fall prevention fences shall be installed appropriately.

14.1 General

(1) The purpose of revetments is to protect the land areas behind them from waves, storm surges, and tsunamis.

(2) For the performance requirements and performance criteria of revetments, refer to the descriptions about seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(3) Revetments to which damage might significantly affect human lives, properties, and/or socioeconomic activities and which are required to protect the land areas behind them from the design tsunami and accidental waves shall be given a purpose to protect the land areas behind them from the actions concerned above, in addition to the purpose mentioned in (1).

(4) Revetments shall be designed in accordance with this document and by reference to the **Technical Standards and Commentary for Shore Protection Facilities** [1], the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors** [2], the **Design Concept for Parapets against Tsunamis (Provisional Edition)** [3], the **Guidelines for Tsunami-Resistant Design of Breakwaters** [4] and the **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores** [5]. If the facility of interest is a shore protection facility, it shall conform to the **Technical Standards for Shore Protection Facilities**.

(5) This section covers ordinary reclamation revetments. For reclamation revetments for reclaimed areas used as final disposal sites for general waste (disposal sites defined in Paragraph 2, Article 5 of the **Order for Enforcement of the Waste Management and Public Cleansing Act**) or used as final disposal sites for industrial wastes (disposal sites defined in Item (14), Article 7 of the said order), the performance verification shall be conducted in accordance with **Part III, Chapter 11**, **2 Waste Disposal Seawalls**.

14.2 Items to be Considered in Setting of Basic Cross Section

(1) In setting the basic cross section of a revetment, the following items shall be examined in principle:

① The revetment shall have the crown height that ensures prevention of waves and storm surges from affecting preservation and use of the reclaimed land.

② Stability shall be secured against the actions of waves, earth pressure, and others.

③ The revetment shall have a structure that prevents leakage of the landfill soil.

④ Consideration shall be given to the effect on surrounding water areas, including prevention of outflow of turbid water during reclamation work.

⑤ When the revetment is an amenity-oriented revetment, it shall have a structure that allows people to use it in safe and comfortable ways.

(2) Generally, reclaimed areas are surrounded by revetments unless there are mooring facilities. Therefore, reclamation revetments should serve as stable earth retaining work that is stable against waves, prevents the landfill soil from leaking out, and protects the reclaimed areas behind them from wave overtopping and storm surges. Reclamation revetments facing the open sea are exposed to more severe conditions of waves and other actions than ordinary reclamation revetments and require careful examination of performance under those conditions.

(3) When setting the cross section of a revetment, it is necessary to appropriately consider its ancillary facilities, including apron work, drainage ditches, and drainage holes that are provided for protecting the land area behind the revetment from wave overtopping and drainage facilities that are provided for preventing the land area behind the revetment from getting flooded, to ensure that the revetment can adequately protect the land area behind it from waves and storm surges.

(4) When setting the basic cross section of a revetment, it is necessary to pay attention to prevention of washing-out of backfill soil behind the revetment body in consideration of the structural type. It is also necessary to take measures to prevent washing-out of backfill soil, for example, by placing sand invasion prevention sheets or sand invasion prevention plates, as appropriate. In particular, when setting the basic cross section of an amenity-oriented revetment, it is necessary to consider appropriate measures to prevent washing-out of backfill soil behind the revetment body, as appropriate.

(5) There are cases where a temporary reclamation revetment is constructed with a structure just enough to prevent backfill soil from leaking out during reclamation work and a final reclamation revetment or mooring facilities are constructed after the completion of reclamation work. Temporary revetments can be classified into the following types:

① Temporary revetment that has a structure built using low-cost materials and construction and is not intended to be used in the future

② Temporary revetment that is intended to be used as a final revetment in the future after reinforcement of its structure

Temporary revetments may be composed of wood fences, stone frames, and the like Rubble-mound breakwaters may be used as temporary revetments. There are temporary revetments with semi-permanent structures composed of light-weight steel sheet piles in place of wood fences or composed of corrugated cells. There are also cases where structural types generally used for final revetments are used for temporary revetments.

In the performance verification of a temporary revetment, it is necessary to appropriately set the safety level and the limit value of allowable deformation in consideration of the purpose of the revetment. When doing so, it is necessary to ensure that the temporary revetment has the required stability against waves to which it will be exposed before completion of a final revetment or quaywall. It is also necessary to determine the crown height to ensure that the revetment prevents waves and storm surges from affecting the reclaimed area before it is replaced by the final revetment or quaywall.

14.3 Points to Remember Concerning Land Reclamation and Construction of Revetments

(1) For land reclamation, refer to **Part III, Chapter 2**, **6 Land Reclamation**.

(2) For points to remember concerning land reclamation and construction of revetments, refer to the followings:

① For reclamation of soft clayey soil, it is necessary to take measures, such as backfilling of rubble, to reduce the earth pressure acting on the revetment and prevent the landfill soil from leaking out through joints or the foundation.

② In cases where landfilling work is done by a suction dredger and the foundation ground of a reclamation revetment has high permeability, there is a possibility that the soil of the foundation ground and the dumped soil might flow out because of surplus water and result in a failure of the revetment body and/or outflow of soils. Therefore, attention should be paid to these possibilities in the performance verification and construction of the revetment. In general, materials discharged by suction dredgers into reclaimed areas are in the form of slurry. Therefore, it is necessary to carefully determine the position of the opening of the discharge pipe of each suction dredger and the layout of spillways to ensure that the rear side of the revetment body will not be directly exposed to slurry flows.

③ In cases where a reclamation revetment is built adjoining to an existing land area, construction of the revetment may cause the groundwater level to rise or may result in deterioration of groundwater quality. Adequate attention should be paid to these possibilities when studying the reclamation layout plan and the revetment structure. It is preferable to investigate the conditions of the groundwater in the land area in advance. In addition, in cases where it is likely that reclamation revetment construction will cause deterioration of the groundwater quality, countermeasures such as construction of a cut-off wall must be considered.

④ In the case of reclamation where a large water area is enclosed by revetments, the opening through which seawater flows into and out of the area because of the tidal range becomes smaller with the progress of revetment construction, and a considerably rapid flow occurs at closing sections because of the difference in the water level between the inside and the outside of revetments. Therefore, careful consideration is required for the structure of revetments at the final closing section, which should have a cross section that ensures adequate stability of the structure against the expected flow speed. The flow velocities at closing sections are affected by the water area being closed, the cross-sectional areas of the closing sections, the average water depth, the tidal range, and other factors.

In the closing sections, it is preferable that consolidation work be conducted at a location with good ground before the flow velocity increases as work progresses. There are also cases in which a submerged weir or broad-crested weir is used depending on the flow velocities at the closing sections.

⑤ There are cases where a reclaimed area is partitioned depending on the sequence of land reclamation work and the reclamation method. Generally, there are no strict requirements for partitions in regard to waves, the crown height, the degree of prevention of soil leakage, the importance, and others. The performance verification of partitions may be conducted in the same manner as conducted for final or temporary revetments.

14.4 Setting of Crown Height of Revetment

(1) Setting of Crown Height

① The crown height of a revetment shall be set to an appropriate height in consideration of the wave overtopping rate, the tidal level during a storm surge, and other conditions so that the revetment can contribute to preservation of the reclaimed area behind it and prevent waves and storm surges from affecting the use of the revetment and the land area behind it.

② It is preferable to examine measures to reduce the wave overtopping rate of a revetment from a comprehensive point of view in consideration of waves, the tide level and other environmental conditions, the geographical features of the seabed around the revetment, the possibilities of future construction of detached breakwaters and submerged breakwaters in the water area in front of the revetment, the shape of the cross section of the revetment, the shape of the crest of the revetment (the possibility of future construction of a recurved parapet), and other factors.

③ For calculating the wave overtopping rate and the wave run-up height, refer to **Part II, Chapter 2**, **4.4.7 Wave Run-up Height, Wave Overtopping and Transmitted Waves (1)**.

④ The crown height of a revetment can be set by using the following method [6]:

(a) The required crown height h_d above the design high water level of the revetment can be set as follows, using the required crown height h_c above the water level commensurate with the importance of the land area behind the revetment, or the required crown height h_c' allowing for earthquake ground motions and the crest settlement d_s determined from the ground conditions such as consolidation.

$$h_d = \max(h_c, h_c') + d_s \qquad \textbf{(14.4.1)}$$

The required crown height h_c above the water level in **equation (14.4.1)** shall be a value obtained by adding a margin height to the calculated crown height for the design wave above the design high water level of the revetment.

(b) The required crown height h_c above the water level can also be calculated by setting the exceedance probability P for the permissible rate of overtopping. The exceedance probability P for the permissible rate of overtopping can be calculated using **equation (14.4.2)**. The mean value and the standard deviation of h_c/h_{cd} can be assumed to be 1.00 and 0.15, respectively. These values can be applied to revetments of any structural type and with any permissible rate of overtopping, because they were obtained from the results of a statistical analysis of 89 facilities based on the Study on the Dimensions of Embankment and Seawall [7], which provides data on existing embankments and seawalls all over Japan.

$$P = 1 - \int_0^z \frac{1}{\sqrt{2\pi}\, z\zeta} \exp\left\{ -\frac{1}{2}\left(\frac{\ln z - \lambda}{\zeta} \right)^2 \right\} dz$$

(14.4.2)

provided, however, that

$$z = \frac{h_c}{h_{c_d}}$$

where

P : exceedance probability of permissible rate of overtopping

h_c : required crown height above water level (m)

h_{cd} : calculated crown height for design wave above design high water level of revetment (m)

ζ : standard deviation of $\ln(h_c/h_{cd})$; given by $\zeta = \sqrt{\ln\left\{1 + \left(\frac{\sigma}{\mu}\right)^2\right\}}$

λ : mean value of $\ln(h_c/h_{cd})$; given by $\lambda = \ln\mu - \frac{1}{2}\zeta^2$

μ : mean value of h_c/h_{cd} (can be assumed to 1.00)

σ : standard deviation of h_c/h_{cd} (can be assumed to 0.15)

Equation (14.4.2) is shown graphically in **Fig. 14.4.1**. For example, assuming the exceedance probability of the permissible rate of overtopping is 0.01, the required crown height h_c above the water level, which is obtained by adding a margin height to the calculated crown height h_{cd}, is given as 1.40 times the calculated crown height h_{cd}.

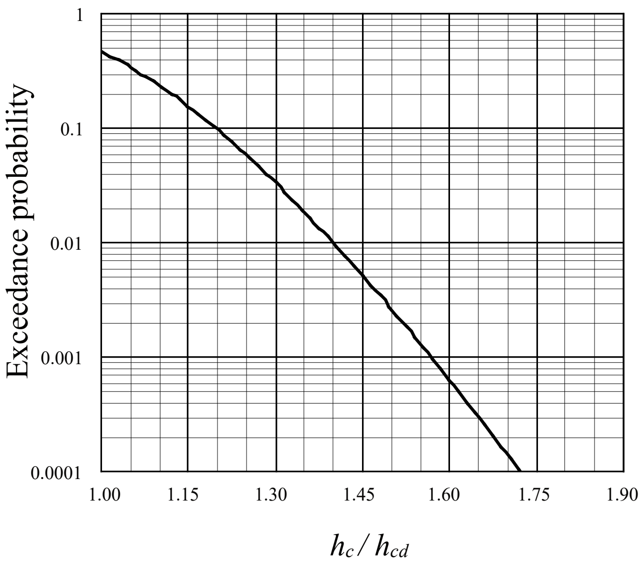
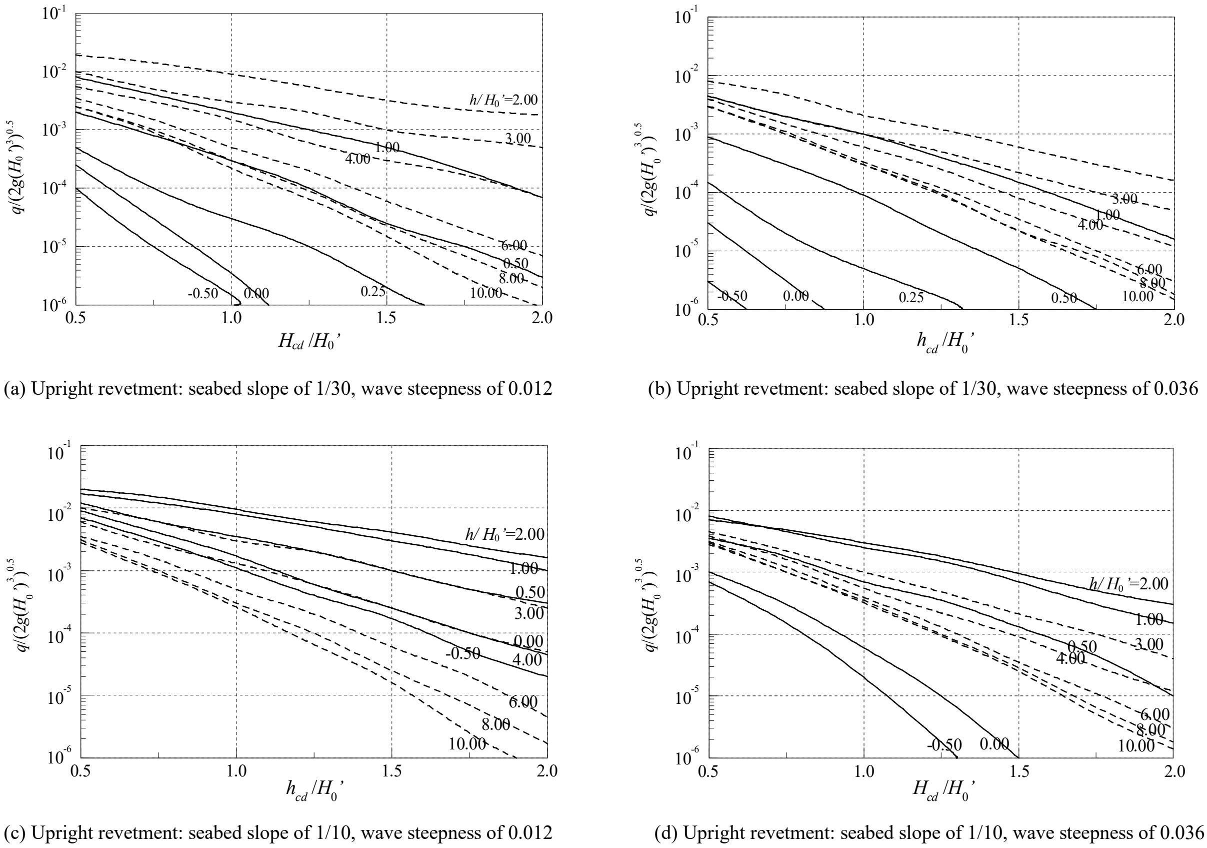


Fig. 14.4.1 Relationship of Exceedance Probability of Permissible Rate of Overtopping to h_c/h_{cd} (Required Crown Height above Water Level / Calculated Crown Height)

(c) The calculated crown height of a revetment shall be calculated as the crown height that satisfies the permissible rate of overtopping. For upright revetments and upright wave-absorbing revetments, this calculation shall be based on the diagrams for estimating wave overtopping rate given in **Part III, Chapter 2**, **4.4.7 Wave Run-up Height, Wave Overtopping and Transmitted Waves (2)**. For other types of revetments, this calculation shall be based on the diagrams for estimating allowable settlement that were drawn by Sekimoto et al.[8] The diagrams for estimating wave overtopping rate given in **Part III,**

Chapter 2, **4.4.7 Wave Run-up Height, Wave Overtopping and Transmitted Waves (2)** show the relationship of the non-dimensional wave overtopping rate to the ratio of the wave height to the water depth at the toe of the slope h/H_0'. Diagrams for calculating allowable settlement have been proposed [9], which show the relationship of the non-dimensional wave overtopping rate to the relative crown height h_{cd}/H_0' based on the diagrams for estimating wave overtopping rate. Those proposed diagrams can be used for easily obtaining the calculated crown height of a revetment (See **Figures 14.4.2** and **14.4.3**). The notation of those diagrams is similar to that of the diagrams for estimating wave overtopping rate given in **Part III, Chapter 2**, **4.4.7 Wave Run-up Height, Wave Overtopping and Transmitted Waves (2)**.



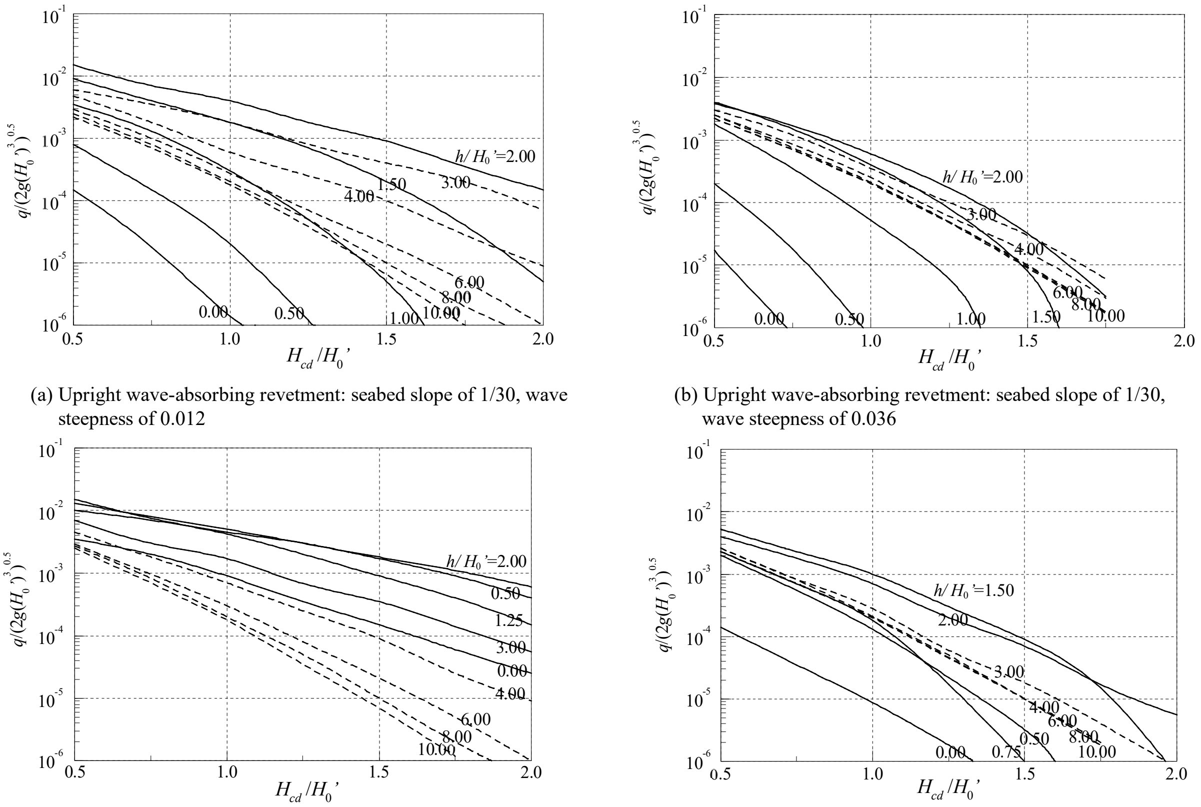
(a) Upright revetment: seabed slope of 1/30, wave steepness of 0.012

(b) Upright revetment: seabed slope of 1/30, wave steepness of 0.036

(c) Upright revetment: seabed slope of 1/10, wave steepness of 0.012

(d) Upright revetment: seabed slope of 1/10, wave steepness of 0.036

Fig. 14.4.2 Relationship of Non-dimensional Wave Overtopping Rate to the Relative Crown Height h_{cd}/H_0' (Uptight Revetments)



(a) Upright wave-absorbing revetment: seabed slope of 1/30, wave steepness of 0.012

(b) Upright wave-absorbing revetment: seabed slope of 1/30, wave steepness of 0.036

(c) Upright wave-absorbing revetment: seabed slope of 1/10, wave steepness of 0.012

(d) Upright wave-absorbing revetment: seabed slope of 1/10, wave steepness of 0.036

Fig. 14.4.3 Relationship of Non-dimensional Wave Overtopping Rate to the Relative Crown Height h_{cd}/H_0' (Uptight Wave-absorbing Revetments)

(d) The required crown height h_c' allowing for earthquake ground motions can be determined by adding the crest settlement because of the action of earthquake ground motions to the required crown height h_c above the water level. The required crown height h_c above the water level can be obtained by setting the return period for waves to be considered in the verification, considering the period required for recovery from damage caused by an earthquake, and determining the crown height that satisfies the permissible rate of overtopping of the said waves. For calculating the crest settlement because of the action of earthquake ground motions, refer to **Reference (Part III], Chapter 1**, **2 Fundamentals of Seismic Response Analysis**. The crest settlement because of earthquake ground motions can also be estimated from the horizontal deformation of a mooring facility of similar structural type. For example, for gravity-type revetments refer to reference 9).

(e) For the crest settlement d_s that can be determined from the ground conditions such as consolidation, refer to **Part III, Chapter 2**, **3.5 Settlement of Foundation**.

(f) For revetments that require consideration of accidental actions, including the design tsunami and accidental waves, the crown height required to resist these actions shall be considered in the first term of the right-hand side of **equation (14.4.1)**.

(g) When setting the crown height above the design high water level for a revetment, it is possible to include a margin height of up to 1 m above the required crown height h_d, as needed, considering uncertainties in design conditions and countermeasures against long-term sea level rise.[1)]

⑤ The crown height of a reclamation revetment may be reduced when wave-dissipating work is constructed on the front of the revetment. It must be noted, however, that wave overtopping might occur when the difference between the crown height of the revetment and the water level in the reclaimed area becomes smaller during reclamation work.

⑥ For measures to reduce wave overtopping rate of an upright or sloping revetment because of swelling waves, refer to reference 10) and other literature.

14.5 Actions

(1) For actions due to the tide level, refer to **Part II, Chapter 2**, **3 Tidal Level**.

(2) For actions due to waves, refer to **Part II, Chapter 2**, **4 Waves**.

(3) The wave force acting on a revetment shall be set appropriately by reference to **Part II, Chapter 2**, **6 Wave Force**. The water pressure acting on a wall body like a parapet on top of the revetment needs to be set appropriately in consideration of the simultaneous actions of the increased static water pressure caused by water level rise and the wave pressure caused by waves, by reference to **Part II, Chapter 2**, **6.2.10 Wave Force and Hydrostatic Pressure during Storm Surge (When the Tide Level is High)**.

(4) For the soil conditions of landfill soil, foundation ground and the like, refer to **Part II, Chapter 3 Geotechnical Conditions**.

(5) For actions because of earthquake ground motions, refer to **Part II, Chapter 6 Earthquakes**.

(6) For dynamic water pressure during the action of earthquake ground motions, refer to **Part II, Chapter 4**, **3.2 Dynamic Water Pressure**.

(7) As water levels in reclaimed areas, two water levels are generally set, these being the water level in the reclamation site and the residual water level. The water level in the reclamation site is used in seepage calculations and the performance verification of surplus water treatment facilities. The residual water level is the water level immediately behind a revetment and is mainly used in examination of the stability of the revetment. However, in cases where the water level at a position near the revetment is higher than the residual water level, using the residual water level in the examination of circular slip failure may result in underestimation of the danger of circular slip failure. In such cases, it is necessary to examine the stability of the revetment for the water level in reclamation site in addition to the stability for the residual water level.

① **Water level in a reclamation site**

The water level in a reclamation site shall be set by considering the stability of the revetment both during construction and after completion, and the influence on the surrounding waters. Regarding the influence on the surrounding waters, particular attention should be paid to overtopping flows due to waves generated behind the revetment during construction. It must be noted that, if the water level in the reclamation site is excessively high in comparison with the water level at the front of the revetment, an increased amount of water, including polluted water, may seep out of the revetment and its foundation ground.

② **Residual water level**

(a) Most reclamation revetments have low-permeable structures in order to reduce the seepage of polluted water out of reclamation sites. For this reason, the residual water level behind them is generally higher than that behind ordinary mooring facilities or revetments.

(b) In past construction projects of reclamation revetments with gravity-type structures, there were more cases in which permeability is reduced by increasing the layer thickness of the levee-widening earth or the backfilling sand than cases in which the permeability of the revetment body itself was reduced. For a revetment having a structure like this, the water level just behind the revetment body shows behavior similar to that behind the body of an ordinary gravity-type revetment. Therefore, the performance verification of the revetment body may be conducted by using the same residual water level as that used for ordinary gravity-type revetments.

(c) For reclamation revetments with sheet pile structures, there are cases where grout is poured into sheet pile joints or a double sheet pile structure is adopted to increase the water-tightness of sheet piles. In these cases, the residual water level behind a reclamation revetment tends to be higher than that behind an ordinary sheet pile revetment. For a reclamation revetment having a sheet pile structure like this, it is necessary to set an appropriate residual water level, giving due consideration to the water-tightness of the revetment and, for a double sheet pile structure, taking account of the crown height of sheet piles and construction conditions.

14.6 Performance Verification

14.6.1 Common Items

(1) For the performance verification of revetments, refer to the descriptions about seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(2) In case of reclamation using suction dredgers, there are cases in which suspended soft soil concentrates behind the revetment and the greater-than-expected earth pressure acts on the revetment body, and cases in which the action of the water pressure at the back side of the structure extends as far as the crest of the revetment. Therefore, it is necessary to give adequate consideration to these possibilities when conducting the performance verifications.

(3) In general, it takes a long time to reclaim land. Therefore, it is necessary to conduct the performance verification in consideration of various conditions during reclamation work. Particularly, when there is a possibility that circular slip failure might occur, the stability during reclamation work shall be examined for each cross section at each construction step. In a case where the reclamation site is subjected to a great action of waves, it is also necessary to examine the stability against waves during reclamation work by reference to **Part III, Chapter 4**, **3 Breakwaters Having Basic Functions**.

(4) In order to estimate the quantity of polluted water seeping out of a reclamation revetment into the sea, it is necessary to perform an analysis of seepage flows. In general, Darcy's law can be applied to seepage flow analysis. However, as will be discussed in the following text, the cross section of a revetment consists of different materials, including sheet piles and concrete members, and backfilling sand. Furthermore, the permeability of sheet piles will differ from that of their joints. For this reason, there are cases in which Darcy's law cannot be applied.

In analysis of seepage flows in these cases, it is realistic to treat the cross section of the revetment as a structure comprising materials to which Darcy's law can be applied. Therefore, it is necessary to convert the coefficient of permeability and the wall width in order to apply Darcy's law in an approximate manner.

Though the seepage flow analysis should cover the area behind a reclamation revetment in which the water level can be considered uniform, the analysis can be performed by setting the area commensurate with the required accuracy, considering the structure of the revetment body, conditions of backfilling sand and other conditions. It must be noted, however, that when the permeability of the landfill soil deposited in the reclaimed area is low, the water level behind the reclamation revetment will have a steep gradient in the landfill soil.

① **Permeability of steel sheet pile structures**

(a) The permeability of steel sheet pile structures cannot be derived from Darcy's law. Therefore, it is common to use an appropriate equivalent width and the equivalent coefficient of permeability for that width to determine the permeability in the seepage flow analysis. When doing so, it is preferable to use results of in-situ measurements because it is difficult to say that on-site conditions of joints can be well-simulated in laboratory tests.

(b) Reference 11) gives an example of analyzing the permeability of steel sheet pile structures in situ. The analysis is based on measurements of residual water levels at five steel sheet pile quaywalls. In the analysis, it was assumed that the part of a sheet pile wall below the seabed is an impermeable layer and the part of the wall above the seabed is a 1-m thick permeable layer to which Darcy's law can be applied. The resultant coefficient of permeability, that is, the equivalent coefficient of permeability, was in the range of $1\text{x}10^{-5}$ to $3\text{x}10^{-5}$ cm/s. The same analysis was carried out for two steel pipe sheet pile quaywalls with a diameter of approximately 80 cm and having L-T joints, and the results indicated that the coefficient of permeability for those quaywalls was $6\text{x}10^{-5}$ cm/s. The coefficient of permeability for the backfilling material for the quaywalls mentioned above was in the order of 10^{-2} to 10^{-3} cm/s.

(c) The permeability of steel sheet pile joints has the following characteristics:

In cases of structures with no backfilling material, the permeability of sheet pile joints is similar to that of orifices with abrupt reduction in the cross-sectional area and can be expressed in **equation (14.6.1)** with the constant $n = 0.5$. [12), 13)]

$$q = Kh^n \qquad \textbf{(14.6.1)}$$

where

q : flow rate per unit joint length (cm^3/s/cm)

h : difference in the water level between the front and the rear of the sheet pile (cm)

K,n : constants

In cases of structures with a backfilling material, the characteristics of the backfilling material greatly affect the quantity of seepage through joints. In a part of the backfilling material near a sheet pile joint, there are areas to which Darcy's law cannot be applied. There has been an effort to evaluate the permeability in this part as a composite joint composed of a certain thickness of soil, including the backfilling material, and the sheet pile joint. This idea is effective for conducting the seepage flow analysis. Shoji et al. [14] proposed an empirical equation based on the comprehensive permeability tests considering both the difference in the degree of tensile force in joints and conditions with or without sand filling. The test results indicated that, for backfilled structures with joints filled with sand, the constant n could be approximated to 1.0 and the K value representing the results of the tests was derived.

(d) The degree of reduction in permeability as a result of sealing sheet pile joints against water varies depending on conditions such as the type and use of water sealant, and should be determined based on reliable data, such as results of tests conducted in consideration of the construction conditions at the site. According to results of field tests [15], there were cases in which the quantity of water seeping out of joints with water sealant applied was about 20% to 40% of that of joints with no water sealant applied.

② **Permeability of foundation ground**

(a) Permeability of natural ground

The permeability of the natural ground as a whole can be evaluated using the coefficients of permeability for each soil layer composing the natural ground. In calculating the coefficients of permeability for each soil layer, refer to **Part II, Chapter 3**, **2.2.3 Hydraulic Conductivity of Soil**. In ground which was formed by natural sedimentation, the coefficient of permeability displays directionality, and in many cases, it is larger in the horizontal direction than in the vertical direction. In a case where a structure is placed on the natural ground, the void ratio decreases because of the compression or consolidation of the ground, resulting in a decrease in the coefficient of permeability.

When evaluating the coefficient of permeability based on a laboratory test or Hazen's formula, it is important to accurately grasp the conditions of soil layers based on careful sampling.

(b) Permeability of improved ground

In cases where soil improvement is to be carried out as part of construction of a reclamation revetment, it is necessary to not only evaluate the permeability of the natural ground but also examine how the permeability will be changed by soil improvement.

In the soil between sand piles, in the ground below a replaced sand layer, and in parts of soil that have not been improved by using the deep mixing method, the coefficient of permeability decreases over a long period of time because of consolidation. In addition, sand piles may cause changes in the coefficient of permeability due to disturbance of clayey soil around them and due to clogging of the piles themselves.

When determining the coefficient of permeability of improved parts and non-improved parts of the foundation ground after soil improvement, it is necessary to conduct a well-balanced examination considering the simplification of compositions of the foundation ground and the revetment in the seepage flow analysis, the revetment structure, the accuracy of the coefficient of permeability of water sealing work, and other factors. It is also possible to examine approximate values obtained from a research about similar existing facilities.

(c) In case that the foundation ground is made of rocks, the permeability shall be determined based on thorough preliminary research, because the rock ground may contain cracks, fissures and/or fracture zones, which affect the permeability.[16]

(5) Verification of performance against Level 2 earthquake ground motions

① For the verification of performance against Level 2 earthquake ground motions, refer to **Part II, Chapter 5**, **2.2.4 Performance Verification for Deformation of Facilities during Earthquake** and **Reference (Part III), Chapter 1**, **2 Fundamentals of Seismic Response Analysis**.

② Because revetments have various shapes, it is necessary to set analysis conditions appropriate for the shape of the revetment to be analyzed. For example, gravity-type revetments and sheet pile revetments are considered to exhibit behaviors similar to those of gravity-type quaywalls and sheet pile quaywalls, respectively. Therefore, it can be considered that analysis programs that have been proved to be applicable to gravity-type quaywalls or sheet pile quaywalls are also applicable to gravity-type revetments or sheet pile revetments, respectively.

③ For revetments that are should be quickly inspected of earthquake resistance to prepare for tsunamis that might be triggered by trench earthquakes, such as Tonankai and Nankai earthquakes, a chart-type earthquake-resistance diagnosis system has been proposed. The system allows users to easily predict deformation, such as settlement, from a chart created based on many results of analyses with the Finite Element Analysis Program for Liquefaction Process (FLIP) and facility conditions input by users.[17)] It should be noted, however, that this technique was established for simplified diagnosis of deformation of elongated shore facilities and thus cannot be used for verification of the performance of revetments against Level 2 earthquake ground motions in principle.

14.6.2 Performance Verification of Gravity-type Revetments

(1) Structural types of gravity-type revetments include the caisson type, the L-block type, the cellular-block type, the type made of precast concrete members such as blocks, and the cast-in-place concrete type.

(2) There are common provisions applicable to all revetments and specific provisions applicable to each structural type of revetments. For the performance verification to be conducted in accordance with the former, refer to **Part III, Chapter 4**, **14.6 Performance Verification**. For the performance verification to be conducted in accordance with the latter, refer to **Part III, Chapter 5**, **2.2 Gravity-type Quaywalls** and **Part III, Chapter 5**, **2.11 Upright Wave-absorbing Type Quaywalls**.

14.6.3 Performance Verification of Sheet Pile Revetments

(1) Sheet pile revetments are composed of steel sheet piles, concrete sheet piles, or other sheet piles, and include cantilevered sheet pile revetments, sheet pile revetments with anchorage work, and double sheet pile revetments. It is difficult to start construction of a sheet pile revetment with anchorage work before reclamation has progressed to a certain stage, and it is necessary to manage construction of the revetment by checking the progress of reclamation and the stability conditions examined in advance.

(2) There are common provisions applicable to all revetments and specific provisions applicable to each structural type of revetments. For the performance verification to be conducted in accordance with the former, refer to **Part III, Chapter 4**, **14.6 Performance Verification**. For the performance verification to be conducted in accordance with the latter, refer to **Part III, Chapter 5**, **2.3 Sheet Pile Quaywalls**, **Part III, Chapter 5**, **2.4 Cantilevered Sheet Pile Quaywalls**, **Part III, Chapter 5**, **2.5 Sheet Pile Quaywalls with Raking Pile Anchorages**, **Part III, Chapter 5**, **2.6 Open-type Quaywall with Sheet Pile Wall Anchored by Forward Batter Piles** and **Part III, Chapter 5**, **2.7 Double Sheet Pile Quaywalls**.

14.6.4 Performance Verification of Cellular-bulkhead Revetments

(1) This type of revetment has a cellular structure composed of steel sheet piles, steel plates, or other members. It has high watertightness, just like a steel sheet pile revetment; therefore, it is appropriate for prevention of leakage of landfill soil.

(2) There are common provisions applicable to all revetments and specific provisions applicable to each structural type of revetments. For the performance verification to be conducted in accordance with the former, refer to **Part III, Chapter 4**, **14.6 Performance Verification**. For the performance verification to be conducted in accordance with the latter, refer to **Part III, Chapter 5**, **2.9 Embedded-type Cellular-bulkhead Quaywalls** and **Part III, Chapter 5**, **2.10 Placement-type Cellular-bulkhead Quaywalls**.

14.6.5 Performance Verification of Rubble Mound Revetments

(1) This type of revetment is built in areas where water is relatively shallow, and has a body composed of rubble. It is necessary to take measures to prevent leakage of landfill soil. Shielding work shall be constructed on the front of the revetment to make it resistant to waves. In cases where rubble is available at low price, a rubble mound sloping revetment may be built in an area where water is considerably deep, in expectation of the effectiveness of rubble in purifying seawater and making fish swarm around it.

(2) For the performance verification of rubble mound revetments, refer to **Part III, Chapter 4**, **14.6 Performance Verification**, **Part III, Chapter 4**, **3.3 Gravity-type Breakwaters (Sloping Breakwaters)** and **Part III, Chapter 2**, **2.7 Armor Stone and Blocks**.

14.6.6 Performance Verification of Revetments Covered with Wave-dissipating Blocks

(1) This type of revetment is built in areas exposed to the intensive wave force and composed of the revetment body of each structural type with wave-dissipating work in front of it.

(2) For the performance verification of revetments covered with wave-dissipating blocks, refer to **Part III, Chapter 4**, **14.6 Performance Verification**, **Part III, Chapter 4**, **3.4 Gravity-type Breakwaters (Breakwaters Covered with Wave-dissipating Blocks)** and the descriptions about the performance verification of revetments of each structural type.

14.6.7 Revetments Serving as Facilities Prepared for Accidental Incidents

(1) For the performance verification of revetments serving as facilities prepared for accidental incidents, refer to the descriptions about seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(2) For evaluating the stability against the design tsunami and tsunamis with intensity higher than that of the design tsunami to develop the tsunami-resistant design, refer to the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors** [2], the **Design Concept for Parapets against Tsunamis (Provisional Edition)** [3] and the **Guidelines for Tsunami-Resistant Design of Breakwaters** [4].

14.6.8 Structural Details

(1) Revetments should be provided with scouring prevention work depending on wave conditions.

(2) Revetments shall be provided with appropriate leakage prevention work in consideration of the properties of landfill soil, the revetment structure, the residual water level, and other factors.

(3) If there is a possibility that sediment might flow or wash out of the ground behind a revetment because of the action of waves and others, it is necessary to take appropriate countermeasures by reference to **Part II, Chapter 2**, **6.5 Pressure of Waves Transmitted through Mound and Pressure of Waves in Joints that Act Behind a Revetment** and other references.

(4) Revetments shall be provided with stairs and other ancillary facilities, as necessary.

(5) Revetments may be provided with a parapet to reduce wave overtopping.

(6) If there is a possibility that waves might overtop a revetment, it is necessary to construct apron work to protect the area behind the revetment. The width of the apron work shall be determined in consideration of the wave overtopping rate, the wave run-up height, the structural type of the revetment, and other factors. In addition, drainage ditches, drainage holes, and other appropriate drainage facilities shall be provided to eliminate seawater that has gone beyond the revetment because of wave overtopping. The cross-sectional areas of these drainage facilities shall be determined appropriately in consideration of the wave overtopping rate, the rainfall, and other conditions.

(7) For other structural details, refer to **Part III, Chapter 4**, **Protective Facilities for Harbors** and **Part III, Chapter 5**, **Mooring Facilities**.

14.7 Symbiotic Revetments

(1) As revetments contributing to maintenance of good environments of ports and harbors, symbiotic revetments [18] are designed to allow living beings to grow in mudflats, rocky shores, and other areas in the ports and harbors according to environmental conditions to which the revetments are subjected (**References (Part I), Chapter 3**, **2 Symbiotic Port Facilities**). Existing revetments may be improved to be symbiotic revetments by adding a function that allows living beings to grow.

(2) Factors that affect the capability of a symbiotic revetment to fulfill the purpose to allow living beings to grow (**Reference (Part I)**, **Chapter 3**, **2 Symbiotic Port Facilities**) shall be clarified through environmental research,

numerical modeling, and other techniques. In the performance verification of the revetment, it shall be verified that its structure, cross section, and ancillary facilities are appropriate for the revetment to fulfill its purpose.

(3) The performance requirement for symbiotic revetments is that they shall have a function that allows living things to grow. Factors (dominating actions) that affect the function include the presence or absence of a foundation for living things to grow, external forces such as waves and currents, and an environment necessary for living things to grow. Conditions of the environment necessary for living things to grow include the water depth and underwater visibility that affect the light intensity necessary for photosynthesis and the water temperature that affects the activity of living things. Specifically, for a revetment intended to form seaweed beds, it is necessary that the structure and cross section of the revetment and the foundation and inclination of its ancillary facilities be appropriate for desired seaweed and seagrass to attach to the revetment. It is also necessary that the revetment and its ancillary facilities not block sunlight to such an extent as to affect the light intensity necessary for seaweed and seagrass to grow.

(4) The performance verification of a symbiotic revetment shall be conducted by confirming, based on available knowledge, that the environment of the place where symbiosis of living things is desired is within the range of conditions under which desired living things can grow. For example, for revetments intended to form seaweed beds, the light intensity and water temperature that affect photosynthesis and breathing of seaweed are considered as environmental conditions to be considered in the performance verification. Therefore, in the performance verification of those revetments, it shall be verified that these environmental conditions are within the range of conditions that allow formation of desired seaweed beds. If it is possible to predict changes in the environmental conditions after construction of a symbiotic revetment, environmental changes in the future, and other future possibilities, techniques such as numerical modeling concerning growth of living things may be used for verifying that the environmental conditions are within the range of conditions under which living things can grow.

(5) For the performance verification of symbiotic revetments, refer to **Part III, Chapter 4**, **4 Symbiotic Breakwaters**, **Reference (Part I)**, **Chapter 3**, **2 Symbiotic Port Facilities** and the **Guidelines for Development and Maintenance of Symbiotic Port Facilities** [18)].

(6) Amenity functions may be added to symbiotic functions to make an environmentally friendly revetment that has synergistic effects of those functions.

14.8 Revetments Having Amenity Functions

(1) For the performance verification of amenity-oriented revetments, refer to reference 19).

(2) Provisions about the performance verification of revetments of each structural type may be applied to the performance verification of amenity-oriented revetments.

(3) It is preferable that a revetment to be constructed in a green area having a waterfront line should be designed as an amenity-oriented revetment[20)] and have additional functions that allow users to look at the sea, get close to the sea, and get familiar with the sea.

(4) Amenity functions, such as fishing facilities, may be added to a revetment to make it a multipurpose revetment.[21)]

(5) An amenity-oriented revetment shall have the cross section determined in consideration of the risk that users might fall into the sea, and shall be provided with fall prevention fences and other appropriate ancillary facilities, as needed.

(6) In cases where a facility has an area that is available for people to walk in normal times and likely to be exposed to overtopping waves when waves are high, it is necessary to alert people to the danger of wave overtopping by setting up a sign or taking other appropriate means.

(7) Walkways and slopes of revetments shall have widths, pitches, and other dimensions that allow elderly users and physically disabled users, including those in wheelchairs, to move safely.[22), 23), 24)]

(8) Amenity-oriented functions of a revetment may be enhanced with consideration for inhabitation of living things (**References (Part I)**, **Chapter 3**, **2 Symbiotic Port Facilities**).

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21) MLIT, Ports and Harbours Bureau: Guidelines for the Multipurpose Use of Breakwaters (Second Edition) (Draft), 2017 (in Japanese)
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24) MLIT: Policy Outline for Universal Design, 2005 (in Japanese)

15 Coastal Dikes

[Ministerial Ordinance] (Performance Requirements for Coastal Dikes)

Article 21

The provisions of Article 16 shall apply mutatis mutandis to the performance requirements for coastal dikes.

[Public Notice] (Performance Criteria of Coastal Dikes)

Article 44

The provisions of Article 39 shall apply mutatis mutandis to the performance criteria for coastal dikes.

[Interpretation]

10. Protective Facilities for Harbors

(12) Performance criteria of coastal dikes (Article 21 of the Ministerial Ordinance and the interpretation related to Article 44 of the Public Notice)

Performance criteria of seawalls and their interpretation shall be applied correspondingly to coastal dikes.

15.1 General

(1) The purpose of coastal dikes is to protect the land areas behind them from waves, storm surges, and tsunamis.

(2) For the performance requirements and performance criteria of coastal dikes, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(3) Coastal dikes to which damage might significantly affect human lives, properties, and/or socioeconomic activities and which are required to protect the land areas behind them from the design tsunami and accidental waves shall be built to protect the land areas behind them from the actions concerned, in addition to the purpose mentioned above in (1).

(4) Coastal dikes shall be designed in accordance with this document and by reference to the **Technical Standards and Commentary for Shore Protection Facilities** [1)], the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors** [2)], the **Design Concept for Parapets against Tsunamis (Provisional Edition)** [3)], the **Guidelines for Tsunami-Resistant Design of Breakwaters** [4)], the **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores** [5)] and references 6) and 7). If the facility of interest is a shore protection facility, it shall conform to the **Technical Standards for Shore Protection Facilities**.

15.2 Items to be Considered in Setting of Basic Cross Section

For items to be considered in setting the basic cross section of a coastal dike, refer to **Part III, Chapter 4**, **14.2 Items to be Considered in Setting of Basic Cross Section**.

15.3 Setting of Crown Height of Coastal Dike

For setting the crown height of a coastal dike, refer to **Part III, Chapter 4**, **14.4 Setting of Crown Height of Revetment**.

15.4 Actions

For setting actions to be considered in the performance verification of coastal dikes, refer to **Part III, Chapter 4**, **14.5 Actions**.

15.5 Performance Verification

15.5.1 Performance Verification

For the performance verification of coastal dikes, refer to **Part III, Chapter 4**, **14.6 Performance Verification**.

15.5.2 Coastal Dikes Serving as Facilities Prepared for Accidental Incidents

For the performance verification of coastal dikes serving as facilities prepared for accidental incidents, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**).

[References]

1) National Association of Agricultural Sea Coast, National Association of Fisheries Infrastructure, National Association of Sea Coast, Ports and Harbours Association of Japan: Technical Standards and Commentary for shore Protection Facilities, 2018 (in Japanese)

2) MLIT, Ports and Harbours Bureau: Guidelines for Earthquake-resistant Design of Storm Surge Barriers (Breast Walls) in Ports, 2013 (in Japanese)

3) Fisheries Agency, Fishery Infrastructure Department, Fishing Communities Promotion and Disaster Prevention Division and MLIT, Ports and Harbours Bureau, Coast Administration and Disaster Prevention Division: Design Concept of Breast Walls for Tsunami (Tentative Edition), 2015 (in Japanese)

4) MLIT, Ports and Harbours Bureau: Guideline for Tsunami-Resistant Design of Breakwaters (Partial Revision), 2015 (in Japanese)(http://www.nilim.go.jp/lab/bcg/sokuhou/.le/120514.pdf).

5) Coastal Development Institute of Technology: Technical Manual for Flap-type Land Gates at Ports and Coasts, 2016 (in Japanese)(http://www.nilim.go.jp/lab/fcg/labo/report_ver2.pdf).

16 Jetties

[Ministerial Ordinance] (Performance Requirements for Jetties)

Article 22

1 The performance requirements for jetties shall be such that the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism are satisfied for the purpose of mitigating the influence of littoral drift through effective control of littoral drift.

2 The provisions of Article 14, paragraph (1), item (ii) shall apply mutatis mutandis to the performance requirements for jetties.

[Public Notice] (Performance Criteria of Jetties)

Article 45

The provisions of Article 38 shall apply mutatis mutandis to the performance criteria for jetties.

[Interpretation]

10. Protective Facilities for Harbors

(13) Performance Criteria of Jetties (Article 22 of the Ministerial Ordinance and the interpretation related to Article 45 of the Public Notice)

① Performance criteria of sediment control jetties and their interpretations shall be applied correspondingly to jetties.

② In the performance verifications of jetties, appropriate consideration shall be given to the effects of an increase in the earth pressure due to sedimentation of littoral drift and the effects of waves and river currents, as needed, in addition to the previous paragraph.

③ In setting the layout and dimensions of a jetty, appropriate consideration shall be given to the predominant direction of waves and water currents, topography, expected conditions of use of the jetty, the impact on the natural environment and other factors in order to ensure that the jetty has serviceability.

④ The layout of jetties includes the positions where they are built as well as their directions and the spacing between them. The dimensions of jetties include their structures, crown heights, crown widths, and lengths. In determining the layout of jetties, attention shall be paid to the fact that construction of jetties may cause excessive reduction in longshore sediment transport and thus increase the possibility of shoreline retreat on the surrounding coast.

(1) For the performance verification of jetties, refer to **Part III, Chapter 4**, **3 Breakwaters Having Basic Functions**, with modifications made as necessary in consideration of the structural type. Note that it is necessary to provide appropriate consideration to the effects of an increase in the earth pressure because of sedimentation of littoral drift and the effects of scouring caused by waves and river currents.

(2) For the lengths, spacing, structures, and other details of jetties to be constructed on the updrift side of a port for the purpose of siltation prevention, refer to **Part III, Chapter 4**, **9 Sediment Control Jetties** and the **Technical Standards and Commentary of Shore Protection Facilities** [1].

[References]

1) Technical Committee for Coastal Protection Facilities: Technical standards and commentary of coastal protection facilities, Japan Port Association, pp.3-77-3-85, 2004

17 Parapets

[Ministerial Ordinance] (Performance Requirements for Parapets)

Article 23

The provisions of Article 16 shall apply mutatis mutandis to the performance requirements for parapets.

[Public Notice] (Performance Criteria of Parapets)

Article 46

The provisions of Article 39 shall apply mutatis mutandis to the performance criteria for parapets.

[Interpretation]

10. Protective Facilities for Harbors

(14) Performance Criteria of Parapets (Article 23 of the Ministerial Ordinance and the interpretation related to Article 46 of the Public Notice)

Performance criteria of seawalls and their interpretations shall be applied correspondingly to parapets.

17.1 General

(1) The purpose of parapets is to protect the land areas behind them from waves, storm surges, and tsunamis.

(2) Parapets are structures built on the landward side of a shoreline in cases where there is a fishing port, a port, a harbor, or the like along the shoreline and it is difficult to build coastal dikes, revetments, and similar structures that might interfere with the use of the port or harbor. **Fig. 17.1.1** shows the conceptual diagram of a parapet.

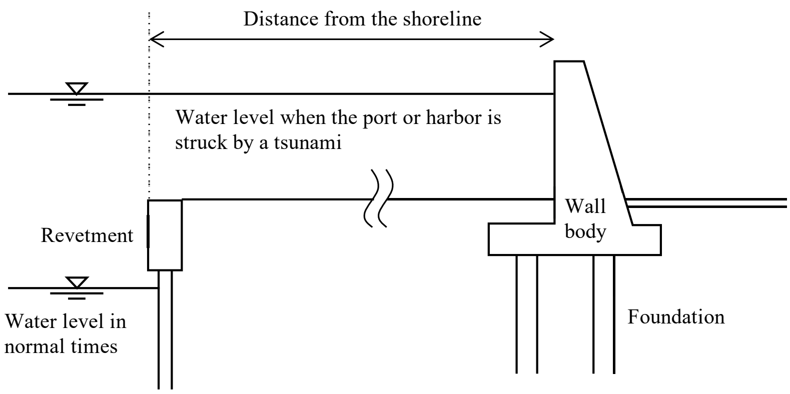

Fig. 17.1.1 Conceptual Diagram of a Parapet

(3) For the performance requirements and performance criteria of parapets, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**).

(4) Parapets to which damage might significantly affect human lives, properties, and/or socioeconomic activities and which are required to protect the land areas behind them from the design tsunami and accidental waves shall be built to protect the land areas behind them from the actions concerned, in addition to the purpose mentioned above in (1).

(5) Parapets shall be designed in accordance with this document and by reference to the **Technical Standards and Commentary for Shore Protection Facilities** [1], the **Guidelines for Tsunami-Resistant Design of Seawalls (Parapets) in Port and Harbors** [2], the **Design Concept for Parapets against Tsunamis (Provisional Edition)** [3], the **Guidelines for Tsunami-Resistant Design of Breakwaters** [4] and the **Technical Manual for Flap Gate Type Land Locks at Ports, Harbors and Seashores** [5]. If the facility of interest is a shore protection facility, it shall conform to the **Technical Standards for Shore Protection Facilities**.

17.2 Items to be Considered in Setting of Basic Cross Section

For items to be considered in setting the basic cross section of a parapet, refer to **Part III, Chapter 4**, **14.2 Items to be Considered in Setting of Basic Cross Section**.

17.3 Setting of Crown Height of Parapet

For setting the crown height of a parapet, refer to Part III, Chapter 4, 14.4 Setting of Crown Height of Revetment.

17.4 Actions

For setting actions to be considered in the performance verification of parapets, refer to **Part III, Chapter 4**, **14.5 Actions**.

17.5 Performance Verification

17.5.1 Performance Verification

(1) General

For the performance verification of parapets, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**) and the descriptions about revetments (**Part III, Chapter 4**, **14.6 Performance Verification**).

(2) Performance verification of parapets under the variable situation associated with Level 1 earthquake ground motions

In the performance verification of a parapet under the variable situation associated with Level 1 earthquake ground motions, it is possible to use the seismic coefficient for verification obtained by determining the natural period of the parapet through a frame-structure analysis or other technique and calculating the seismic coefficient for verification from this natural period and the acceleration response spectrum.[6), 7)] This is based on the rule that the performance should be evaluated under the condition that the parapet is not affected by deformation or other failure of the foundation ground. Therefore, it is necessary to undertake measures to ensure that the parapet will not be affected by factors such as a lateral flow because of deformation of a revetment. In cases where the ground may have liquefied because of earthquake ground motions, it is necessary to assess whether or not the ground has liquefied and appropriately consider measures against liquefaction by reference to **Part II, Chapter 7**, **Ground Liquefaction**.

An earthquake ground motion to be considered in the performance verification of earthquake-resistance shall be set by considering the effects of the surface ground through the calculation of seismic response of the ground. It is necessary to use the seismic response analysis code that allows appropriate evaluation of the amplification of the earthquake ground motion in soft ground. (Refer to **Part II, Chapter 6**, **1.2.3 Calculation of Seismic Response of Surface Ground**.) The time history of acceleration on the ground surface shall be calculated through the one-dimensional seismic response analysis as described in **Part II, Chapter 6**, **1.2.3 Calculation of Seismic Response of Surface Ground** using the time history of acceleration of the earthquake ground motion set for the engineering bedrock as the input earthquake ground motion. The acceleration response spectrum can be obtained from the calculated acceleration time history and used for calculating the response acceleration for the natural period of the parapet, and the calculated response acceleration can be divided by the acceleration of gravity to obtain the characteristic value of the seismic coefficient for verification. The damping constant for calculating the acceleration response spectrum may be assumed to be 0.4. **Fig. 17.5.1** shows an example of an ordinary procedure for setting the seismic coefficient for verification.

The natural period for a parapet with pile foundation can be calculated through a frame-structure analysis as described in **Part III, Chapter 5**, **5.2.3 (14) Earthquake Ground Motion to be Considered in Performance Verification of Earthquake-resistance**. The frame-structure of the parapet shall be modeled taking account of the ground spring that expresses the subgrade reaction in a part for which the resistance caused by ground deformation is considered. As for the wall body, the horizontal beam element corresponding to the footing and the vertical beam element corresponding to the part of the wall body above ground shall be used in the analysis. The natural period of the parapet can be calculated by placing a node corresponding to the center of gravity of the wall body, making a horizontal load act there, determining the spring constant for the entire parapet based on the relation with horizontal displacement under a very small load, and taking account of the weight of the wall body.

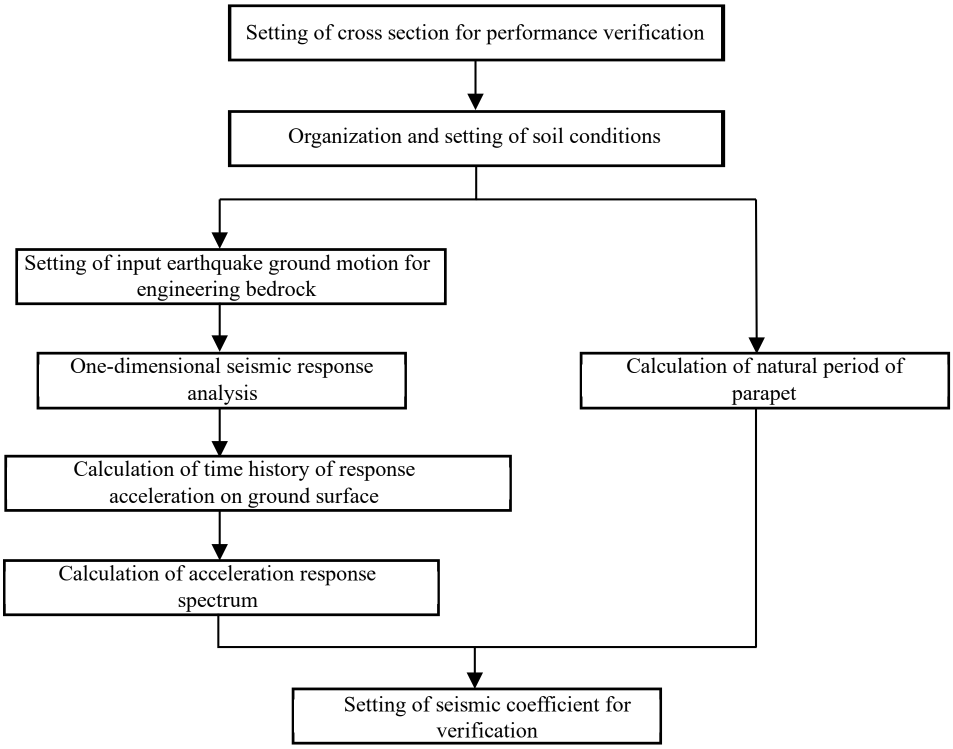

Fig. 17.5.1 Ordinary Procedure for Setting Seismic Coefficient for Verification of a Parapet

17.5.2 Parapets Serving as Facilities Prepared for Accidental Incidents

For the performance verification of parapets serving as facilities prepared for accidental incidents, refer to the descriptions on seawalls (**Part III, Chapter 4**, **10 Seawalls**).

[References]

1) Japan Port Association (JPA): Technical standards and commentaries for shore protection facilities, 2018.

2) Port and Harbour Bureau, Ministry of Land Infrastructure, Transport and Truism (MLIT): Guideline for tsunami-resistant design for parapet-type seawalls , 2013.

3) Fisheries Agency, Fishery Infrastructure Department, Fishing Communities Promotion and Disaster Prevention Division and MLIT, Ports and Harbours Bureau, Coast Administration and Disaster Prevention Division: Design Concept of Breast Walls for Tsunami (Tentative Edition), 2015 (in Japanese)

4) Port and Harbour Bureau, Ministry of Land Infrastructure, Transport and Truism (MLIT): Guideline for tsunami-resistant design for breakwaters (revised edition), 2015.

5) Costal Development Institute of Technology (CDIT): Technical manual for Flap Gate-type seawall in port and coastal areas, 2016.

6) E. Kohama and H. Fukawa: Numerical analysis and model testing on a method for evaluation of coastal parapet levees' seismic coefficients for Level-1 earthquakes, Report of PARI, Vol.56, No.3, pp.49-115, 2017.

7) H. Fukawa and E. Kohama: A study on the applicability of frame analysis and response spectrum method to evaluate seismic coefficients of coastal parapet levees with pile foundations, Journal of JSCE A1, Vol.73, No.4, pp.I-431-I-442, 2017.

18 Siltation Prevention Facilities

18.1 General

(1) In cases where siltation of harbors and waterways is expected, the mode of siltation needs to be understood on the basis of an adequate investigation of the phenomena of the potential causes of siltation and appropriate countermeasures should be taken considering the various types of effects caused by siltation prevention works, safe navigation of ships, economy, and so forth.

(2) Causes of Siltation

Siltation is a phenomenon where littoral drift, windblown sand, discharged sediments through river, and so on invade harbor water areas, such as waterways and basins, and settle on to the sea bottom of the area, hindering port functions with the water depth to be shallow. There are cases where the water depth becomes shallower than the required depth without any substantial change of sediment volume, such as the formation of a sand wave[1)] or collapse of side slopes of dredged waterways. The causes of siltation are listed below:

① Invasion and accumulation of littoral drift (mainly caused by waves and wave induced current)

② Settling and accumulation of river discharged sediments

③ Plunging and deposition of windblown sand

④ Movement of sediments within the objective area and change in the location of deposition

⑤ Movement of sediments because of disturbances in the harbor, collapse of slopes in waterways, and formation of sand waves

(3) Modes of Siltation

The modes of siltation in water areas surrounded by breakwater and such can be classified as shown in **Fig. 18.1.1** according to the source of sediment, courses and modes of sediment invasion, and deposition processes. Siltation in open water areas, such as off-the-harbor waterways, has, for example, the following various modes:

① Siltation often accompanies scouring of a neighboring area, as shown in **Fig. 18.1.2 (a)**, in relatively shallowly dredged waterways and others if waves are dominating action and the sea bottom is sandy.

② Relatively equal siltation including the side slopes in waterways often occurs as shown in **Fig. 18.1.2 (b)** in relatively shallowly dredged waterways where the bottom sediment is soft mud with silt and clay.

③ The siltation rate is the higher at the bottom of the waterway as shown in **Fig. 18.1.2 (c)** in the waterways deeply dredged from the surrounding sea bottom.

④ Waterways dredged by cutting a natural sand bar in straits and others tend to silt up so that the bar topography is restored.

⑤ When dredging the sea bottom where a sand wave naturally exists, the sand wave tends to restore the sea bottom topography.

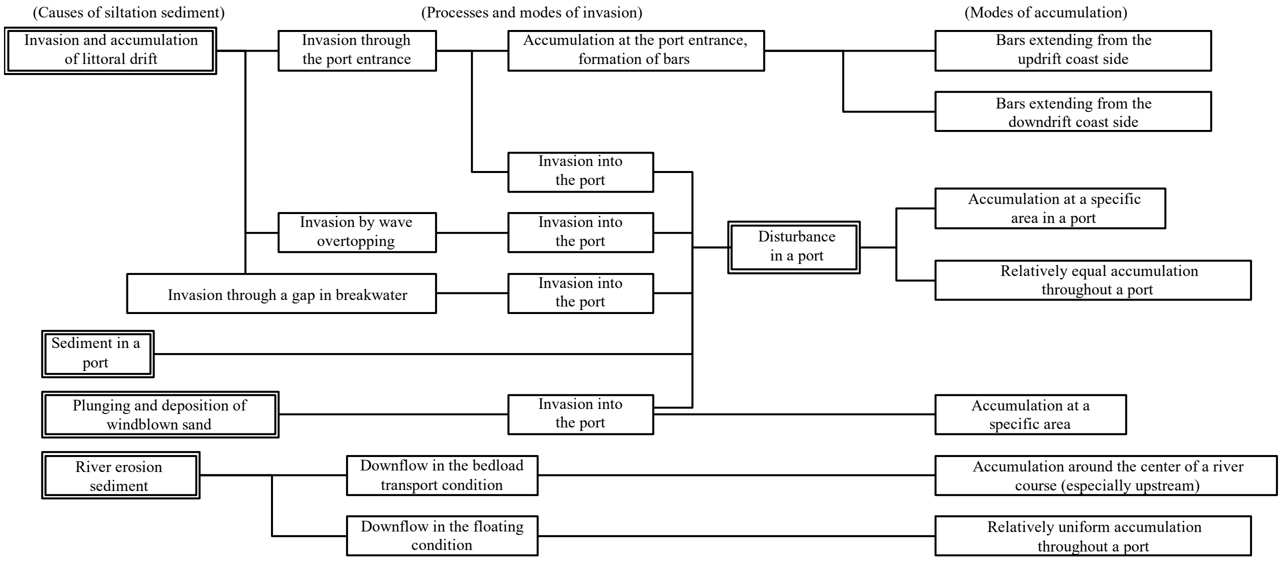

Fig. 18.1.1 Modes of Siltation in Harbor Water Areas

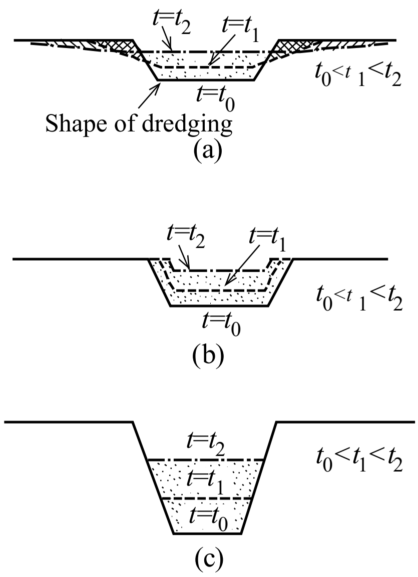

Fig. 18.1.2 Modes of Siltation in Waterways (Where t Denotes the Elapsed Time)

(4) Types of Siltation Prevention Measure Works

The siltation prevention measure works are listed below:

① To prevent siltation by building breakwater and such, as shown in **Table 18.1.1**.

② To effectively trap sediments by outbreak, pocket dredging upstream of an estuary harbor, or other measures and to perform its maintenance dredging.

③ To perform maintenance dredging.

Table 18.1.1 Facilities Used as Semi-permanent Siltation Prevention Measure Works

Countermeasure against longshore sediment transport	Prevention of invasion through the port entrance	Breakwater, jetty (training jetty)
	Prevention of invasion by wave overtopping	Raising of breakwaters
	Prevention of sediment transparent invasion	Sediment infiltration prevention work
River erosion control facility	Increase in the river bedload transport power	Training jetty
	Prevention of erosion sediment invasion	Separation levee
	Reduction of erosion sediment	Division works
Windblown sand prevention work	Reduction of windblown sand	Afforestation, windblown sand control forest
	Prevention of windblown sand invasion	Sand invasion prevention fence, among others

Among those measure works, sediment invasion prevention measure works by laying structures may include building submerged breakwaters by driving sheet panels as well as concrete blocks.

(5) Selection of Siltation Prevention Measure Works

Since the concept of siltation prevention measure works differs for each measure, the most appropriate measure will be selected. When selecting a siltation prevention measure work, it should be determined by adequately investigating the actual conditions and mechanisms of siltation, thoroughly considering the influence on the surrounding environment, and referring to past practical examples, among others. It is also beneficial to perform a study using a movable bed model experiment.

18.2 Facilities for Trapping Littoral Drift or River Erosion Sediment

(1) When the aim is to prevent siltation due to longshore littoral drift by means of maintenance dredging, an appropriate facility to trap the sand should be built accordingly at a proper location, at which the facility can trap and prevent littoral drift from invading waterways and basins. This facility should take measures to improve the wave conditions when dredging and increase the dredging efficiency.

It is preferable to fully study and determine the type and layout, among other criteria, of these longshore sediment transport trap facilities by considering their capability to trap littoral drift, the dredging conditions, the economy, and so forth.

(2) Facilities to Trap Littoral Drift

As a method for trapping longshore littoral drift, provisions to limit sand deposition areas are commonly employed in various countries, by means of building detached breakwaters or partially reducing the crown height of updrift breakwaters. Besides, countermeasures against local siltation by a sand bar's restoration action in open-cut waterways crossing a sand bar in the sea floor of straits and other structures and pocket dredging considered as a measure against siltation due to, for example, river discharged sediment, can be considered to be trapping facilities of littoral drift.

(3) Proper Positioning of Littoral Drift Trapping Facilities

Littoral drift trapping facilities can be installed at areas where sediment deposition occurs easily under natural conditions, as shown in **Fig. 18.2.1 (a)**, **(b)**, and **(c)**, and artificial conditions can be created to encourage sediments to settle out of flows with a high concentration of littoral drift, as shown in **Fig. 18.2.1 (d)**, **(e)**, and **(f)**. In order to identify such specific locations and capture littoral drift in the most efficient manner, an adequate understanding of the moving condition and mechanism of littoral drift is indispensable. Furthermore, when selecting the positions for littoral drift trapping facilities, in addition to the littoral drift trapping efficiency, in cases where trapped sediments are dredged, it is preferable to give adequate consideration to dredging conditions, in other words, to easily maintain the water depth necessary for the navigation of dredgers and to keep calm conditions during dredging works, among other goals.

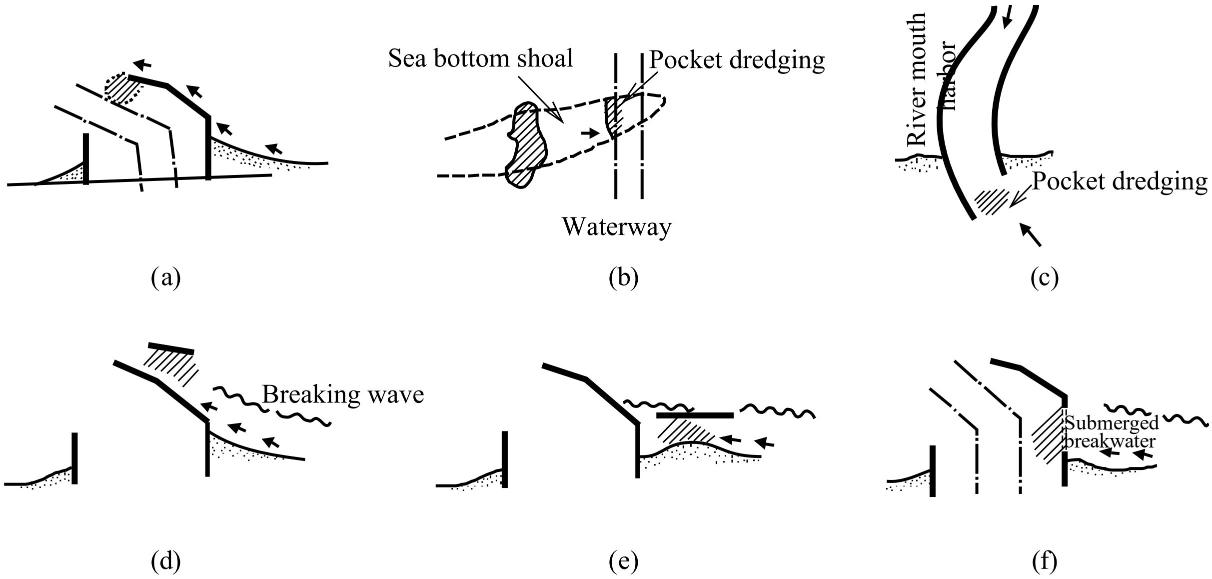

Fig. 18.2.1 Positioning of the Trapping Facility of Littoral Drift

(4) Size of the Littoral Drift Trapping Facility

The size of the littoral drift trapping facility generally depends on the volume of trapped sediments and physical conditions required for settlement and accumulation of littoral drift. The required conditions for settlement and accumulation of littoral drift are determined by the result of field measurements, past performance, movable bed model experiment, and so forth.

18.3 Windblown Sand Prevention Work

18.3.1 General

(1) When windblown sand becomes a concern for siltation of ports and waterways or for the environmental preservation of surrounding areas, adequate means shall be taken to prevent windblown sand depending on the situation.

(2) Windblown sand, that is, sand that is moved by winds, is carried into harbors or waterways where it settles and deposits, causing siltation. In some cases, it also accumulates on road surfaces or gets dispersed into residential areas as dust, disrupting the daily living of the residents. In particular, there are many instances where open digging of dunes or land reclamation causes problems related to windblown sand, and thorough countermeasures need to be prepared in advance.

18.3.2 Selection of a Work Method

(1) The windblown sand prevention work method is determined by deeply understanding the characteristics of each work method after adequately investigating and studying the current situation of the windblown sand and its expected situation in the future.

(2) The windblown sand phenomenon depends on natural conditions, such as the wind direction and wind velocity, and the characteristics of the sediment (grain size distribution and degree of ground humidity), and these determine characteristics such as the direction, amount, and distribution of the windblown sand. When taking some countermeasures against windblown sand, there is a need to investigate these characteristics and to select a proper method considering the nature of the problems concerning the windblown sand, land use plan of the windblown-sand-prone area, or social conditions such as the economy.

(3) The following windblown sand prevention work methods are generally used.

① **Sediment trapping works and windbreaker fences**

Traditionally, a method where multiple rows of low (about 1 m high) sand fences are built to trap windblown sand, thus growing artificial sand dunes to enhance the effectiveness of sand breaking, is used. In some cases, relatively tall windbreaker fences can be built around to prevent the blowing of sediments and other particles from around the reclaimed ground or the powder stockyard.

② **Sand retaining fences**

A low sand control hedge is built to improve the surface roughness, weaken the wind shear force on the ground level, and reduce the surface sand.

③ **Shielding works**

The sand surface is shielded with artificial materials to reduce the movement of sand.

④ **Afforestation works**

Adequate plants are grown on the sand surface to shield the sand surface. This may be considered to be a kind of shielding method.

⑤ **Plantation works**

Trees are planted on the leeward direction of the windblown sand area to prevent windblown sand.

(4) Windblown sand prevention works conducted from the standpoint of the so-called coastal sand erosion control intended to stabilize sandy seashores normally combine several works. These procedures and works include those indicated in **Fig. 18.3.1**. Refer to Reference 2) for details. For trees suitable for coasts, refer to the **Civil Engineering Handbook**.[3)]

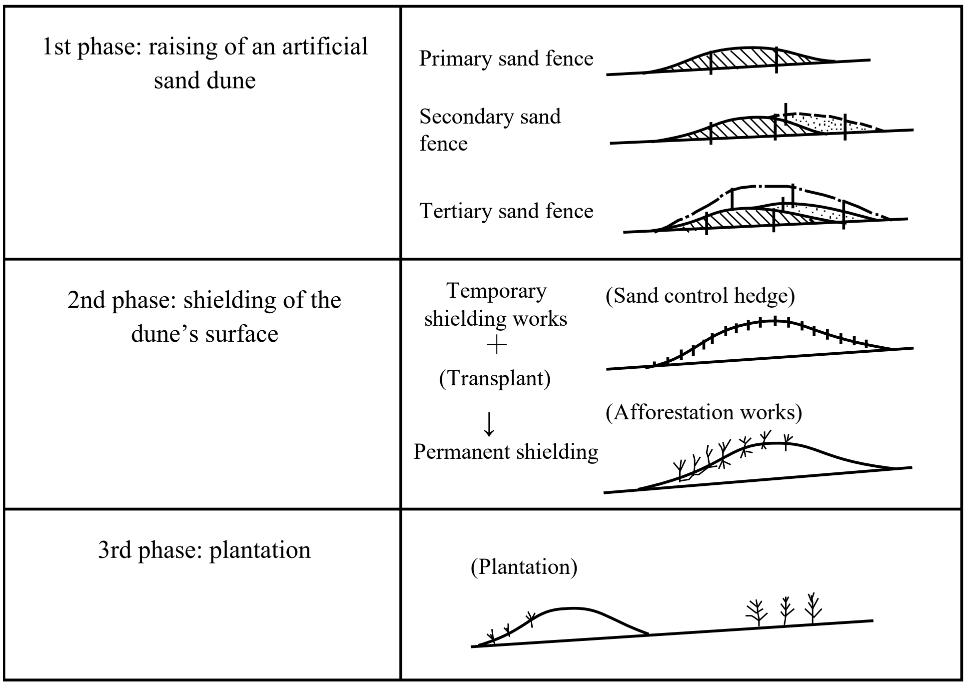

Fig. 18.3.1 Procedure for Artificial Sand Dune Raising Work

[References]

1) OZASA, H.: Field Investigation of Submarine Sand Banks and Large Sand Waves, Rept. of PHRI Vol. 14, No. 2, pp.3-46, 1975 (in Japanese)

2) Tanaka, K., Y. Nakajima, H. Endou and E. Kinnai: Sabo at coast (Coastal erosion control), Sabo Science, Compendium of Sabo Series, III-9, Japan Society of Erosion Control Engineers, Ishibashi-shoten Publishing,1985 (in Japanese)

3) JSCE, Civil Engineering Handbook, Vol. II, pp. 2135-2136, 1989 (in Japanese)