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, and other environmental conditions, as well as ship navigation and other usage conditions of the water areas around the facilities concerned.

Ministerial Ordinance

Necessary Items concerning Protective Facilities for Harbor

Article 24
The matters necessary for the performance requirements of protective facilities for harbor as specified in this Chapter by the Minister of Land, Infrastructure, Transport and Tourism and other requirements shall be provided by the Public Notice.

Public Notice

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 provided in the subsequent article through Article 46.

[Technical Note]

(1) The purposes of protective facilities for harbors include ensuring harbor calmness, maintaining water depth, preventing beach erosion, controlling the rise of water level in the areas behind the facilities during storm surges, and diminishing invading waves by tsunami, as well as protecting harbor facilities and the hinterland from waves, storm surges, and tsunamis. In recent years, water intimate amenity functions have also been required. In general, there are many cases in which the protective facilities for harbors are expected to provide a combination of several of these functions. Accordingly, in performance verifications, due consideration to enable the facilities to fulfill these purposes is necessary.

(2) When constructing protective facilities for harbors, their layout and structural type shall be decided after giving careful consideration to the influences that will be exerted on the nearby water area, facilities, topography, and water currents. The influences caused by the protective harbor facilities for harbors are as follows:

① When the protective facilities are constructed on a coast of sandy beach, they may cause various morphological changes to the surrounding area such as beach accretion or erosion.

② Construction of breakwaters may increase the wave height at the outside of the protective facilities because of reflected waves.

③ In the inside of a harbor, the calmness of water area may be disturbed because of multiple wave reflections triggered by construction of the new protective facilities or harbor oscillations due to the changes of harbor shape.

④ Construction of the protective facilities may bring about changes in the surrounding tidal currents or flow conditions of a river mouth, thus inviting localized changes of water quality.

(3) Because of the fact that the protective facilities also provide a habitat for marine organisms such as fish, marine plants, and plankton, the biological environments must also be taken into consideration when planning a facility layout and making structural design.

(4) When locating the protective facilities adjacent to the areas such as natural park zones or cultural facilities, it is preferable to consider not only the functions of the facilities themselves but also external appearance such as shape and color. In addition, in situations where water intimate amenity functions will be added to the protective facilities, convenience and safety of people must also be taken into consideration.

(5) Because there is a danger that damage to the protective facilities may affect the safety of ships in the harbor, the mooring facilities, hinterland facilities, it is preferable to conduct an adequate examination corresponding to the performance requirements of the protective facilities when constructing, improving, and maintaining those facilities.

2 Common Items for Breakwaters

Ministerial Ordinance

Performance Requirements for Breakwaters

Article 14

1 The performance requirements for breakwaters shall be as specified in the subsequent 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 reduction of the height of waves intruding into the harbor.

(2) Damage to a breakwater due to self weight, variable waves, Level 1 earthquake ground motions, and/or other actions shall not impair the functions of the breakwater concerned and shall not adversely affect its continued use.

2 In addition to the provisions of the preceding paragraph, the performance requirements for breakwaters mentioned in the following are specified in the respective items.

(1) The performance requirements for a breakwater which is required to protect the hinterland of the breakwater concerned from storm surges or tsunamis shall be such that the breakwater satisfies the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable appropriate reduction of the rise in water level and flow velocity due to storm surges or tsunamis in the harbor.

(2) The performance requirements for a breakwater which is provided for use by an unspecified large number of people shall be such that the breakwater satisfies the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to ensure the safety of the users of the breakwater.

(3) The performance requirements for a breakwater in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity, in consideration of its structure type, shall be such that the damage from tsunamis, accidental waves, Level 2 earthquake ground motions and/or other actions do not have a serious impact on the structural stability of the breakwater concerned with respect to the breakwater types even though the damage may impair the functions of the breakwater concerned. Provided, however, that as for the performance requirements for the breakwater which is required to protect the hinterland of the breakwater concerned from tsunamis, the damage due to tsunamis, Level 2 earthquake ground motions and/or other actions shall not adversely affect restoration through minor repair works of the functions of the breakwater concerned.

Public Notice

Performance Criteria for Breakwaters

Article 34

1 The performance criteria which are common for breakwaters shall be as specified in the subsequent items.

(1) Breakwaters shall be arranged appropriately so as to satisfy the harbor calmness provided in item iii) of Article 31, 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 intended wave-absorbing function.

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

(1) The performance criteria for the breakwaters which are required to protect the hinterland from storm surge shall be such that the breakwaters are arranged appropriately so as to reduce the rise of water level and flow velocity in the harbor due to storm surge and have the dimensions necessary for their function.

(2) The performance criteria for the breakwaters which are required to protect the hinterland from tsunamis shall be such that the breakwaters are arranged appropriately so as to reduce the rise of water level and flow velocity in the harbor due to tsunamis and have the dimensions necessary for their function.

(3) The performance criteria for the breakwater which is utilized by an unspecified large number of people shall be such that breakwaters have the dimensions necessary to secure the the safety of users in consideration of the environmental conditions to which the facilities concerned are subjected, the utilization conditions, and others.

(4) The performance requirements for the breakwater in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity by the damage to the breakwater concerned shall be such that the degree of damage owing to the actions of tsunamis, accidental waves, or Level 2 earthquake ground motions, which are the dominant actions in the accidental action situation, is equal to or less than the threshold level corresponding to the performance requirements.

[Commentary]

(1) Performance Criteria for Breakwaters

① Common criteria for breakwaters

(a) Harbor calmness (usability)

1) Allowable transmitted wave height
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. In the performance verifications of breakwaters, the allowable transmitted wave height or wave transmission ratio shall be set appropriately in order 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) Dimensions for securing harbor calmness
The dimensions for securing harbor calmness shall indicate a structure including shape and crown height which 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 settlement of the ground.

② Specific breakwaters

(a) Storm surge protection breakwaters (usability)
The dimensions of storm surge protection breakwaters shall indicate the crown height, opening width, and water depth at the opening. In setting the arrangement, 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 so that the breakwater demonstrates a peak cut effect in reducing the water and flows of water due to storm surges.

(b) Tsunami protection breakwaters (usability)
The dimensions of tsunami protection breakwaters shall indicate the crown height, opening width, and water depth at the opening. In setting the arrangement, 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 so that the breakwater demonstrates a peak cut effect in reducing the water level and flows of water due to tsunamis.

(c) Amenity-oriented breakwaters (usability)
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, 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.

(d) Breakwaters of facilities prepared for accidental incidents
The settings in connection with the performance criteria and design situations (limited to accidental situations) which are common to breakwaters of facilities prepared for accidental incidents shall be as shown in **Attached Table 15**. The reason for indicating “damages” in the “verification items” column of **Attached Table 15** is that it is necessary to use a comprehensive term taking account that

the verification items will vary depending on the type of structure. In the performance verifications of breakwaters of facilities prepared for accidental incidents, among the settings for the performance criteria and the design situations in connection with accidental situations associated with Level 2 earthquake ground motion, tsunamis, and accidental waves, those for which the performance verification is necessary shall be set appropriately, depending on the type of structure of the objective breakwater.

Attached Table 15 Settings for Performance Criteria and Design Situations limited to Accidental Situations Common to Breakwaters of Facilities Prepared for Accident

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
14	2	3	34	2	4	Safety	Accidental	L2 earthquake ground motion	Self weight, water pressure	Damage	–
								Tsunami	Self weight, water pressure, water flows	Damage	–
								Accidental waves	Self weight, water pressure	Damage	–

(e) Tsunami protection breakwaters of facilities against accidental incidents

The settings in connection with the performance criteria and the design situations limited to accidental situations of tsunami protection breakwaters of facilities prepared for accidental incidents shall be as shown in **Attached Table 16**. In the performance verification of tsunami protection breakwaters of facilities prepared for accidental incidents, among the settings for the performance criteria and the design situations in connection with the accidental situations associated with Level 2 earthquake ground motion, tsunamis, and accidental waves, those for which the performance verification is necessary shall be set appropriately, depending on the type of structure of the tsunami protection breakwater of interest.

It may be noted that, as the performance criteria in connection with the accidental situations which 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.

Attached Table 16 Settings for Performance Criteria and Design Situations limited to Accidental Situations Common to Tsunami Protection Breakwaters of Facilities Prepared for Accidental Incidents

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
14	2	3	34	2	4	Restorability	Accidental	L2 earthquake ground motion	Self weight, water pressure	Deformation of breakwater body	Limit value of residual deformation
								Tsunami	Self weight, water pressure, water flows	Sliding and overturning of breakwater body, bearing capacity of foundation ground	Limit value of sliding Limit value of overturning Limit value of bearing capacity
						Safety		Accidental waves	Self weight, water pressure	Sliding and overturning of breakwater body, bearing capacity of foundation ground	Limit value of sliding Limit value of overturning Limit value of bearing capacity

[Technical Note]

2.1 Principals of Performance Verification

[1] General

Maintenance of harbor calmness shall be examined from the two viewpoints which include 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, 4.5 Concept of Harbor Calmness** and **Chapter 3, 3 Basins** can be used as references.

[2] Layout

(1) Breakwaters are constructed to maintain the harbor calmness, facilitate smooth cargo handling, ensure the safety of ships during navigation or anchorage, and protect port facilities. To fulfill these requirements, the following are required:

① Breakwaters should be so located that the harbor entrance is at the location not facing the direction of the most frequent waves and the direction of the highest waves in order to reduce entrance of waves to 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 where 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 in order to achieve the calmness in a harbor but is inconvenient for navigation. The direction of most frequent waves and the direction of the highest waves are not necessarily the same. In this 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.

(2) In situations where 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.

(3) 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 which 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, 4.3.4[3] Transformation of Waves at Concave Corners near the Heads of Breakwaters and around Detached Breakwaters** can be used as reference; for breakwaters to be constructed on sandy beaches, **Part II, Chapter 2, 6.3 Littoral Drift** can be used as reference.

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

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

(6) In the areas surrounding shoals, the wave height often increases owing to 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 very large structure may be required when a breakwater is placed over or directly behind a shoal.

(7) For detached breakwaters which are to be constructed in isolation offshore, if the length of the breakwater is less than several times that of the incident waves, the distribution of the wave heights behind the breakwater will fluctuate greatly due to the effect of diffracted waves from the two ends of the breakwater, which will affect the stability of the breakwater body; therefore, caution is necessary. For the effects of diffracted waves, **Part II, Chapter 2, 4.3.2 Wave Diffraction** and **Part II, Chapter 2, 4.3.4 [3] Transformation of Waves at Concave Corners near the Heads of Breakwaters and around Detached Breakwaters** can be used as reference.

[3] Selection of Structural Type and Setting of Cross Section

(1) In setting the cross sections of breakwaters, it is preferable to select the type of structure 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.

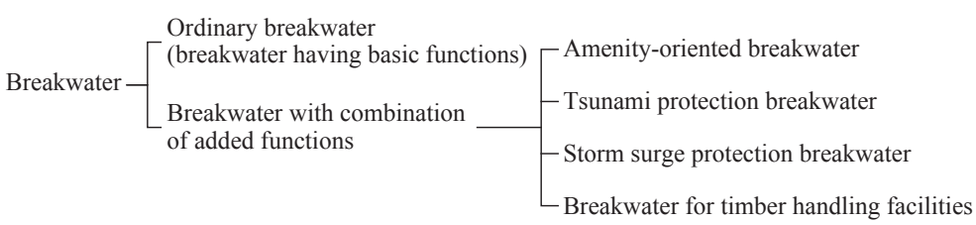
(2) Breakwaters are generally classified as shown in **Fig. 2.1.1** by the type of structure and functions or purposes. In this figure, ordinary breakwater means a breakwater having basic functions.

(3) Selection of a permeable type breakwater structure is advantageous for promoting circulation of sea water 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 when adopting this type.

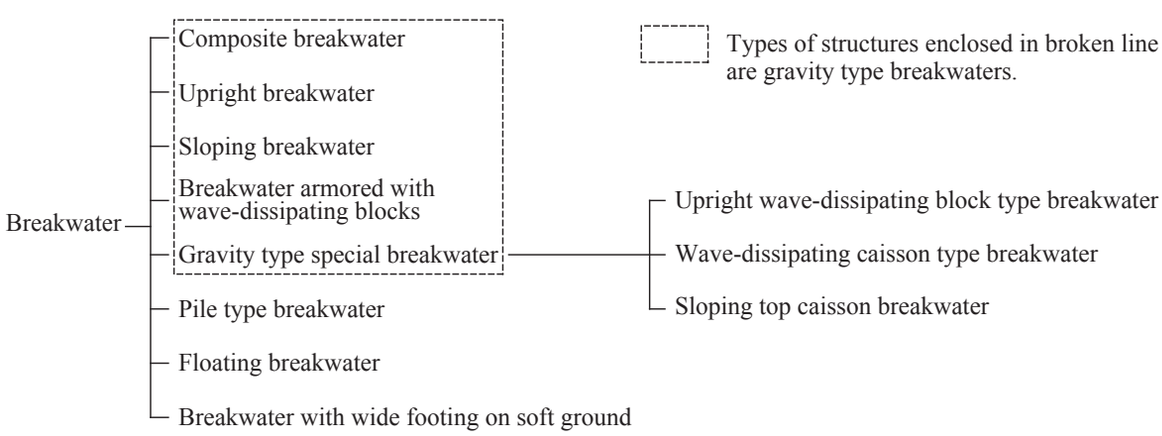
(4) There are also cases in which creative ingenuity is used to promote adhesion of marine life inside and outside the harbor.[1), 2), 3), 4), 5), 6), 7), 8), 9), 10)]

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

(6) In determining the cross-sectional dimensions of the wave-dissipating work in the wave-dissipating function of a breakwater, it is necessary to give 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.



(a) Classification by function



(b) Classification by type of structure

Fig. 2.1.1 Classification of Breakwaters

2.2 Performance Verification

(1) In the performance verification 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, considering the harbor calmness necessary for cargo handling and refuge. In the performance verifications in connection with the harbor calmness of basins, **Part II, Chapter 2, 4.5 Concept of Harbor Calmness** can be used as reference. Furthermore, it is preferable that conditions be set to enable protection of the harbor 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. In the existing breakwaters, there are many examples in which the crown height is determined as follows.

① In a harbor of large ships' calling, 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 where it is not necessary to consider the influence of storm surge.

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) Even in case of a harbor of large ships' calling with 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.

(4) In the performance verification for the effects of reflected waves, **Part II, Chapter 2, 4.3.4 Wave Reflection** can be used as reference.

(5) In **3.1 Gravity-type Breakwaters (Composite Breakwaters)**, the standard performance verification method and the partial factors are shown in respective types of structures. However, the breakwaters used in recent years have included types with multiple structural features. In this case, it is necessary to determine the partial factors based on an appropriate evaluation of the probability distributions associated with design parameters such as wave force, considering each structural features. Reference **11)** presents a method of determining the partial factors for a sloping-top caisson breakwater covered with wave-dissipating blocks as an example of cases of this type and can be used as reference.

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 shall be as specified in the subsequent items:

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

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

[Commentary]

(1) Performance Criteria for Gravity-type Breakwaters

① Composite breakwaters

(a) Among the settings in connection with the performance criteria and design situations excluding accidental situations of gravity-type breakwaters, those pertaining to composite breakwaters shall be as shown in **Attached Table 17**.

As settings pertaining to composite breakwaters, in addition to these provisions, the settings in connection with the **Public Notice, Article 22, Paragraph 3** (Scouring and Sand Washing Out) and **Article 28 Performance Criteria for Armor Stones and Blocks** can be applied as necessary, and the settings in connection with **Article 23** through **Article 27** can be applied depending on the type of members comprising the objective composite breakwater.

Attached Table 17 Settings in Connection with Performance Criteria and Design Situations (excluding accidental situations) of Composite Breakwaters (Gravity-type Breakwaters)

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
14	1	2	35	1	1	Serviceability	Permanent	Self weight	Water pressure	Circular slip failure of ground	Failure probability in permanent situation for self weight (Pf = 4.5 x 10^{-4})
					2		Variable	Variable waves	Self weight, water pressure	Sliding of breakwater body	Exceedence probability of sliding for allowable sliding
										Overturning of breakwater body, bearing capacity of foundation ground	System failure probability for variable situations associated with waves (P_f = 8.7 x 10^{-3})
								L1 earthquake ground motion	Self weight, water pressure	Sliding/overturning of breakwater body, bearing capacity of foundation ground	Limit value for sliding Limit value for overturning Limit value for bearing capacity (Target values of maximum and residual deformation)

(b) Permanent situation when dominating action is self weight

1) Failure probability

The index expressing the danger of ship failure for the permanent situation when the dominating action is self weight is the probability that the ground will fail due to circular slip failure. Its standard allowable value is P_f = 4.5 x 10^{-4}. This standard limit value can be determined as the safety level for caisson type composite breakwaters that roughly minimizes the expected total cost which is the sum of the initial construction cost and the expected value of recovery costs.

(c) Variable situation when dominating action is variable waves

1) System failure probability
The indices expressing the danger of failure occurring due to sliding, overturning and foundation failure of the breakwater body, for the variable situation when the dominating action is variable waves is the system failure probability based on the balance of forces for the action of waves with a 50 year return period. Its standard limit value is $P_f = 8.7 \times 10^{-3}$. This standard limit value can be determined as the average safety level possessed by caisson type composite breakwaters and breakwaters covered with wave-dissipating blocks which were designed by using the conventional design method based on factor of safety.

2) Exceedence probability of sliding displacement relative to allowable sliding displacement
One of the indexes of the danger of failure occurring due to sliding of the breakwater body for the variable situation when the dominating action is variable waves is the probability that the sliding displacement of the breakwater body will exceed the allowable sliding displacement. When conducting a performance verification using this index, the allowable sliding displacement of the breakwater body and the limit value of its exceedence probability shall be set appropriately considering the importance of the object facilities. Methods of setting these items comprise the method of setting the limit value as the probability that the total sliding displacement during the design working life will exceed the allowable sliding displacement, and the method of setting the limit value as the probability that the sliding displacement for multiple design waves having different return periods will exceed the allowable sliding displacement. In addition to the verification of sliding of the breakwater body described here, verification shall also be conducted appropriately for overturning and bearing capacity.

(d) Variable situation when dominating action is Level 1 earthquake ground motion

1) Necessity of performance verification
In the performance verifications in connection with Level 1 earthquake ground motion, the necessity of verification shall be judged based on the relative relationship of the degree of influence of variable waves and Level 1 earthquake ground motion on the stability of the breakwater body. In general, there are many cases in which the performance verification for Level 1 earthquake ground motion is omitted in the performance verifications of breakwaters. However, caution is necessary in judging the necessity of the performance verification for Level 1 earthquake ground motion when the construction position of the facilities is deep and the design wave height is small, as actions associated with Level 1 earthquake ground motion can become the dominant factor in such cases.

2) Deformation
The deformation of the breakwater body is defined as the index of the danger of failure occurring due to sliding or overturning of the breakwater body for the variable situation when the dominating action is Level 1 earthquake ground motion. When performing a performance verification using this index, the allowable deformation of the breakwater body shall be set appropriately.

3) Others
In the performance verifications in connection with Level 1 earthquake ground motion, there is a danger that the functions required in the objective breakwater may be impaired by settlement of the ground and liquefaction of the ground. Therefore, appropriate consideration shall be given to the effects of the settlement and the liquefaction of the ground due to the action of Level 1 earthquake ground motion as stipulated in the **Public Notice, Article 15 Ground Subsidence** and **Article 17 Ground Liquefaction**.

② Upright breakwaters

(a) The performance criteria for upright breakwaters shall be applied to the performance criteria for composite breakwaters.

③ Sloping breakwaters
Among the settings in connection with the performance criteria and designs states excluding accidental situations of gravity-type breakwaters, those pertaining to sloping breakwaters shall be as shown in **Attached Table 18**. As performance criteria for sloping breakwaters, in addition to these criteria, the settings in connection with the **Public Notice, Article 22, Paragraph 3** (Scouring and Sand Washing Out) and **Article 28 Performance Criteria for Armor Stones and Blocks** shall be applied as necessary.

Attached Table 18 Settings for Performance Criteria and Design Situations (excluding accidental situations) of Sloping Breakwaters

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
14	1	2	35	1	1	Serviceability	Permanent	Self weight	Water pressure	Circular slip failure of ground	Resistant moment
					2		Variable	Variable waves	Self weight, water pressure	Sliding and overturning of superstructure	Limit value of sliding Limit value of overturning
										Bearing capacity of foundation ground	Limit value of bearing capacity
								L1 earthquake ground motion	Self weight, water pressure	Bearing capacity of foundation ground	Limit value of bearing capacity

④ Breakwaters armored with wave-dissipating blocks

(a) The performance criteria and the design situations of breakwaters covered with wave-dissipating blocks is as shown in the **Attached Table 17**, except a failure probability in the permanent situation for self weight is $P_f = 2.0 \times 10^{-4}$.

Because the breakwater covered with wave-dissipating blocks is a structural type having a wave-dissipating structure, the **Public Notice, Article 34, Paragraph 1, Item (2) (**Usability related to Wave-dissipating Function) shall be applied to the performance criteria in addition to this criteria.

⑤ Gravity-type special breakwaters

(a) Upright wave-dissipating block type breakwaters

1) The performance criteria and the design situations of gravity-type breakwaters is as shown in the **Attached Table 17**, except a system failure probability in the variable situation for waves is $P_f = 2.1 \times 10^{-2}$. This standard limit value can be determined as the average safety level possessed by upright wave-dissipating block type breakwaters which were designed by using the conventional design method based on safety factors.

Because the upright wave-dissipating block type breakwater is a structural type having a wave-dissipating section, the **Public Notice, Article 34, Paragraph 1, Item (2)** (Usability related to Wave-dissipating Function) shall be applied as performance criteria in addition to this criteria.

(b) Wave-absorbing type caisson breakwaters

1) The performance criteria and the design situations of gravity-type breakwaters is as shown in the Attached Table 17, except a system failure probability in the variable situation for waves is $P_f = 2.0 \times 10^{-2}$. This standard limit value can be determined as the average safety level possessed by wave-absorbing type caisson breakwaters which were designed by using the conventional design method based on safety factors.

Because the wave-absorbing type caisson breakwater is a structural type having a wave-dissipating structure, the **Public Notice, Article 34, Paragraph 1 Item (2)** (Usability related to Wave-dissipating Function) shall be applied as performance criteria in addition to this criteria.

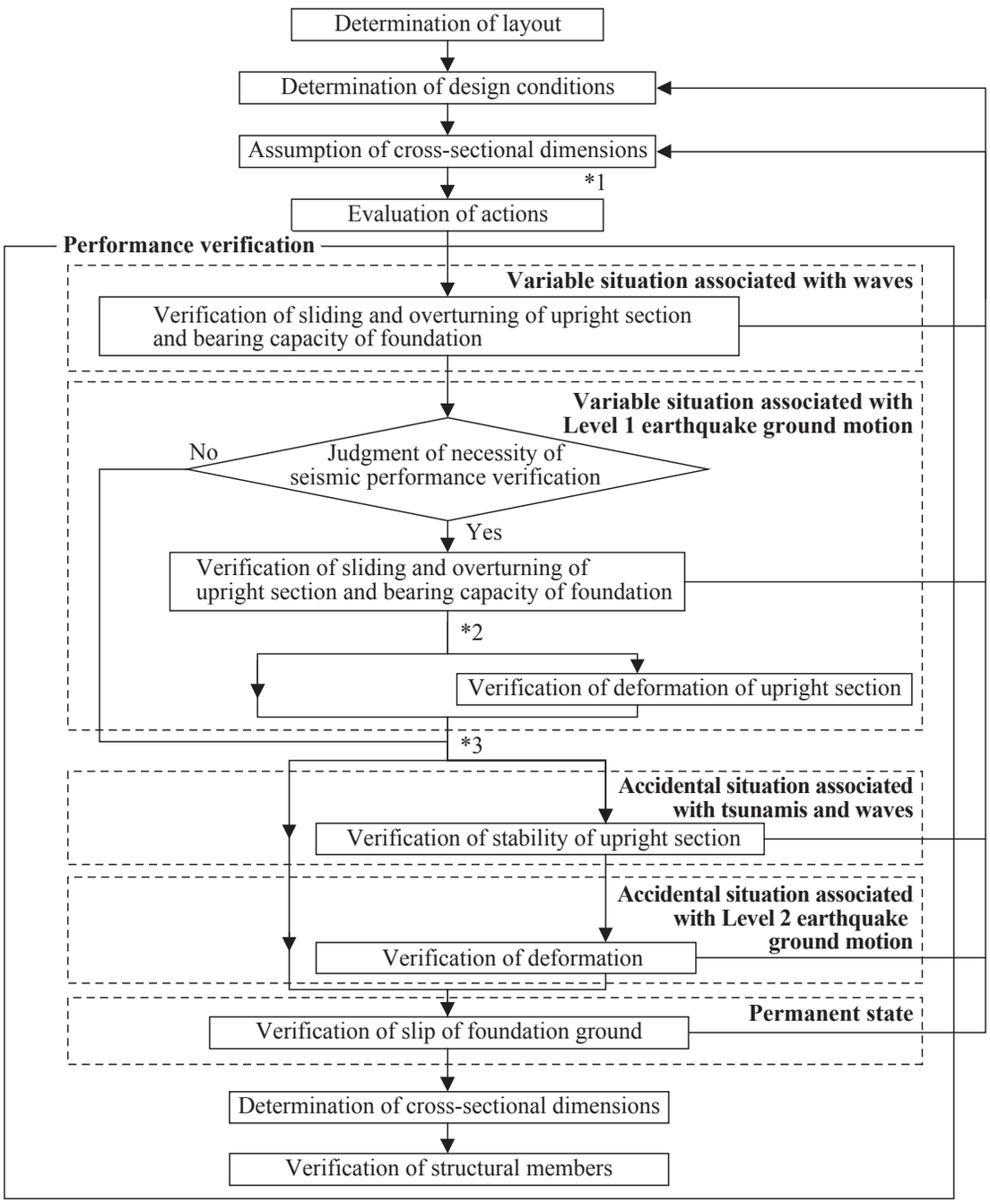
(c) Sloping-top caisson breakwaters

1) The performance criteria and the design situations of sloping-top caisson breakwaters is as shown in the **Attached Table 17**, except a system failure probability in the variable situation for waves is $P_f = 1.5 \times 10^{-2}$. This standard limit value can be determined as the average safety level possessed by sloping-top caisson breakwaters which were designed by using the conventional design method based on safety factors.

[Technical Note]

3.1.1 Principals of Performance Verification

(1) An example of the performance verification procedure for composite breakwaters is shown in **Fig. 3.1.1**. Because the assessment of the effect of liquefaction due to ground motion is not shown in the figure, it is necessary to conduct an appropriate examination as to whether or not liquefaction can be expected and the countermeasures for liquefaction referring to **Part II, Chapter 6 Ground Liquefaction**. The detailed procedure for judging the necessity of the performance verification of seismic-resistant shall be as shown in **3.1.4 Performance Verification**, **(11) Judgment of Necessity of Performance Verification of Seismic-resistant**.



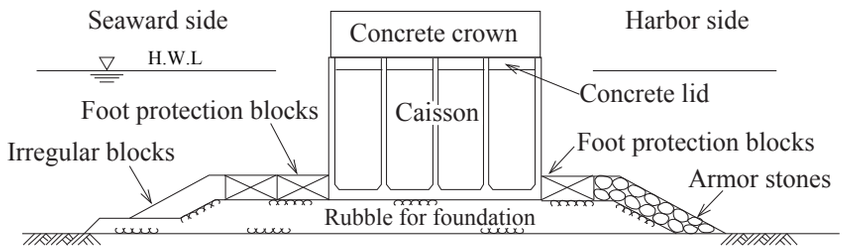
*1: The evaluation of the effects of liquefaction and settlement are not shown, so this must be separately considered.

*2: The analysis of deformation due to Level 1 earthquake ground motion 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.

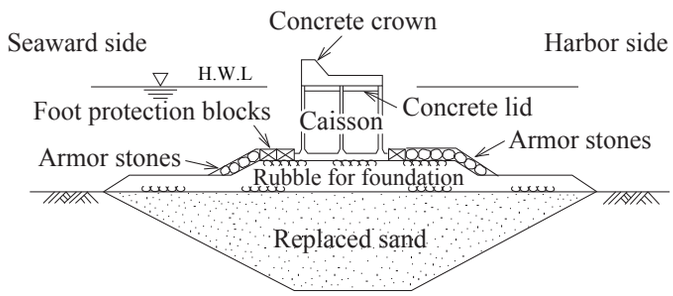
*3: For facilities where damage to the objective facilities is assumed to have a serious impact on life, property, and social activity, it is preferable to conduct a verification for the accidental situations when necessary. Verification for accidental situations associated with waves shall be conducted 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. 3.1.1 Example of Performance Verification Procedure for Composite Breakwaters

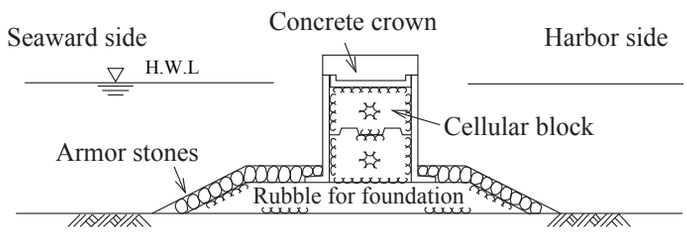
(2) Examples of the cross sections of composite breakwaters are shown in **Fig. 3.1.2**.



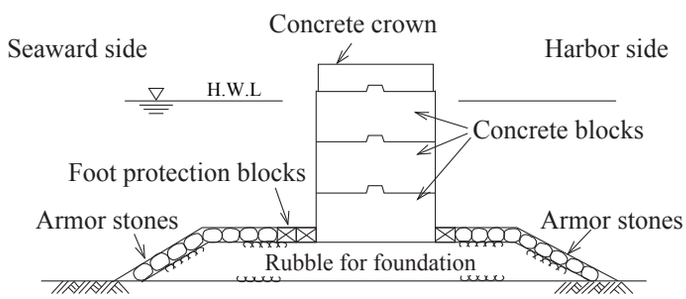
(a) Caisson type composite breakwater (sandy ground)



(b) Caisson type composite breakwater (soft ground)



(c) Cellular block type composite breakwater



(d) Concrete block type composite breakwater

Fig. 3.1.2 Examples of Cross Sections of Composite Breakwaters

3.1.2 Actions

(1) The design tidal level when calculating wave force is generally examined in the condition in which the facilities are most unstable. Specifically, in harbors where it is not necessary to consider the effect of storm surge, the mean monthly-highest water level and mean low water level are assumed, and in harbors where consideration of storm surge is necessary, an appropriate deviation is added to the mean monthly-highest water level and mean low water level. For slip failure of the ground, the mean monthly-lowest water level is used, and for calculations of settlement, the mean water level is used. Caution is required in the performance verifications, as there are cases in which the most dangerous water level differs depending on the verification items and object of verification.

(2) The wave parameters necessary in the performance verifications are the wave height, wave direction, wavelength, period, etc. In determining these parameters, **Part II, Chapter 2, 4 Waves** can be used as reference. For data on wind for use in wave hindcasting, **Part II, Chapter 2, 2 Winds** can be used as reference. It may be noted that data on wind are necessary in calculating wind pressure when designing lighthouses. The duration of waves is also considered to be an element which affects the stability of breakwaters. However, at present, this has not been adequately clarified. Therefore, caution is necessary, as damage to breakwaters facing the open sea and in particular, damage to the breakwater mound, would appear to be due to the effect of repeated waves over an extended period of time. Furthermore, because there are also cases in which facilities are damaged during construction, it is necessary to decide the parameters for waves during construction considering the construction plan and construction process.

(3) If the crest of the rubble mound is high and the berm width of the rubble mound is moderately wide, there are cases in which these conditions induce impulsive breaking wave force. Due caution should be paid in connection with the occurrence of impulsive breaking wave force, referring to **Part II, Chapter 2, 4.7.2 Wave Forces on Upright Walls**. Because there are cases in which the intensity of wave pressure will increase if the crown height of the breakwater is increased, caution is also necessary in this case.

(4) In the performance verifications, there are cases in which the wave that induces the greatest danger to the upright section differs from the most dangerous wave in mass calculations for armor units; therefore, caution is necessary.

(5) In cases where the still water level differs inside and outside the breakwater, it is preferable to consider the hydrostatic pressure equivalent to that difference in water level.

(6) It is necessary to consider the buoyancy of the breakwater body below the still water level. When the still water level differs inside and outside the breakwater, buoyancy can be considered for the breakwater body below the water surface joining the water levels on the two sides of the breakwater.

(7) In cases where erosion, sedimentation, changes in the gradient of the sea bottom can be expected after construction of a breakwater, the effects of those phenomena should also be considered.

(8) For dynamic water pressure during earthquakes, **Part II, Chapter 5, 2.2 Dynamic Water Pressure** can be used as a reference.

3.1.3 Setting of Basic Cross Section

(1) In cases where the foundation ground is soft and settlement can be expected, the crown height should include a height margin in advance, or a structure whose height can easily be increased should be adopted.

(2) In cases where the foundation ground is soft and remarkable settlement or extensive sinking of the rubble is conceivable, countermeasures should be taken, such as soil improvement, use of mattresses under the rubble mound to disperse actions from the body of the breakwater.

(3) The thickness of the concrete crown should be 1.0 m or more in situations where the design significant wave height is 2 m or greater, and is desired at least 50 cm when the design significant wave height is less than 2 m to avoid its destruction by overtopped waves.

(4) If the height of the caisson top is low, constraints will be encountered on caisson placement, sand filling, and placement of the concrete lid and concrete crown. Therefore, the height of the top of caissons is generally set higher than the mean monthly-highest water level. In case of block type breakwaters, it is preferable that the height of the top of the uppermost layer of blocks or cellular blocks be set at least higher than the mean water level (M.W.L.), and if possible, higher than the mean monthly-highest water level, so as to facilitate construction of the superstructure works.

(5) It is preferable that the water depth of the crest of the rubble section be as deep as possible in order to avoid the action of impulsive breaking wave force. Provided, however, that in the case of caissons, the upright section shall be set at a depth at which installation is possible. The mound width on the seaward side of the rubble mound should be sufficiently wide, depending on the wave height, paying attention to reduce the unfavorable effect of the action of impulsive breaking wave force as much as possible, in referring to **Part II, Chapter 2, 4.7.2(4) Impulsive Breaking Wave Force.**

(6) The berm width of the rubble mound shall be set so as to secure the specified stability against slip failure of ground and eccentric and inclined loads. In addition, it is preferable that the berm width on the seaward side be set to a width of at least 5m or more in a condition that does not include the footing, paying attention to reduce the favorable effect of the action of impulsive breaking wave force. However, this shall not apply in the case of hybrid caissons and other special structural types. On the harbor side, a berm width on the order to 2/3 that at the seaward side is acceptable. If this berm width is satisfied, it shall be assumed that the structure demonstrates the standard

strength constants c'=20kN/m^2, ϕ=35° for rubble mound in the simplified Bishop method used in the verifications of stability for eccentric and inclined loads. The partial factors used in the performance verifications are all values for cross sections having an adequate berm width. Caution is necessary when the berm width is narrow, as it is considered that the structure cannot demonstrate the standard strength constants. Reference equations for the harbor-side berm width BM_2 include equation **(3.1.1)** as proposed by Yoshioka et al.[1)] and others.

$$BM_2 = 1.0 + 0.2H_{1/3} + 0.3(H_C + T_U) + 0.2B_C \tag{3.1.1}$$

where

$H_{1/3}$: significant wave height (m)
H_C : caisson height (m)
T_U : thickness of superstructure work (m), in structures having a parapet, the parapet is not included
B_C : width of breakwater body (m), in structures having a footing, the footing is not included

(7) A high rubble backing is effective for increasing the sliding resistance of the upright section. However, caution is necessary in this case, as the rubble is easily scattered by overtopped waves. When necessary, it is preferable to provide armor using cubic blocks or deformed blocks. In the performance verifications, an appropriate performance verification shall be performed, referring to **3.1.4 (8) When Harbor Side of Upright Section is Strengthened**, which is presented below.

(8) A rubble mound foundation is effective to spread broadly the weight of the upright section, to provide a level ground where the upright section is placed, and to prevent scouring by waves. To achieve these functions, the thickness of rubble mound is desired to be 1.5 m or greater.

(9) The slope gradient of the rubble mound foundation is determined based upon the calculation of stability. In many cases, the seaward side of the breakwater normally may be a gradient between 1: 2 to 1: 3, and the harbor side may be a gradient between 1: 1.5 to 1: 2, depending upon wave conditions.

3.1.4 Performance Verification

(1) Items to be Considered in Verification of Stability of Composite Breakwaters

The composite breakwaters are structures in which stability is maintained by the weight of the breakwater body, so that the following items are generally examined.

① Sliding of the upright section
② Overturning of the upright section
③ Bearing capacity of the foundation ground
④ Slip failure of the ground
⑤ Settlement
⑥ Stability against Level 1 earthquake ground motion

For the partial factors used in the performance verification of these items, see **Table 3.1.1** in **(6) Performance Verification and Partial Factors for Sliding, Overturning, Foundation Failure, and Circular Slip Failure**.

The performance verifications in connection with accidental situations associated with Level 2 earthquake ground motion shall be conducted in accordance with **(13) Performance Verification for Level 2 earthquake ground motion**. The performance verifications in connection with accidental situations in respect of tsunamis shall be conducted in accordance with **(14) Performance Verification for Tsunamis**.

(2) Examination of Sliding of Breakwater Body

① In examination of the stability of the breakwater body against sliding, equation **(3.1.2)** can be used. In the following, the subscript d denotes design values.

$$f_d(W_d - P_{B_d} - P_{U_d}) \geq P_{H_d} \tag{3.1.2}$$

where

f : friction coefficient between bottom of body and foundation
W : weight of body (kN/m)
P_B : buoyancy (kN/m)
P_U : uplift force (kN/m)
P_H : horizontal wave force (kN/m)

The design values in the equation can be calculated using the following equations. In the following, the

symbol γ is the partial factor for its subscript, and the subscripts k and d denote the characteristic value and design value, respectively.

$$\begin{aligned} f_d &= \gamma_f f_k \\ P_{U_d} &= \gamma_{P_U} P_{U_k} \\ P_{H_d} &= \gamma_{P_H} P_{H_k} \end{aligned} \tag{3.1.3}$$

The design value W_d of the weight of the breakwater body can be calculated by the following equation, using the characteristic value W_{RC} of the weight of reinforced concrete, the characteristic value W_{NC} of the weight of non-reinforced concrete, and the characteristic value W_{SAND} of the weight of the filling sand.

$$W_d = \sum \gamma_W W_k \tag{3.1.4}$$

In cases where a caisson has a footing with a rectangular cross section extending to both the seaward and landward sides, the following equation can be used in calculating the design value P_{Bd} of buoyancy.

$$P_{B_d} = \rho_w g \left\{ \left(\gamma_{wl} wl_k + h \right) B_C + 2h_f B_f \right\} \tag{3.1.5}$$

where

$\rho_w g$: unit weight of sea water (kN/m³)
w_l : water 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)

It is preferable to determine the tidal level by calculating the ratio (hereinafter, r_{wl}) of the highest high water level, H.H.W.L., and the mean monthly-highest water level, H.W.L., based on the records of observation of tidal levels. However, at harbors where tidal levels are not monitored, r_{wl} for object harbor may be set referring to the distribution of r_{wl} shown in **Fig. 3.1.3**, and the partial factors may be selected from **Table 3.1.1**.

Fig. 3.1.3 Distribution of r_{wl} 2)

② In calculations of wave force, **Part II, Chapter 2, 4.7.2 Wave Forces Acting on Upright Walls** can be used as reference.

③ In order to increase the friction coefficient between the upright section and the rubble mound surface, there are cases in which friction enhancement mats are laid at the bottom of the upright section. For friction enhancement mats, **Part II, Chapter 11, 9 Friction Coefficient** can be used as reference.

(3) Examination of Overturning of Breakwater Body

In examination of the stability of the breakwater body against overturning, equation **(3.1.6)** can be used. In the following, the symbol γ is the partial factor for its subscript, and the subscripts k and d denote the characteristic value and design value, respectively.

$$a_1 W_d - a_2 P_{B_d} - a_3 P_{U_d} \geq a_4 P_{H_d} \tag{3.1.6}$$

where

W : weight of 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), see **Fig. 3.1.4**

The design values P_{Hd} and P_{Ud} of the wave force in equation **(3.1.6)** can be calculated using equation **(3.1.3)**; the design value W_d of the weight of the breakwater body can be calculated using equation **(3.1.4)**. In cases where a caisson has a footing with a rectangular cross section extending to both the seaward and landward sides, equation **(3.1.5)** can be used in calculating the design value P_{Bd} of buoyancy.

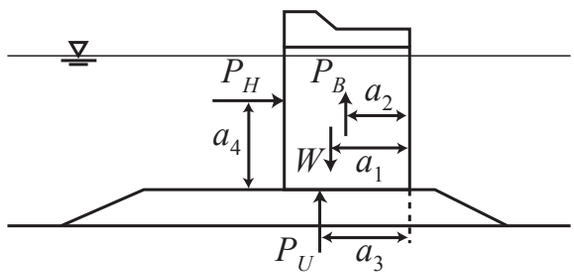


Fig. 3.1.4 Arm Lengths when Calculating Moments

(4) Examination of Bearing Capacity of Foundation Ground

① Examination of the stability against foundation failure at the bottom of the upright section can be conducted in accordance with **Chapter 2, 2.2.5 Bearing Capacity for Eccentric and Inclined Actions**. As standard partial factors for use in the performance verification, the values shown in **Table 3.1.1** can be used.

② In examinations of the bearing capacity of foundation ground, equation **(3.1.7)** can be used. The method shown here is the simplified Bishop method, and is one method of calculating circular slip by the discrete method. The simplified Bishop method is adopted because it is the model which can best explain stability with respect to bearing capacity, in comparison with the modified Fellenius method and friction circle method, by experiments in a centrifugal field.[4] However, deformation experiments with rubble mounds under the action of eccentric and inclined loads have demonstrated that when a rubble mound fails, the sliding surface does not necessarily occur along the circular arc with the lowest stability against slip failure. Caution is also necessary in adoption of this method as numerical analysis using the discrete element method has shown that the actual failure mechanism is different from circular slip failure according to the simplified Bishop method.[5]

In the following equation, the symbol γ is the partial factor for its subscript, and the subscript d denote the characteristic value.

$$\sum\left[\left\{c'_d s+\left(w'_d+q_d\right)\tan\phi'_d\right\}\sec\theta\Big/\left(1+\tan\theta\tan\phi'_d/F_f\right)\right] \Big/ \left[\gamma_a\ \left[\sum\left\{\left(w'_d+q_d\right)\sin\theta\right\}+a_1 P_{H_d}/R\right]\right]=F_f\geq 1.0 \tag{3.1.7}$$

where

P_H : horizontal wave force (kN/m)
a_1 : arm length of horizontal wave force (m)
c' : for cohesive soil ground, undrained shear strength, and for sandy ground, apparent cohesion in drained condition (kN/m^2)
s : width of slice segment (m)
w' : weight of slice segment (kN/m)
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)
γ_a : structural analysis factor

The design values in the equation can be calculated using the following equations.

$$
\begin{aligned}
c'_d &= \gamma_{c'} c'_k \\
w'_d &= \gamma_{w'} w'_k \\
q_d &= \gamma_q q_k \\
\tan\phi'_d &= \gamma_{\tan\phi'} \tan\phi'_k \\
P_{H_d} &= \gamma_{P_H} P_{H_k}
\end{aligned}
\tag{3.1.8}
$$

③ For the load width $2b'$ of the surcharge, adopting the average value, using the biases of the average value of the design parameters, is standard. In addition, the partial factor γ_q of the surcharge is set for the average value and not for the characteristic value. These calculations can be performed using the following equation **(3.1.9)** and equation **(3.1.10)**. In these equations, μ denotes the average value of the parameter of the subscript, and μ/X_k denotes the bias (average value/characteristic value) of the average value of the parameter X.

$$
\begin{aligned}
\overline{2b'} &= \frac{2\sum \overline{M}}{\sum \overline{V}} = \frac{2\left\{\left(a_1 \sum \overline{W} - a_2 \overline{P_B} - a_3 \overline{P_U}\right) - a_4 \overline{P_H}\right\}}{\sum W - \overline{P_B} - \overline{P_U}} \\
&= \frac{2\left\{\left(a_1 \sum \frac{\mu_W}{W_k} W_k - a_2 \frac{\mu_{P_B}}{P_{B_k}} P_{B_k} - a_3 \frac{\mu_{P_U}}{P_{U_k}} P_{U_k}\right) - a_4 \frac{\mu_{P_H}}{P_{H_k}} P_{H_k}\right\}}{\sum \frac{\mu_W}{W_k} W_k - \frac{\mu_{P_B}}{P_{B_k}} P_{B_k} - \frac{\mu_{P_U}}{P_{U_k}} P_{U_k}}
\end{aligned}
\tag{3.1.9}
$$

$$
\overline{q} = \sum \overline{V} \Big/ \overline{2b'} \tag{3.1.10}
$$

where

W_i : weight of parts comprising 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)

In the equation, $\overline{X}$ denotes the average value of the parameter X. The bias of the average value of buoyancy can be calculated using equation **(3.1.11)**. In **Table 3.1.1**, the bias of the average value of tidal levels is assumed to be 1.00; therefore, here, $\mu_{PB}/P_{Bk} = 1.00$ should be used.

$$
\frac{\mu_{P_B}}{P_{B_k}} = \frac{\left(\frac{\mu_{wl}}{wl_k} wl_k + h\right) B_c + 2h_f B_f}{\left(wl_k + h\right) B_c + 2h_f B_f} \tag{3.1.11}
$$

where

w_l : tidal 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)

(5) Examination for Slip of Ground

① It is necessary to conduct an examination of stability with respect to slip failure referring to **Chapter 2, 3.2.1 Stability Analysis by Circular Slip Failure Surface**, considering the characteristics of the ground and the characteristics of the structure.

② In case soil improvement is to be performed, **Chapter 2, 4 Soil Improvement Methods** can be used as reference.

③ As the tidal level used in examination of slip failure of the ground, it is preferable to use the tidal level which is most dangerous for the facilities. In determination of the tidal level, **Part II, Chapter 2, 3 Tidal Level** can be used as reference.

④ Verification of circular slip failure of the foundation ground in the permanent situation for self weight can be conducted using equation **(3.1.12)**. In the following, the symbol γ is the partial factor for its subscript, and the subscripts k and d denote the characteristic value and design value, respectively.

$$\sum\left[\left\{c'_d s+(w'_d+q_d)\cos^2\theta\tan\phi'_d\right\}\sec\theta\right]\geq\sum\left\{\left(w'_d+q_d\right)\sin\theta\right\} \tag{3.1.12}$$

where

c' : for cohesive soil ground, undrained shear strength, and for sandy ground, apparent cohesion in drained condition (kN/m2)
s : width of slice segment (m)
w' : weight of slice segment (kN/m)
q : spatially distributed load acting on slice segment, obtained by dividing effective weight of breakwater body by width of breakwater body (kN/m)
ϕ' : apparent angle of shear resistance based on effective stress (°)
θ : angle formed by slice segment with bottom (°)

The design values in the equation can be calculated using the following equations.

$$c'_d=\gamma_{c'}c'_k$$
$$q_d=\gamma_q q_k$$
$$\tan\phi'_d=\gamma_{\tan\phi'}\tan\phi'_k \tag{3.1.13}$$

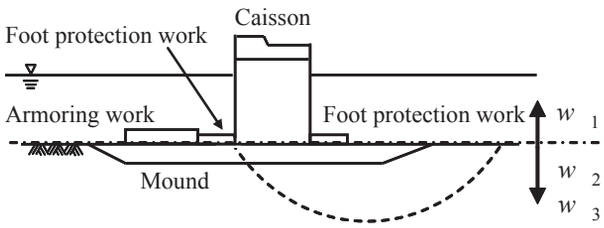
When all of the soil layers are below water level, the design value w'_d of the weight of the slice segments can be calculated using equation **(3.1.14)**. Because the unit weights of the soil layers and mound used when calculating the weight of the slice segments contribute to both the action side and the resistance side, the unit weights of the soil layers and mound are classified as w_1, w_2, and w_3, considering their positional relationship, and the partial factors γ_1, γ_2, and γ_3 are set for each, respectively. Caution is necessary with regard to the soil layers and mound falling under these divisions, as the values will differ depending on the position of the mound as shown in **Fig. 3.1.5**.

$$w'_d=\sum_i\left(\gamma_{w_{ni}}w_{ni_k}-P_{Bi_d}\right) \tag{3.1.14}$$

where

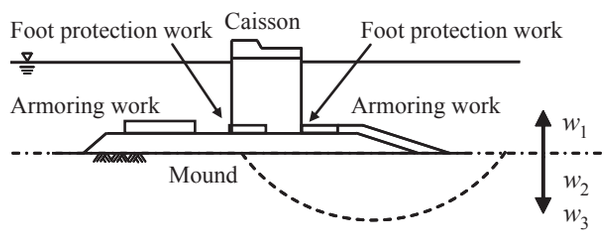
w' : weight of slice segment (kN/m)
w_{ni} : unit weight of soil layer comprising slice segment (kN/m)
n : shows number of soil layers (n = 1, 2, 3; see **Fig. 3.1.5**)
P_{Bi} : buoyancy acting on slice segment being considered (kN/m)

In calculating the characteristic value of buoyancy, equation **(3.1.5)** can be used as reference, excluding the terms in connection with the footing.



Division of unit weight	Soil layer, mound, etc.
w_1	Caissons, armoring work, foot protection work, wave-dissipating work, above level of sea bottom
w_2	Sandy soil layer below level of mound and sea bottom
w_3	Cohesive soil layer below level of sea bottom

(1) When position of mound is lower than level of sea bottom



Division of unit weight	Soil layer, mound, etc.
w_1	Caissons, mound, armoring work, foot protection work, wave-dissipating work, above level of sea bottom
w_2	Sandy soil layer below level of sea bottom
w_3	Cohesive soil layer below level of sea bottom

(2) When position of mound is higher than level of sea bottom

Fig. 3.1.5 Classification of Weight of Slice Segments

(6) Performance Verification and Partial Factors for Sliding, Overturning, Foundation Failure, and Circular Slip Failure

① For the standard system failure probability of sliding, overturning, and foundation failure of the upright section of composite breakwaters in variable situations due to the action of waves, and the partial factors for the standard failure probability for circular slip failure in the permanent situation, the values shown in **Table 3.1.1** can be used as reference 3), 6). The standard system failure probability for sliding and overturning of the upright section of composite breakwaters, and for the bearing failure of the foundation ground, has been obtained based on evaluation by reliability theory for the average safety level of breakwaters designed by the conventional design method.

For circular slip failure, a value of 3.3, converted to failure probability, 4.5×10^{-4}, is set as the reliability index which minimizes the expected total cost. Here, the expected total cost is expressed by the sum of the initial construction cost and the expected value of the recovery cost due to failure.

If the safety level based on minimization of the expected total cost is evaluated by reliability theory, the partial factors are as shown in **Table 3.1.1 b).** If based on the average value of the safety levels in the design methods of the past, the reliability index is 6.5, failure probability: 3.1×10^{-11}. For details, Reference **6)** can be used as reference.

② In the table, α, μ/X_k, and V are the sensitivity factor of each design parameter, bias of the average value, and coefficient of variation, respectively.

③ For the partial factors in connection with circular slip failure, when the soil under the breakwater body is improved by the sand compaction pile (SCP) method with a replacement ratio of 30–80%, the partial factors shown in **4.10.6 Performance Verification** for the sand compaction pile method in **Chapter 2, 4 Soil Improvement Methods** shall be used.

Table 3.1.1 Standard Partial Factors

(a) Variable situations associated with waves

Target system reliability index β_T			2.38			
Target system failure probability P_{fT}			8.7×10^{-3}			
Target reliability index β_T' used in calculation of γ			2.40			
			γ	α	μ/X_k	V
Sliding	γ_f	Friction coefficient	0.79	0.689	1.060	0.150
	γ_{P_H}, γ_{P_U}	Change of water depth: Mild	1.04	-0.704	0.740	0.239
		Change of water depth: Steep	1.17		0.825	0.251
	γ_{wl}	r_{wl} = 1.5	1.03	-0.059	1.000	0.200
		r_{wl} = 2.0, 2.5	1.06		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	0.98	0.030	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.02	0.025	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.01	0.150	1.020	0.040
Overturning	γ_{P_H}, γ_{P_U}	Change of water depth: Mild	1.15	-0.968	0.740	0.239
		Change of water depth: Steep	1.31		0.825	0.251
	γ_{wl}	r_{wl} = 1.5	1.04	-0.092	1.000	0.200
		r_{wl} = 2.0, 2.5	1.09		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	0.98	0.044	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.02	0.040	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	0.232	1.020	0.040
Bearing capacity of foundation ground	γ_{P_H}	Change of water depth: Mild	1.12	-0.894	0.740	0.239
		Change of water depth: Steep	1.26		0.825	0.251
	γ_q	Surcharge on slice segment	0.91	0.640	0.605	0.061
	$\gamma_{w'}$	Weight of slice segment	1.00	0.032	1.000	0.030
	$\gamma_{\tan\phi'}$	Ground strength: Tangent of angle of shear resistance	0.96	0.288	1.000	0.059
	$\gamma_{c'}$	Ground strength: Cohesion	0.99	0.072	1.000	0.059
	γ_a	Structural analysis factor	1.00	–	–	–

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: RC: reinforced concrete, NC: non-reinforced concrete.
*3: Change of water depth Mild/Steep: Gradient of sea bottom <1/30/≧1/30.
*4: r_{wl} denotes the ratio of the highest high water level (H.H.W.L.) and mean monthly-high water level (H.W.L.).
*5: γ_q is applied to the average value of the surcharge. The average value of the surcharge is obtained using $\bar{q} = \sum \bar{V} / 2b'$.
*6: In calculations of wave force, Goda's formulas is used.

Table 3.1.1 Standard Partial Factors

(b) Permanent situation

Target reliability index β_T				3.3			
Target failure probability P_{fT}				4.5×10^{-4}			
				γ	α	μ/X_k	V
Circular slip failure	$\gamma_{c'}$		Ground strength: Cohesion	0.90	0.285	1.00	0.038
	$\gamma_{\tan\phi'}$		Ground strength: Tangent of angle of shear resistance	0.90	0.380	1.00	0.038
	γ_{w_i}	When mound is positioned below level of sea bottom	1 Wave-dissipating work, etc. above level of sea bottom	1.00	-0.007	1.00	0.03
			2 Sandy soil below mound and level of sea bottom	0.90	0.070		
			3 Clayey soil below level of sea bottom	0.90	0.125		
		When mound is positioned above level of sea bottom	1 Mound, wave-dissipating work, etc. above level of sea bottom	1.00	-0.007	1.00	0.03
			2 Sandy soil below level of sea bottom	0.90	0.070		
			3 Clayey soil below level of sea bottom	0.90	0.125		
	γ_q		Spatially-distributed load	1.10	-0.463	1.02	0.04

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: γ_{w1}, γ_{w2}, and γ_{w3} are partial factors for the weight of the slice segment; classification follows that in **Fig. 3.1.5**.
*3: Wave-dissipating work, etc. includes wave-dissipating work, armoring work, foot protection work, etc.
*4: In application of the partial factors for circular slip failure, reference shall be made to the notes shown in **Chapter 2, 3 Stability of Slope**, **3.1(7) Partial Factors**. When soil is improved by the sand compaction pile (SCP) method with a replacement ratio of 30–80%, the partial factors shown in **4.10.6 Performance Verification** for the sand compaction pile method in **Chapter2, 4 Soil Improvement Methods** shall be used.

(7) Reliability-based Design Methods Considering Sliding Displacement

The performance verification method based on partial factors shown in (6) is a reliability-based design method based on the balance of forces basically limited to the design wave height. However, even in cross sections verified by this method, the probability that displacement will occur during the design working life is not 0, and furthermore, that probability will also differ depending on such features as the appearance of high waves and the water depth.

On the other hand, for the sliding mode, reliability-based design methods using the probability of appearance of displacement and the amount of displacement as indexes have also been proposed, and these methods of performance verification may also be used. Where the sliding stability of the breakwater body is concerned, Shimosako et al.[7] proposed a method of verification of the average sliding displacement, expected sliding displacement, of breakwaters during the design working life using the sliding model of the breakwater body proposed by Tanimoto et al.[8]

Table 3.1.2 shows an example of setting of the allowable values of the exceedence probability for composite breakwaters. When this method is used, the conditions of sliding displacement which determine the cross section will differ, depending on such features as the appearance of high waves and water depth. As a result, it is possible to set cross sections having approximately the same stability regardless of the design conditions. As the average value of the exceedence probability of a total sliding displacement of 30cm by the conventional design method, Reference **17)** can be used as reference. For examples of setting for breakwaters covered with wave-dissipating blocks, **3.4.3 Performance Verification** of **3.4 Gravity-type Breakwaters (Breakwaters Covered with Wave-dissipating Blocks)** can be used as reference.

Table 3.1.2 Example of Setting of Allowable Values of Exceedence Probability for Composite Breakwaters [16]

		Importance of facilities		
		High	Ordinary	Low
Sliding displacement	10cm	15%	30%	50%
	30cm	5%	10%	20%
	100cm	2.5%	5%	10%

(8) When Harbor Side of Upright Section is Strengthened

① When the harbor side of the upright section is strengthened with a mound of rubble stones or concrete blocks, careful attention must be paid to the following matters:

(a) The possibility of hindrance to ship navigation and mooring for within the harbor.

(b) In verification of the stability of the upright section for sliding and overturning ignoring strengthening section behind the breakwater, the design value of resistance assuming the partial factor is 1.0 must exceed the design value of the actions. If design value of resistance/design value of action is small, there will be a danger of violent rocking of the upright section, increase in the heel pressure, and sliding or overturning of the upright section to the seaward side during wave troughs.

(c) Adequate armoring must be provided so that the strengthening section will not be damaged by overtopped waves.

(d) The height of the strengthening section h should preferably be 1/3 or greater of the height of the upright section, and the width b should be the same as or greater than the height h.

(e) In the case of concrete block strengthening, construction should be made to ensure that there are no voids between the concrete blocks and the upright section.

② When the harbor side of the upright section is strengthened with rubble or blocks, if the height of the strengthening material a is greater than 1/3 of the height of the upright section, and the top width b is greater than height a, the performance verification for sliding can be conducted using equation **(3.1.15)**. In the following equation, the symbol γ is the partial factor for its subscript, and the subscripts k and d denote the characteristic value and design value, respectively.

$$f_d\left(W_d - P_{B_d} - P_{U_d}\right) + R_d \geq \gamma_a P_{H_d} \tag{3.1.15}$$

where

f : friction coefficient between bottom of breakwater body and foundation
W : weight of breakwater body (kN/m)
P_B : buoyancy (kN/m)
P_U : uplift force (kN/m)
P_H : horizontal wave force (kN/m)
γ_a : structural analysis factor
R : sliding resistance of strengthening rubble or blocks (kN/m)

Among the design values used in the equation, the design values of wave force P_{Hd} and P_{Ud} and the design value of the weight of the breakwater body W_d can be calculated using equation **(3.1.3)** and equation **(3.1.4)**, respectively. In cases where a caisson has a footing with a rectangular cross section extending to both the seaward and landward sides, equation **(3.1.5)** can be used in calculating the design value P_{Bd} of buoyancy. The design value of sliding resistance R_d can be calculated by the following equation.

$$R_d = \gamma_r R_k \tag{3.1.16}$$

The characteristic value of sliding resistance R_k can be calculated by the following method.

(a) Sliding resistance of rubble.

$$R_k = W_s \tan(\theta + \phi) \tag{3.1.17}$$

where

W_s : weight in water of rubble above sliding surface, excluding uppermost armor layer (kN/m)
θ : angle of sliding surface (°)
ϕ : $\phi = \tan^{-1} f_1$, f_1 is the coefficient of friction between rubble stones, f_1=0.8 (°)

(b) Takeda et al.[20] have shown experimentally that resistance force R can be expressed by equation **(3.1.18)**, based on the assumption that R is a function of the ratio of the wave height and breakwater installation depth, see **Fig. 3.1.6**.

$$R_k = \alpha W_s \tag{3.1.18}$$

Provided, however, that when $H/h' \leq 0.5$, H/h'=0.5.

where

W_s : weight in water of rubble or blocks (kN/m)

α : friction coefficient

Rubble : $\alpha = 0.9 + 0.2(H/h' - 0.5)$

Blocks : $\alpha = 0.4 + 0.2(H/h' - 0.5)$

H : wave height (m)

h' : installation depth of breakwater (m)

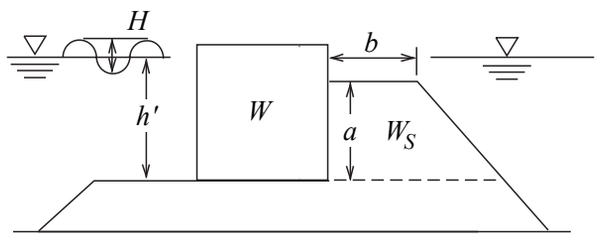


Fig. 3.1.6 Sliding Resistance Surface of Strengthening Section

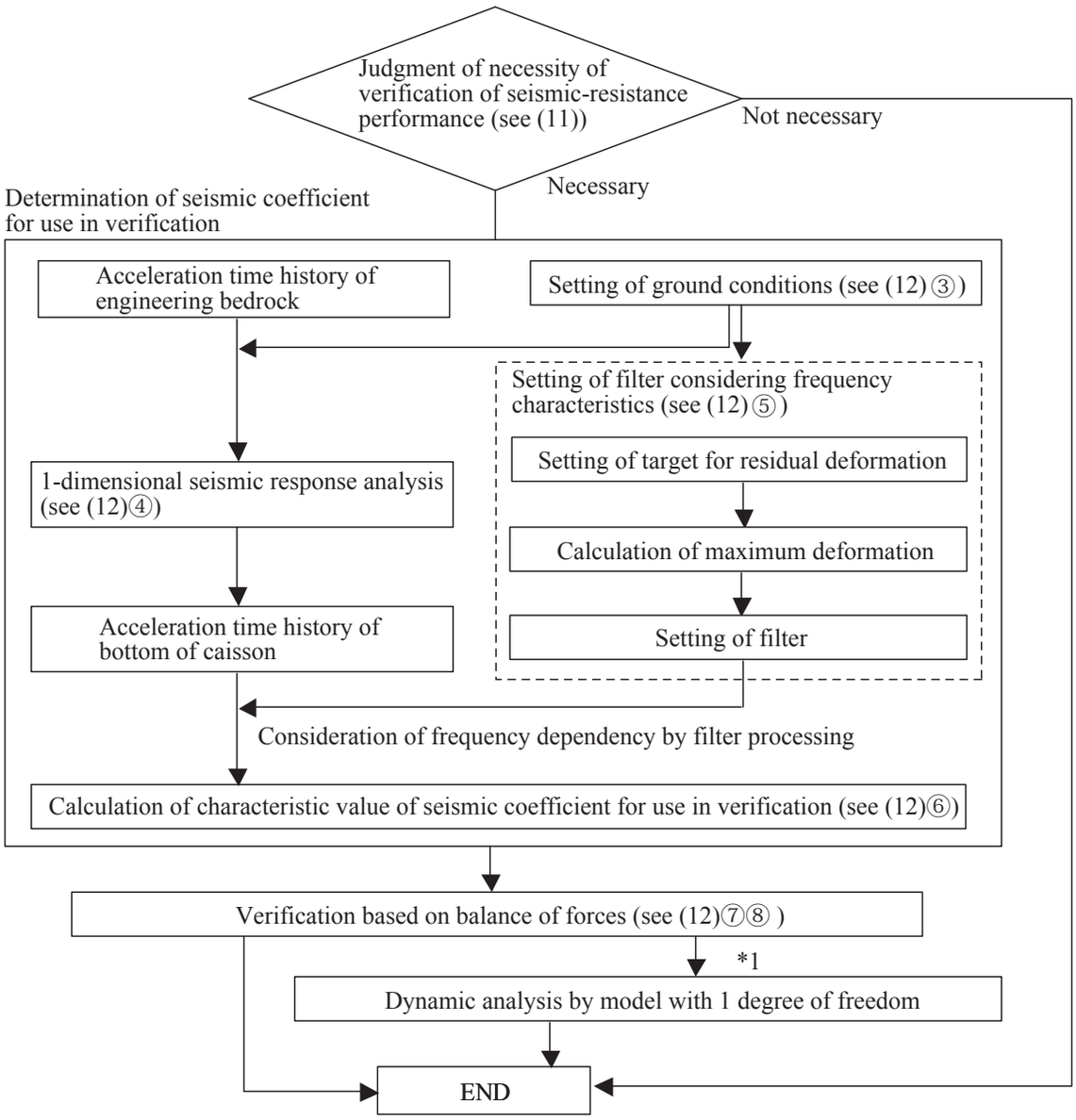
③ Regarding the bearing capacity of the foundation ground and slip failure of the ground when the harbor side of the upright section is strengthened, it is preferable to conduct an appropriate examination referring to the above-mentioned **(4) Examination of Bearing Capacity of Foundation Ground** and **(5) Examination of Slip of Ground**.

(9) All partial factors shown here are values when the design working life is the normal 50 years. When it is necessary to evaluate the stability of facilities during construction, verification must be conducted appropriately, considering the conditions in which the facilities are placed, the return period of the actions, and the relationship with the verification of the stability of the facilities when completed. In the performance verifications, the description in **3.4.4 (6)** can be used, as equivalent to breakwaters covered with wave-dissipating blocks.

(10) Performance Verification of Seismic-resistant

In general, the performance verification for Level 1 earthquake ground motion is frequently omitted with breakwaters. However, in cases where the installation depth is great and the design wave height is small, there are cases in which actions due to Level 1 earthquake ground motion become predominant. In such cases, performance verification of seismic-resistant is necessary.

The general procedure for performance verification of seismic-resistant of breakwaters is as shown in **Fig. 3.1.7**.



*1: For breakwaters where damage to the objective facilities is assumed to have a serious impact on life, property, and socioeconomic activity, it is preferable to confirm the amount of deformation by dynamic analysis.

Fig. 3.1.7 Example of Procedure Performance Verification of Seismic-resistant

(11) Judgment of Necessity of Performance Verification of Seismic-resistant [23), 24)]

For sliding and overturning due to Level 1 earthquake ground motion, the necessity of performance verification of seismic-resistant is decided from the relationship between the cross-sectional dimensions of the breakwater body determined in the variable situation in respect of waves and Level 1 earthquake ground motion. The judgment of necessity can be made based on **Fig. 3.1.8**, from the relationship between the maximum acceleration on the seismic bedrock and the ratio B_w/h of the breakwater body width B_w not including the footing and the water depth h (a condition in which the ratio of the resistance force and effect of actions is smallest). The performance verification of seismic-resistant can be omitted for cases where the maximum acceleration on the seismic bedrock is positioned below the curve in the figure. It should be noted that this figure is prepared assuming the allowable value of residual deformation of the upright section of the breakwater for Level 1 earthquake ground motion is 30cm. Therefore, if other allowable values are adopted, it is preferable to conduct a concrete verification of the deformation.

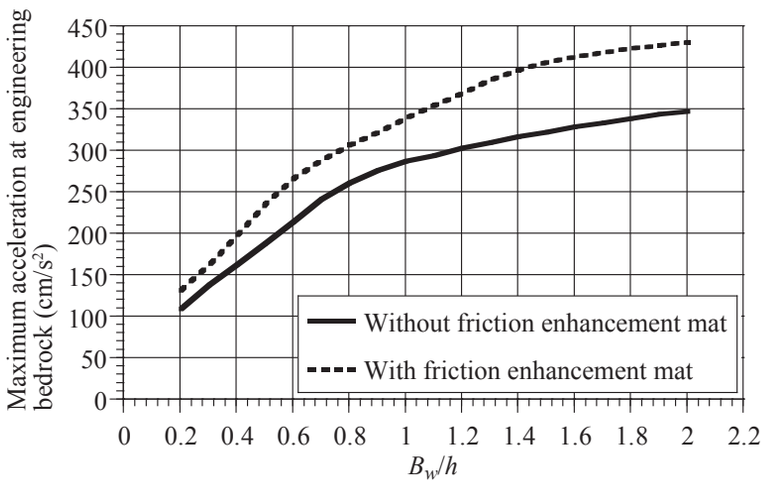


Fig. 3.1.8 Diagram of Judgment of Necessity of Performance Verification of Seismic-resistant

(12)Seismic Coefficient for Verification of Sliding, Overturning, and Bearing Capacity of Upright Section for Level 1 earthquake ground motion

① General

In the performance verifications for sliding and overturning of the upright section and failure due to insufficient capacity of the foundation ground in variable situations in respect of Level 1 earthquake ground motion, it is possible to evaluate whether performance is maintained by a direct evaluation of deformation by detailed methods such as dynamic analysis methods. However, verifications can also be performed by simplified methods such as the seismic coefficient method. In this case, the seismic coefficient for the verification which is to be used in the performance verification needs to be set appropriately, corresponding to the deformation of the facilities in question, considering the frequency characteristics of the ground motion. In general, the seismic coefficient for verification assumes Level 1 earthquake ground motion in the seismic bedrock as the input ground motion and is smaller than the seismic coefficient (α_{max}/g) obtained as the ratio of the maximum acceleration α_{max} in the acceleration time history of the bottom of the caisson obtained by a one-dimensional seismic response analysis and the gravitational acceleration g.

② An outline of the method of calculating the seismic coefficient for verification is shown in **Fig. 3.1.9**. First, the Level 1 earthquake ground motion in the seismic bedrock is set, and the acceleration time history at the bottom of the caisson is calculated by a one-dimensional seismic response analysis using this as the input ground motion. The result of a fast Fourier transform (FFT) of the acceleration time history obtained in this manner is multiplied by a filter which considers the frequency characteristics of the ground motion, and the acceleration time history at the bottom of the caisson after filter processing is calculated by performing an inverse fast Fourier transform (IFFT) on the result of the previous calculation. The characteristic value of the seismic coefficient for verification is then calculated using the maximum value of this acceleration time history.

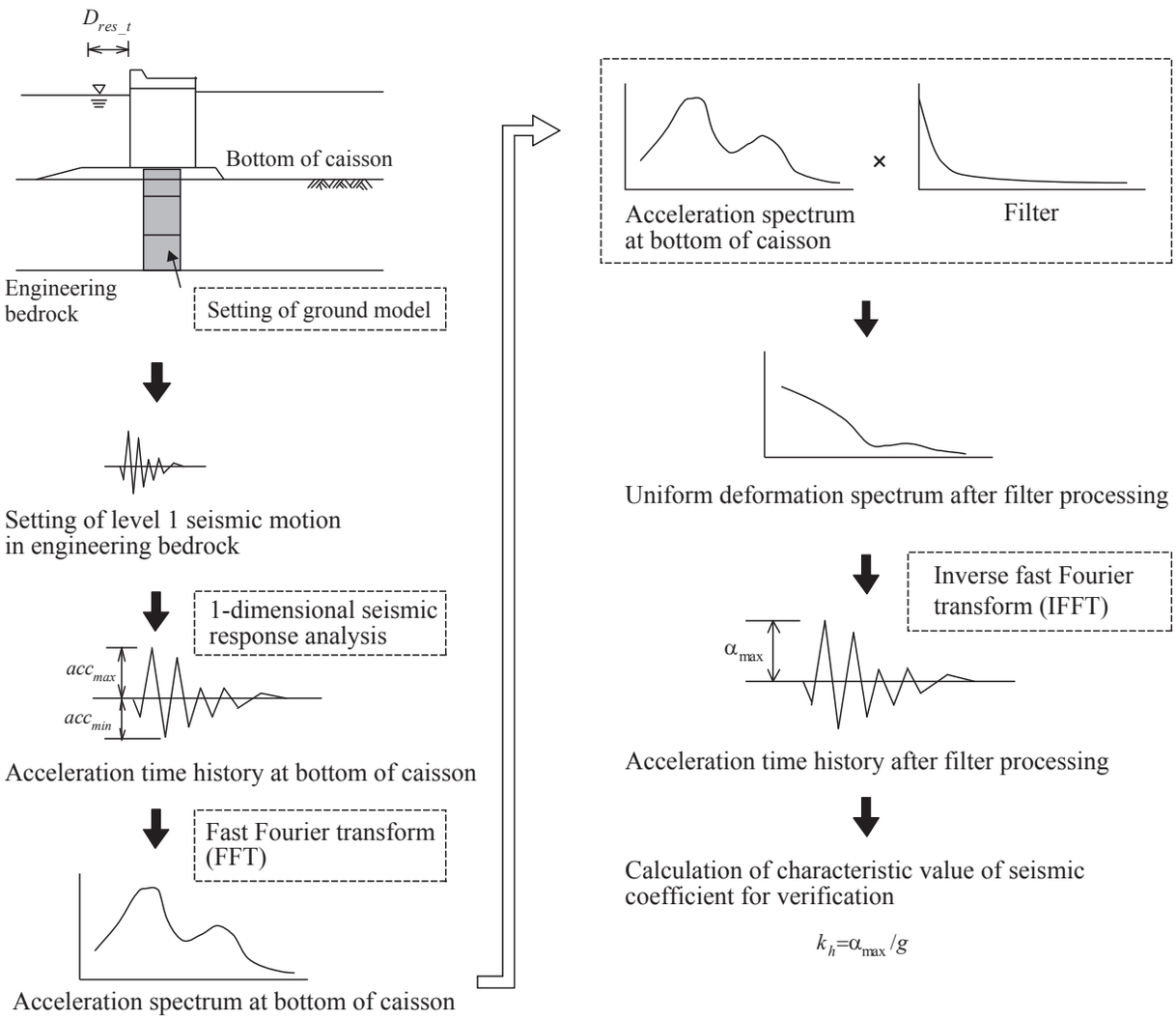


Fig. 3.1.9 Outline of Calculation Method of Seismic Coefficient for Verification

③ Setting of ground conditions

In calculation of the seismic coefficient for verification, it is necessary to set the ground conditions so as to enable an appropriate evaluation of the characteristics of the ground at the location concerned. In setting the ground conditions, **Part II, Chapter 3, Geotechnical Conditions**, **ANNEX 4, 1 Seismic Response Analysis of Local Soil Deposit**, and **Chapter 5, 2.2 Gravity-type Quaywalls (2.2.2(1)③ Setting of Geotechnical Conditions)** can be used as reference.

④ One-dimensional seismic response analysis

The acceleration time history at the bottom of caissons shall be calculated by a 1-dimensional seismic response analysis which can appropriately consider the features of the ground at the location concerned, assuming the Level 1 earthquake ground motion set for the seismic bedrock as the input ground motion. One-dimensional seismic response analysis shall be performed based on an appropriate technique and setting of the analysis conditions, referring to **ANNEX 4, 1 Seismic Response Analysis of Local Soil Deposit** and **Chapter 5, 2.2 Gravity-type Quaywalls (2.2.2(1) ③ Setting of Geotechnical Conditions)**.

⑤ Setting of filter considering frequency characteristics and deformation

(a) Setting of maximum deformation

In calculation of the seismic coefficient for verification for breakwaters, evaluation is not possible using residual deformation in its unmodified form as an index because the process of accumulation of deformation is different from that in quaywalls due to the effects of the frequency characteristics of the ground motion and the repetition of actions. Therefore, among ground motions, the maximum value of deformation when a certain wave acts is defined as the maximum deformation, and a filter is calculated in such a way that a constant value of the maximum deformation can be obtained independent of frequency. Because the relationship shown in equation **(3.1.19)** exists between the maximum deformation D_{max} and the target value of residual deformation

D_{res_t},depending on whether friction enhancement mats are used or not, the maximum deformation can be calculated if residual deformation is given. Here, the standard allowable value of deformation D_{res_t} of a breakwater for Level 1 earthquake ground motion can be given as D_{res_t} = 30cm. The shape of the filter in this case is as shown in **Fig. 3.1.10**.

$$D_{max} = \begin{cases} \dfrac{D_{res\_t}}{0.87R_{acc} + 0.52} & \text{(with friction enhancement mat)} \\ \dfrac{D_{res\_t}}{0.87R_{acc} + 0.44} & \text{(without friction enhancement mat)} \end{cases} \tag{3.1.19}$$

$$R_{acc} = \frac{\left|acc_{max} + acc_{min}\right|}{\max\left(acc_{max}, \left|acc_{min}\right|\right)}$$

where

D_{max} : maximum deformation (cm)
D_{res_t} : target value of residual deformation (D_{res_t} = 30cm)
acc_{max}, acc_{min} : the maximum acceleration and the minimum acceleration in acceleration time history of caisson bottom (cm/s^2)

(b) Setting of filter

The filter which considers the frequency characteristics of ground motion and amount of deformation for use in performance verification for seismic-resistant of breakwaters can be calculated by equation **(3.1.20)** using the maximum deformation obtained in the above **(a) Setting of maximum deformation**. This filter is obtained by evaluating the contribution of the waves of each frequency component comprising the ground motion to the deformation of the breakwater. This shows the relationship between the maximum deformation of the breakwater caisson which is the target and the maximum value of the input acceleration at the bottom of the caisson based on the results of a seismic response analysis for a system with one degree of freedom performed on multiple sine waves using models of quaywalls with different ground conditions and water depths.

$$F = \frac{1}{af^2 + bf + 1} \tag{3.1.20}$$

$$a = \begin{cases} 0.0145D_{max} - 0.022 & \text{(with friction enhancement mat)} \\ 0.0178D_{max} - 0.0035 & \text{(without friction enhancement mat)} \end{cases}$$

$$b = \begin{cases} 0.0074D_{max} + 0.8542 & \text{(with friction enhancement mat)} \\ 0.0095D_{max} + 0.8174 & \text{(without friction enhancement mat)} \end{cases}$$

where

F : filter for use in calculation of seismic coefficient for verification
f : frequency (Hz)
a, b : coefficients
D_{max} : maximum deformation (cm)

⑥ Calculation of characteristic value of seismic coefficient for verification

The seismic coefficient for verification to be used in the performance verification of breakwaters can be calculated by equation **(3.1.21)**.

$$k_h = \alpha_{max} / g \tag{3.1.21}$$

where

α_{max} : the maximum value of acceleration at caisson bottom after filter processing (cm/s^2)
g : gravitational acceleration (cm/s^2)

⑦ When conducting a performance verification based on the balance of forces, the performance verification can be performed using equation **(3.1.22)** and equation **(3.1.23)**. In this case, the cross section obtained in the variable situation is respect of waves can be used as the cross section for verification. The tidal level shall be the condition which gives the smallest ratio of the resistance force and the effect of actions. In the following

equations, the symbol γ is the partial factor for its subscript, and the subscripts k and d denote the characteristic value and design value, respectively.

(Sliding stability) $$\gamma_a \left(k_{h_d} W_d + 2P_{d_{wd}} \right) \leq \mu_d W_d' \quad (3.1.22)$$

(Overturning stability) $$\gamma_a \left(a_1 k_{h_d} W_d + 2a_2 P_{d_{wd}} \right) \leq a_3 W_d' \quad (3.1.23)$$

where

k_h : seismic coefficient for verification
W : weight of caisson (kN/m)
P_{dw} : resultant of dynamic water pressure (kN/m); calculated using equation **(3.1.25)**

$$P_{d_w} = \frac{7}{12} k_h \rho_w g H^2 \quad (3.1.24)$$

$\rho_w g$: unit weight of sea water (kN/m^3)
H : installation depth of caisson (m)
W' : effective weight of caisson in water ($= W - P_B$) (kN/m)
P_B : buoyancy (kN/m)
μ : friction coefficient between caisson and rubble mound; **Part II, Chapter 11, 9 Friction Coefficient** can be used as reference.
a_1–a_3 : arm lengths for actions (m)
γ_a : structural analysis factor

Here, the design value of the seismic coefficient for verification in equation **(3.1.22)** and equation **(3.1.23)** can be calculated by the following equation. For k_{hk}, the seismic coefficient for verification obtained by equation **(3.1.21)** can be used.

$$k_{h_d} = \gamma_{k_h} k_{h_k} \quad (3.1.25)$$

The design value of the weight of the breakwater body and the design value of the buoyancy acting on the breakwater body can be calculated using equation **(3.1.4)** and equation **(3.1.5)**, respectively.

Here, all of the partial factors with the exception of the structural analysis factors can be assumed to be 1.00, and the structural analysis factors for sliding and overturning can be assumed to be 1.2 and 1.1, respectively.

⑧ Verification of the bearing capacity can be performed referring to **Chapter 2, 2.2 Shallow Spread Foundations**, giving appropriate consideration to actions due to ground motion. For breakwaters in which stability with respect to the bearing capacity and settlement of the foundation ground due to Level 1 earthquake ground motion are major problems, it is preferable to conduct a detailed examination by dynamic analysis.

(13) Performance Verification for Level 2 earthquake ground motion

The performance verification in the accidental situation in respect of Level 2 earthquake ground motion is equivalent to that for the gravity-type quaywalls. Therefore, **Part III, Chapter 5, 2.2.3 (8) Performance Verification for Ground Motion (Detailed Methods)** can be used as reference. Provided, however, that the breakwaters are only affected by settlement, with the exception of cases where settlement is a problem, no verification is frequently necessary. A simplified method of predicting the amount of settlement from the results of a 1-dimensional analysis is proposed, and depending on the accuracy necessary in the predicted value of the settlement, it is also possible to substitute the simplified method.

(14) Performance Verification for Tsunamis

① In performance verifications for tsunamis, **6 Tsunami Protection Breakwaters** can be used as reference.

② Partial factors

For the partial factors for use in examination of the stability of the upright section of composite breakwaters in the accidental situation in respect of tsunamis against sliding and overturning and the stability against failure due to insufficient bearing capacity of the foundation ground, **Table 3.1.3** can be used as reference. Provided, however, that the values shown in **Table 3.1.3** are standard values which are used when setting the wave force of the largest class tsunami assumed at the construction location of the facilities as an accidental action. Accordingly, in cases where uncertainty is expected in calculation of the characteristic value of the tsunami force, the structural analysis factor should be set to an appropriate value of 1.0 or larger, as necessary.

Table 3.1.3 Partial Factors used in Performance Verification for Tsunamis

			γ	α	μ/X_k	V
Sliding	γ_f	Friction coefficient	1.00	–	–	–
	γ_{PH}, γ_{PU}	Tsunami force	1.00	–	–	–
	γ_{wl}	r_{wl}=1.5	1.00	–	–	–
		r_{wl}=2.0, 2.5	1.00		–	–
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	1.00	–	–	–
	$\gamma_{W_{NC}}$	Unit weight of NC	1.00	–	–	–
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	–	–	–
	γ_a	Structural analysis factor	1.00	–	–	–
Overturning	γ_{PH}, γ_{PU}	Tsunami force	1.00	–	–	–
	γ_{wl}	r_{wl}=1.5	1.00	–	–	–
		r_{wl}=2.0, 2.5	1.00		–	–
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	1.00	–	–	–
	$\gamma_{W_{NC}}$	Unit weight of NC	1.00	–	–	–
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	–	–	–
	γ_a	Structural analysis factor	1.00	–	–	–
Bearing capacity of foundation ground	γ_{PH}	Tsunami force	1.00	–	–	–
	γ_q	Surcharge on slice segment	1.00			
	$\gamma_{w'}$	Weight of slice segment	1.00			
	$\gamma_{\tan\phi'}$	Ground strength: Tangent of angle of shear resistance	1.00			
	$\gamma_{c'}$	Ground strength: Cohesion	1.00			
	γ_a	Structural analysis factor	1.00			

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: RC: reinforced concrete, NC: non-reinforced concrete.
*3: Change of water depth mild/steep: Gradient of sea bottom <1/30/≧1/30.
*4: r_{wl} denotes the ratio of the highest high water level (H.H.W.L.) and mean monthly-high water level (H.W.L.).

(15) Performance Verification for Accidental Waves

Performance verification for accidental waves can be considered equivalent to the verification of the variable situation in respect of waves upon appropriate evaluation of the actions due to accidental waves. Provided, however, that, the partial factors used in the performance verification in respect of tsunamis shown in **Table 3.1.3** may be as applied to the partial factors when the performing verification is conducted based on the static balance of forces.

(16) Performance Verification for Stability of Sloping Sections

① With breakwaters, the examination is conducted for slip failure of the rubble section. However, this may be examined as slip failure due to eccentric and inclined loads.

② For slip failure due to eccentric and inclined loads, **Chapter 2, 2.2.5 Bearing Capacity for Eccentric and Inclined Actions** can be used as reference.

③ In armor units for the rubble section, in addition to an adequate stable mass against wave force, the thickness should be sufficient to prevent flowing out of the materials in the mound interior.

④ For the required mass of armor units, **Chapter 2, 1.7.2 Required Mass of Armor Stones and Blocks in Composite Breakwater Foundation Mound against Waves** can be used as reference.

⑤ As the required mass of the rubble and blocks under the armor units, it is preferable that the mass of these materials be approximately 1/20 or more that of the armor units. It is preferable that the mass of the stones under these underlying materials be approximately 1/20 or more than that of the underlying materials.

(17)Performance Verification for Stability of Breakwater Head and Concave Corners

① In comparison with the breakwater trunk, there are various unclear points regarding scouring of the foundation

and actions affecting the heads of breakwaters. Therefore, it is preferable that the mass of the armor stones and armor blocks be set larger for the breakwater head than for the trunk. In calculations of the mass of armor units, **Chapter 2, 1.7.2 Required Mass of Armor Stones and Blocks in Composite Breakwater Foundation Mound against Waves** can be used as reference.

② In the case of soft ground, slip failure in the direction of the breakwater extension should also be examined. In this case, the frictional resistance of the sides of the slip surface may also be considered.

③ In the performance verification of concave corners, increase of the wave height should be considered.

④ In breakwater alignment which includes concave corners, in addition to the concentration of waves at the concave corner itself, an increase in wave height based on superposition of the reflected waves from the various parts in the breakwater alignment will also occur around the corners. Because there have been examples of damage which is considered to be attributable to this phenomenon, in determining the breakwater alignment and calculating stability, examination can be performed using **Part II, Chapter 2, 4.3 Wave Transformations** and **4.7.2(8) Calculation of Wave Force considering Effect of Alignment of Breakwater**.

3.1.5 Performance Verification of Structural Members

In the performance verification of structural members for caissons, cellular blocks, and hybrid caissons, **Chapter 2, 1 Structural Members** can be used as reference.

3.1.6 Structural Details

Items for respective types of upright sections are described in (1)to(4). Common items are described in (5) and after.

(1) Caisson Type Composite Breakwaters

① Various materials are used as filling in caissons, including concrete, concrete blocks, stones, gravel, sand and slag. When selecting a filling material, it is preferable to consider construction costs, construction conditions and natural conditions.

In general, sand is frequently used. However, when sand or gravel is used as a filling material, it is necessary to cover the surface completely with a concrete lid or blocks. Slag may absorb water and expand, depending on the type of material. Accordingly, when using slag, attention should be paid to the material properties of the slag as a filling material, including the method of treating the slag before filling the caissons.

② The thickness of the concrete lid should normally be 30 cm or greater, and should be 50 cm or greater under rough sea conditions. There are also examples of the thickness of 1.0 m or greater in the cases where wave conditions are severe and the concrete lids are left without placement of crown concrete for a long time.

③ Because there are many unclear points regarding the wave forces acting on crown concrete, the concrete lid placement should be performed in such a way that the crown concrete is integrated with the breakwater body. Methods of further increasing integration of the concrete lid with the crown concrete include pouring of the crown concrete in such a way that it is squeezed inside the caisson, fabrication of concave/convex shapes in the concrete lid (frequently used with the precast concrete), use of reinforcing bars or shape steel, see **Fig. 3.1.10**. In order to unify the parapet and crown concrete, it is preferable to adopt a method such a providing tenons at construction joints, use of reinforcing bars or shape steel, etc.

④ Because scouring of the foundation occurs easily at the bottom of the upright section, when the structure is not built on bedrock, adequate foot protection work should be performed.

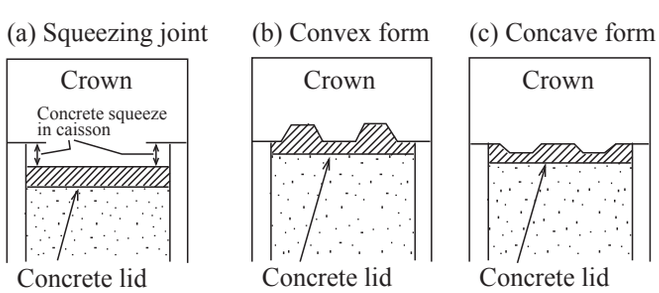


Fig. 3.1.10 Placement Surface of Crown Concrete

(2) Mass Concrete Block type Composite Breakwaters

① Methods of stacking blocks include horizontal stacking and inclined stacking. In general, however, horizontal stacking is used considering ease of construction work. In the crown concrete, it is preferable to provide joints at intervals of 10–20m in the direction of breakwater alignment. In case of horizontal stacking, in order to maintain integration, it is preferable that vertical joints in the cross section perpendicular to the breakwater alignment be arranged in a cross-stitched form so as not to penetrate from the top to the bottom.

② With concrete blocks, in order to avoid sliding, a method of mutual interlocking using concave/convex shaped tenon joints of the shape shown in **Fig. 3.1.11** is generally used. In many cases, the width *a* and height *b* of the convex part are on the order of 50cm and 20cm, respectively, and the width *a*' and height *b*' of the convex parts are approximately 5cm larger than the corresponding parts *a* and *b*.

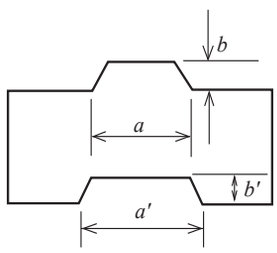


Fig. 3.1.11 Joint in Concrete Block

(3) Cellular Block Type Composite Breakwaters

① It is preferable that a footing be attached to the bottommost layer of cellar blocks secure stability.

② Concrete or stones can be used as filling for cellular blocks. If concrete is used as the filling, poor integration of the upright section cellular block type is eliminated.

③ Because integration is reduced if cellular blocks are laid in two layers, it is preferable to use a one block type whenever possible. If blocks are obliged to be laid in layers, the integration should be increased by interlocking of the upper and lower layers by fabricating convex/concave shape at the top and bottom of the wall of the cellular blocks, as shown in **Fig. 3.1.2(c).**

④ If stones are to be used as filling, a bottom plate shall be provided on the cellular blocks in order to prevent dislodgement of the stones From the cellular section.

(4) Mass Concrete Single Block Type Composite Breakwaters

① In the upright section of mass concrete type composite breakwaters, the size of one block should be 5–10m in order to prevent cracking due to shrinkage or uneven settlement.

② A certain degree of irregularity on the foundation surface does not pose a serious problem. However, together with carefully removing sand, debris, and seaweed on the bedrock in order to assure good adhesion with the concrete, parts in contact with the shuttering form should also be leveled to improve contact with the shuttering form.

(5) The rubble mound foundation of composite breakwater is extremely important to ensure the stability of the upright section. Particularly if the rubble mound beneath the upright section is scoured or washed out, the upright section will lean or easily experience sliding failure, and then the upright structure will be destroyed at worst. It is therefore necessary to protect the rubble mound beneath the upright section with foot protection blocks and prevent damage from scouring or washing-out due to the action of waves or currents.

(6) Uplift forces acting on blocks can be reduced and stability against waves can be greatly improved by providing holes in foot protection blocks.

(7) Because the study by Tanimoto et al.[8)] shows that large holes in foot protection blocks reduce the effect of preventing scouring and washing out, the opening ratio of about 10% is optimal.

(8) It is preferable that two or more rows of foot protection blocks be placed on the seaward side of the upright section and one or more rows on the harbor side.

(9) The required thickness of the foot protection blocks can be determined by using equation (**3.1.26**).[28)]

$$t / H_{1/3} = d_f \left(h' / h\right)^{-0.787} \tag{3.1.26}$$

where

t : required thickness of the foot protection blocks (m)
d_f : 0.18 for the breakwater trunk, 0.21 for the breakwater head (m)
h : design water depth (m)
h' : water depth at the top of rubble mound foundation excluding the foot protection blocks (m)
the application range should be h'/h=0.4–1.0.

(10) For the determination of the dimensions of the foot protection block, the required thickness can be calculated using equation **(3.1.26)** and the dimensions listed can be determined using **Table 3.1.4.** Examples of the block shapes and dimensions are shown in **Fig. 3.1.12**.

Table T 3.1.4 Required Thickness and Dimensions of Foot Protection Blocks

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

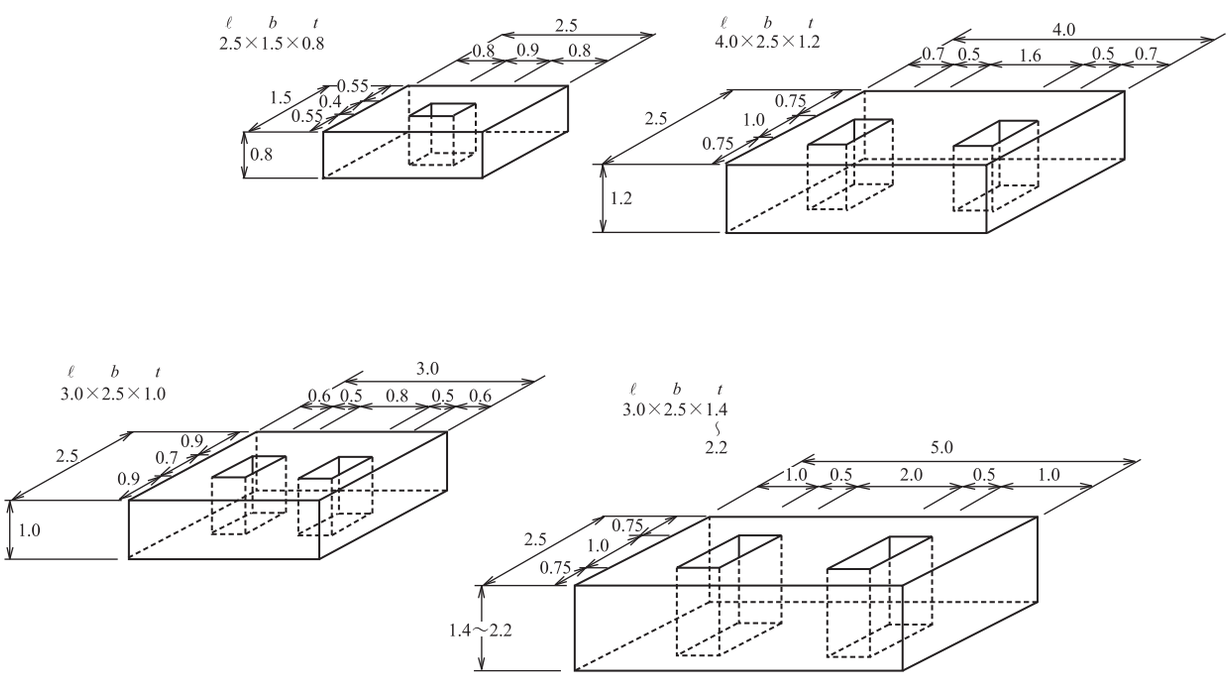


Fig. 3.1.12 Shapes and Dimensions of Foot Protection Blocks

(11) The number of cases of failure of the foot protection blocks inside a harbor has been quite small, and it is acceptable to use a mass that is lighter than the mass of the foot protection blocks of seaward side. In the past designs there were many cases where the mass was one-half of that at the seaward side. However, the mass must not be smaller than the mass required by the waves inside the harbor or the waves during construction. Especially, the mass should be carefully determined where the offshore end of a breakwater under construction remains as a temporary head during the offwork season of each year.

(12) In situations where there are concerns about scouring or flowing-out of rubble stones, preventive countermeasures should be performed. Methods used for scour prevention at the toe of slope are the provision of a berm of rubbles

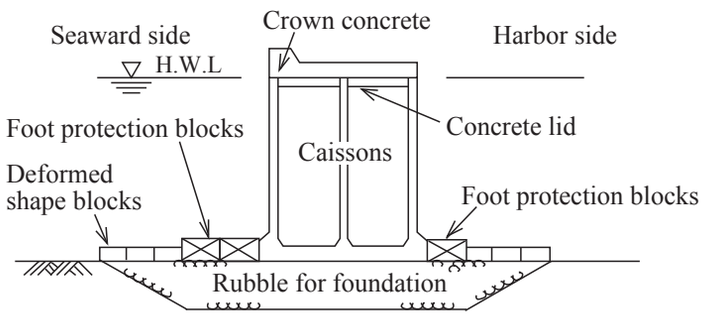
at the toe of slope, and the placement of concrete blocks, mattress work, asphalt mats [29), 30)], or composite resin mats. For the prevention of the settlement of the rubble mound due to washing-out, mattress works and other methods including the spreading of canvas sheets are employed.[31)]

3.2 Gravity-type Breakwaters (Upright Breakwaters)

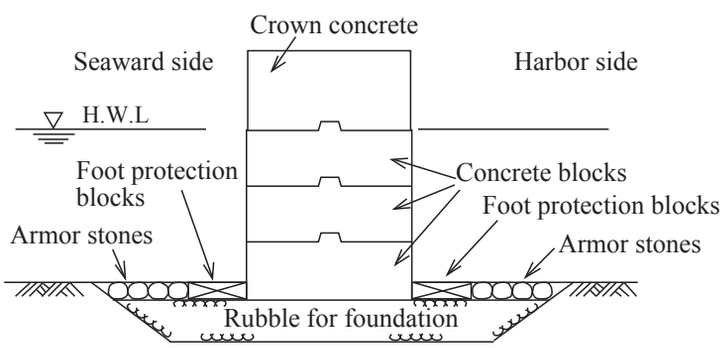
The performance verification of the upright breakwaters can be conducted by applying that of the composite breakwaters. In addition to **3.1.4 Performance Verification**, the following can be used as reference.

3.2.1 Fundamentals of Performance Verification

Examples of the cross sections of the upright breakwaters are shown in **Fig. 3.2.1**.



1) Caisson type upright breakwater



2) Concrete block type upright breakwater

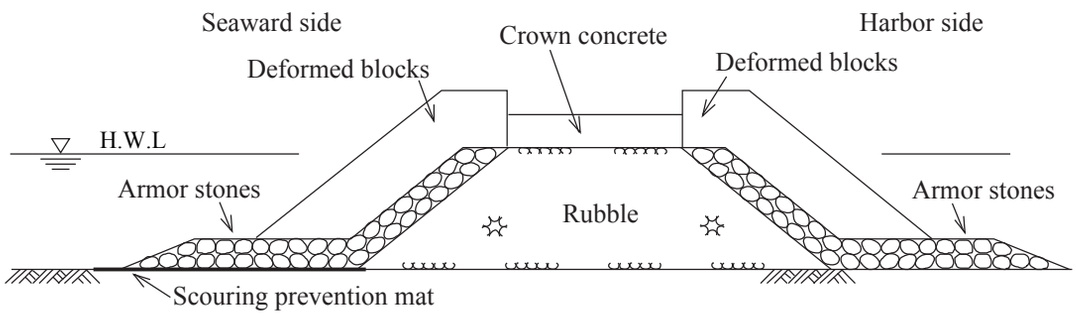
Fig. 3.2.1 Examples of Cross Sections of Upright Breakwaters

3.3 Gravity-type Breakwaters (Sloping Breakwaters)

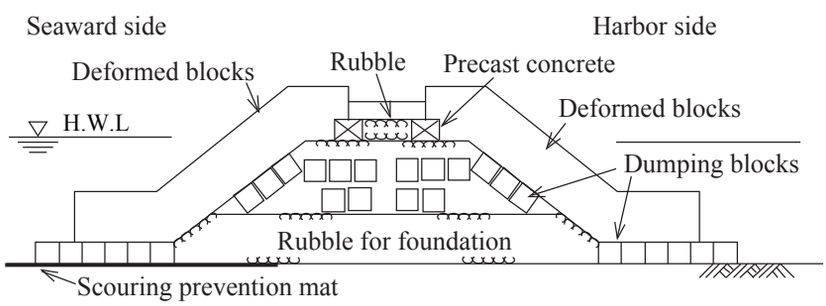
The performance verification of the sloping breakwaters can be conducted by applying that of the composite breakwaters. In addition to **3.1.4 Performance Verification**, the following can be used as reference.

3.3.1 Fundamentals of Performance Verification

Examples of the cross sections of the sloping breakwaters are shown in **Fig. 3.3.1**.



(a) Rubble mound breakwater



(b) Dumping block mound sloping breakwate

Fig. 3.3.1 Examples of Cross Sections of Sloping Breakwaters

3.3.2 Setting of Basic Cross Section

(1) The crown height can be determined by applying that of the composite breakwaters and can be set in accordance with **3.1.3 Setting of Basic Cross Section**.

(2) Because the sloping breakwaters transmit waves, caution is necessary in setting the crown height, as there are cases where the transmitted wave height in the harbor is greater that with upright breakwaters having the same crown height. For wave overtopping and transmitted waves, **Part II, Chapter 2, 4.3.7 Wave Runup Hight, Wave Overtopping and Transmitted Waves** can be used as reference.

(3) The crest width can be set based on the results of appropriate model experiments.

(4) When waves overtop heavily, a sufficiently broad crown width is required because the armor units on the top of the breakwater will become unstable.

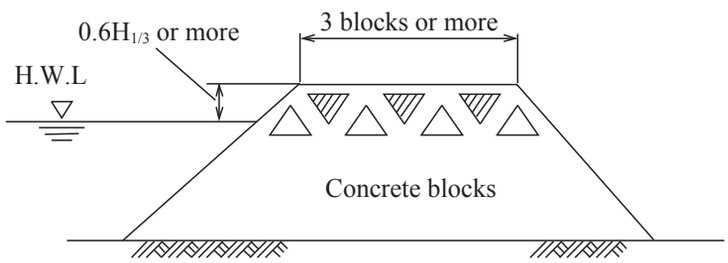
(5) For breakwaters constructed from land as a rubble mound sloping breakwater extending from the shore, in addition to an adequate width necessary for the performance verification, the width should also be determined considering ease of construction.

(6) The slope gradient should be appropriately determined based upon the stability calculation.

(7) For breakwaters on soft ground, the crown height and construction method can be determined by applying those of the composite breakwaters, and can be set based on **3.1.3 Setting of Basic Cross Section**.

(8) If the crest of breakwater covered with deformed concrete blocks is set at an elevation of $0.6H_{1/3}$ above the mean monthly-highest water level, the crown width may be equivalent to that of three or more blocks as shown in **Fig. 3.3.2**. Because the stability of the breakwater top section will depend upon the characteristics of the armor units and wave conditions, however, it is desirable to determine the width based upon appropriate hydraulic model tests.

(9) There are many cases where the slope gradient for rubble mound type sloping breakwaters is about 1: 2 on the seaward side of the breakwater and about 1: 1.5 on the harbor side, and about 1: 1.3 to 1: 1.5 in the case of breakwaters covered with deformed concrete blocks. When the gradient of the slope and the mass of the armor units are different between the upper and lower portions of the slope on the seaward side of the breakwater, the point at which the gradient and the mass of armor units change should be deeper than $1.5H_{1/3}$ below the design water level.



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

3.3.3 Performance Verification

(1) In the verification of the stability of sloping breakwaters having a superstructure, **3.1 Gravity-type Breakwaters (Composite Breakwaters)** can be used as a reference.

(2) Performance verification of stability of superstructure
Examination of the stability of the superstructure in the variable situation in respect of waves is generally performed for sliding and overturning of the superstructure.

(3) Performance Verification of Stability of Sloping Section

① In sloping breakwaters, slip failure of the rubble mound section is examined. The examination of slip of the rubble mound section can be performed for slip due to eccentric and inclined loads.

② For slip failure due to eccentric and inclined loads, **Chapter 2, 2.2.5 Bearing Capacity for Eccentric and Inclined Actions** can be used as a reference.

③ In the armor materials of the rubble mound section, in addition to an adequate stable mass against wave forces, the thickness should also be adequate to prevent sucking-out of the material in the mound interior.

④ In calculation of the necessary mass of armor units, **Chapter 2, 1.7 Armor Stones and Block** can be used as a reference.

⑤ In case regular placing and stone panels are used rather than pellmell placing of the armor material, the necessary mass may be determined depending on the judgment of the responsible engineer. The thickness of the armor layer in case of pellmell placing shall generally be 2 layers.

⑥ As the required mass of the rubble and blocks under the armor materials, it is preferable that the mass of these materials be approximately 1/10 to 1/15 that of the armor units or more. It is preferable that the mass of the stones under these underlying units be approximately 1/20 that of the underlying units or more.

(4) Partial Factors
As partial factors for sliding and overturning of the superstructure of sloping breakwaters, the partial factors shown in **Table 3.3.1** may be used. The partial factors shown in **Table 3.3.1** were set considering the settings in the conventional design method.

As partial factors for use in verification of the bearing capacity of the foundation ground and circular slip failure of the ground, the partial factors shown in **Chapter 2, 2.2.5 Bearing Capacity for Eccentric and Inclined Actions** and **Chapter 2, 3.2.1 Stability Analysis using Circular Slip Failure Surface**, respectively, may be used with the appropriate modifications.

Table 3.3.1 Partial Factors for use in Verification of Stability of Superstructure

			γ	α	μ/X_k	V
Sliding	γ_f	Friction coefficient	1.00	–	–	–
	γ_{P_H}, γ_{P_U}	Change of water depth: Mild	1.00	–	–	–
		Change of water depth: Steep	1.00		–	–
	γ_{wl}	r_{wl}=1.5	1.00	–	–	–
		r_{wl}=2.0, 2.5	1.00		–	–
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	1.00	–	–	–
	$\gamma_{W_{NC}}$	Unit weight of NC	1.00	–	–	–
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	–	–	–
	γ_a	Structural analysis factor	1.20	–	–	–
Overturning	γ_{P_H}, γ_{P_U}	Change of water depth: Mild	1.00	–	–	–
		Change of water depth: Steep	1.00		–	–
	γ_{wl}	r_{wl}=1.5	1.00	–	–	–
		r_{wl}=2.0, 2.5	1.00		–	–
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	1.00	–	–	–
	$\gamma_{W_{NC}}$	Unit weight of NC	1.00	–	–	–
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	–	–	–
	γ_a	Structural analysis factor	1.20	–	–	–

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: RC: reinforced concrete, NC: non-reinforced concrete.
*3: Change of water depth Mild/Steep: Gradient of sea bottom <1/30/≧1/30.
*4: r_{wl} denotes the ratio of the highest high water level (H.H.W.L.) and mean monthly-high water level (H.W.L.).

(5) Performance Verification for Stability of Breakwater Head

It is preferable that the head section of sloping breakwaters be constructed in a semi-circular shape using armor units with a mass 1.5 times that of the trunk section or more. In calculation of the mass of the sloping breakwater and wave-dissipating blocks, **Chapter 2, 1.7 Armor Stones and Blocks** can be used as a reference.

3.4 Gravity-type Breakwaters (Breakwaters Covered with Wave-dissipating Blocks)

The performance verification for breakwaters covered with wave-dissipating blocks is equivalent to that for composite breakwaters. In addition to **3.1.4 Performance Verification**, the following can be used as a reference.

3.4.1 Fundamentals of Performance Verification

Examples of the cross sections of breakwaters covered with wave-dissipating blocks are shown in **Fig. 3.4.1**.

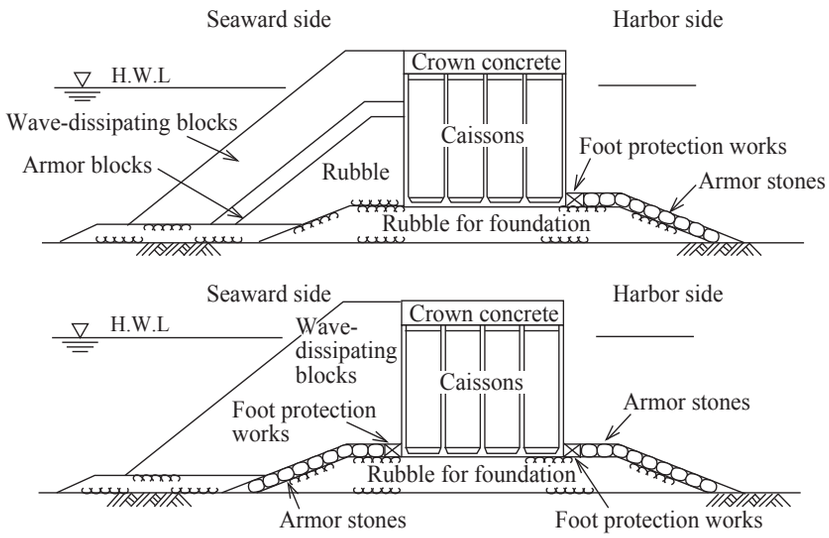


Fig. 3.4.1 Examples of Cross Sections of Breakwaters Covered with Wave-dissipating Blocks

3.4.2 Setting of Basic Cross Section

(1) The crown height of the upright section is equivalent to that of composite breakwaters and shall be set to a height which satisfies performance requirements, referring to **3.1.4 Performance Verification**.

(2) When the crown height of wave-dissipating works is lower than that of the upright section, the impulsive breaking wave force is likely to act on the upright section. Contrary to this, where the former crown height is higher than the latter, blocks at the crown will become unstable.

(3) In order to achieve a sufficient wave-dissipating performance, the crown width of the wave-dissipating works must have the width equivalent to two or more units of wave absorbing blocks [32), 33)].

(4) The thickness of the superstructure and installed crown height of caissons can be considered equivalent to those of the upright breakwaters. The thickness of the rubble mound section can be considered equivalent to that of the composite breakwaters.

(5) With the breakwaters covered with wave-dissipating blocks, overtopping waves and transmitted waves will be smaller in comparison with the upright breakwaters and the composite breakwaters with the same crown heights. For overtopping waves and transmitted waves, **Part II, Chapter 2, 4 Waves** can be used as a reference.

(6) Wave-dissipating works have the functions of decreasing the wave pressure, overtopping waves, transmitting waves and reflecting waves. Accurate evaluation of these functions should preferably be made based upon hydraulic model tests.

(7) If the vertical faces of the upright section are not fully covered with wave-dissipating blocks at the tip of breakwater extension, large wave forces are likely to act on these vertical faces. Caution is necessary.

3.4.3 Performance Verification

(1) Performance Verification and Partial Factors for Sliding, Overturning Foundation, Failure, and Circular Slip Failure

① Partial factors

For the partial factors for the standard system failure probabilities for sliding and overturning of the upright

section of breakwaters covered with wave-dissipating blocks and foundation failure of in the variable situation in respect of the action of waves, and for the standard failure probability for circular slip failure in the permanent situation, the values in **Table 3.4.1** can be used as reference 3), 34). The standard system failure probabilities of sliding and overturning of the upright section of breakwaters covered with wave-dissipating blocks and foundation failure is based on an evaluation by reliability theory of the average safety levels of breakwaters designed by the conventional design method.[3] For circular slip failure, the target reliability index is set at 3.6, converted failure probability of 2.0 x 10^{-4}, which minimizes the expected sum cost expressed by the total of the initial construction cost and the expected value of the recovery costs associated with failure recovery. If the safety level based on minimization of the expected total cost is evaluated by reliability theory, the partial factors are as shown in **Table 3.4.1 (b)**.[34] If based on the average safety level of the conventional design method, the average reliability index is 6.9, converted failure probability of 3.1 x 10^{-12}.[34]

Table 3.4.1 Standard Partial Factors

(a) Variable situations in respect of waves

Target system reliability index β_T			2.38			
Target system failure probability P_{f_T}			8.7×10^{-3}			
Target reliability index β_T ' used in calculation of γ			2.40			
			γ	α	μ/X_k	V
Sliding	γ_f	Friction coefficient	0.77	0.750	1.060	0.150
	γ_{P_H}, γ_{P_U}	Change of water depth: Mild	0.91	-0.636	0.702	0.191
		Change of water depth: Steep	1.01		0.772	0.205
	γ_{wl}	r_{wl}=1.5	1.04	-0.081	1.000	0.200
		r_{wl}=2.0, 2.5	1.08		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	0.98	0.030	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.02	0.031	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.01	0.150	1.020	0.040
Overturning	γ_{P_H}, γ_{P_U}	Change of water depth: Mild	1.01	-0.962	0.702	0.191
		Change of water depth: Steep	1.14		0.772	0.205
	γ_{wl}	r_{wl}=1.5	1.06	-0.133	1.000	0.200
		r_{wl}=2.0, 2.5	1.13		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	0.98	0.050	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.02	0.054	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	0.248	1.020	0.040
Bearing capacity of foundation ground	γ_{P_H}	Change of water depth: Mild	0.97	-0.842	0.702	0.191
		Change of water depth: Steep	1.09		0.772	0.205
	γ_q	Surcharge on slice segment	0.93	0.525	0.367	0.058
	$\gamma_{w'}$	Weight of slice segment	1.00	0.047	1.000	0.030
	$\gamma_{\tan\phi'}$	Ground strength: Tangent of angle of shear resistance	0.95	0.353	1.000	0.061
	$\gamma_{c'}$	Ground strength: Cohesion	0.99	0.112	1.000	0.061
	γ_a	Structural analysis factor	1.00	–	–	–

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: RC: reinforced concrete, NC: non-reinforced concrete.
*3: Change of water depth Mild/Steep: Gradient of sea bottom <1/30/≧1/30.
*4: r_{wl} denotes the ratio of the highest high water level (H.H.W.L.) and mean monthly-high water level (H.W.L.).
*5: γ_q is a term which is multiplied by the average value of the surcharge. The average value of the surcharge is obtained using. $\bar{q} = \sum \bar{V} / \overline{2b'}$

Table 3.4.1 Standard Partial Factors

(b) Permanent situation

Target reliability index β_T				3.6			
Target failure probability P_{fT}				2.0×10^{-4}			
				γ	α	μ/X_k	V
Circular slip failure	$\gamma_{c'}$		Ground strength: Cohesion	0.90	0.327	1.00	0.035
	$\gamma_{\tan\phi'}$		Ground strength: Tangent of angle of shear resistance	0.90	0.364	1.00	0.035
	γ_{w_i}	When mound is positioned below level of sea bottom	1 Wave-dissipating work, etc. above level of sea bottom	1.00	-0.034	1.00	0.03
			2 Sandy soil below mound and level of sea bottom	0.90	-0.027		
			3 Cohesive soil below level of sea bottom	0.90	0.285		
		When mound is positioned above level of sea bottom	1 Mound, wave-dissipating work, etc. above level of sea bottom	1.00	-0.034	1.00	0.03
			2 Sandy soil below level of sea bottom	0.90	-0.027		
			3Clayey soil below level of sea bottom	0.90	0.285		
	γ_q		Spatially distributed load	1.10	-0.410	1.02	0.04

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: γ_1, γ_2, and γ_3 are partial factors for the weight of the slice segments.
*3: The calculated γ_{w2} is 1.0. However, considering convenience in performance verification, the same value as for composite breakwaters, 0.9, was adopted.[34)]
*4: Wave-dissipating work, etc. includes wave-dissipating work, armoring work, foot protection work.
*5: In application of the partial factors for circular slip failure, reference shall be made to the notes shown in **Chapter 2, 3 Stability of Slopes**, **3.1 (7) Partial Factors**. When soil is improved by the sand compaction pile (SCP) method with a replacement ratio of 30–80%, the partial factors shown in **4.10.6 Performance Verification** for the sand compaction pile method in **Chapter 4 Soil Improvement Methods** shall be used.

(2) Performance Verification for Stability of Armoring Section

① In calculating the necessary mass of armor unit for breakwaters covered with wave-dissipating blocks, **Chapter 2, 1.7 Armor Stones and Blocks** can be used as a reference.

② In case regular placing and stone panels are used rather than pellmell placing of the armor material, the necessary mass may be determined at the judgment of the responsible engineer. The thickness of the armor layer in case of pellmell placing shall generally be 2 layers or more.

(3) Performance Verification For Stability of Breakwater Tip
It is preferable that the tip of breakwaters covered with wave-dissipating blocks be constructed in a semi-circular shape using armor units with a mass of 1.5 times that of the trunk or more. For calculation of the mass of wave-dissipating blocks, **Chapter 2, 1.7 Armor Stones and Blocks** can be used as a reference.

(4) Performance Verification for Stability of Wave-dissipating Work
In the performance verifications, **Chapter 2, 1.7 Armor Stones and Blocks** can be used as a reference.

(5) All partial factors indicated here are values for the case where the design working life is the normal 50 years. Because techniques for the period during construction where a structure is left for a certain period with an unfinished cross section have not been particularly examined at the present point in time, for convenience, the verification may be carried out using the same partial factors as for composite breakwaters when completed, using actions of waves with a probability on the order of 10 years.[37)]

(6) The performance verification in the accidental situation in respect of Level 2 earthquake ground motion is equivalent to that for gravity-type quaywalls. The method shown in **(9) Performance Verification for Ground Motion (detailed methods)** of **Chapter 5, 2.2.3 Performance Verification** can be used as a reference.

3.5 Gravity-type Breakwaters (Upright Wave-absorbing Block Type Breakwaters)

Upright wave-absorbing block type breakwaters are mass concrete block type upright breakwaters or composite breakwaters which are constructed by vertical stacking of special blocks, called the upright wave-absorbing block, having a wave-dissipating function. The performance verification for the upright wave-absorbing block type breakwaters is equivalent to that for the composite breakwaters. In addition to **3.1.4 Performance Verification**, the following can be used as a reference.

3.5.1 Principals of Performance Verification

(1) Because various types of structures have been developed for the upright wave-absorbing blocks, it is preferable to select appropriate blocks based on an adequate investigation of their wave-absorbing performance.

(2) The wave reflection coefficient of the upright wave-absorbing blocks greatly depends upon the wave period. When determining the reflection coefficient, it is best to carefully consider the influence of wave period based on hydraulic model tests corresponding to the design conditions. It is also acceptable to estimate it by referring to the data from past experiments.

(3) With the exception of large-scale blocks to be used as a single block structure, the upright wave-absorbing block type breakwaters are generally used in inner bays or the inside of harbors where wave heights are relatively small.

(4) An example of the cross section of the upright wave-absorbing block type breakwater is shown in **Fig. 3.5.1**.

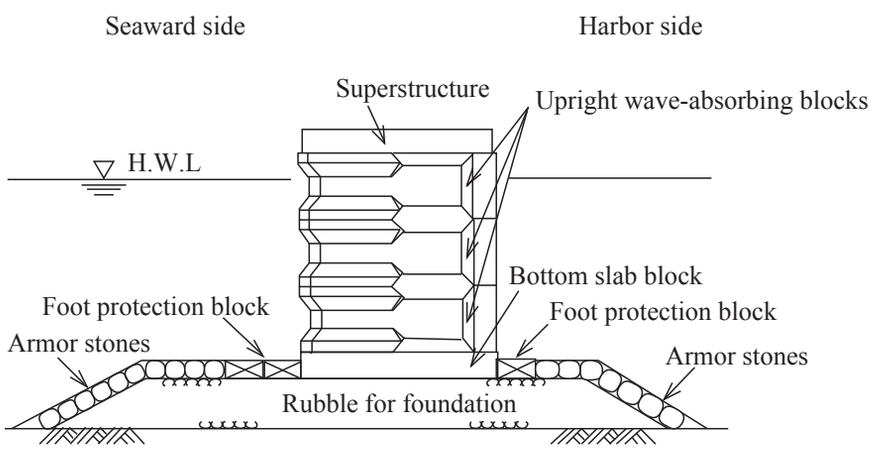


Fig. 3.5.1 Example of Cross Section of Upright Wave-absorbing Block Type Breakwater

3.5.2 Setting of Basic Cross Section

(1) The crown height of the upright wave-absorbing block type breakwaters is equivalent to that of the composite breakwaters and can be decided considering the height which satisfies the performance requirements and height of the wave-absorbing section, referring to **3.1.4 Performance Verification**. The crown height of the wave-absorbing section shall be determined considering the wave-absorbing performance. In structures with permeability, it is preferable that the dimensions of the opening section be determined considering the transmission characteristics.

(2) The wave-absorbing performance of the upright wave-absorbing block type breakwaters will vary, depending on the crown height and bottom elevation of the wave-absorbing block section.

(3) In the upright wave-absorbing block type breakwaters, wave overtopping and transmitted waves are small in comparison with those with the composite breakwaters, but tend to be larger than those with breakwaters covered with wave-absorbing blocks. Accordingly, it is preferable that the crown height be determined giving adequate consideration to the conditions of use behind the breakwater. In addition, in determining the crown height, the thickness required for construction of the crown concrete should be secured.

(4) It is preferable that the crown height h_c' be at least 0.5 times higher or more than the significant wave height used in the stability examination of the facilities above mean monthly-high water level. It is preferable that the bottom height h_u be set to a depth 2 times or greater than the significant wave height used in the stability examination of the facilities below the mean monthly-high water level (see **Fig. 3.5.2**).

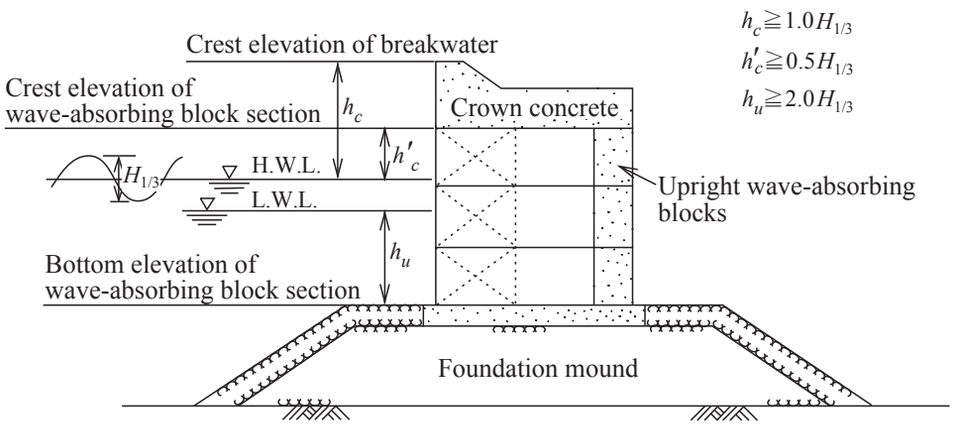


Fig. 3.5.2 Explanatory Diagram for Crown height of Upright Wave-absorbing Block Type Breakwaters

3.5.3 Performance Verification

(1) Performance Verification and Partial Factors for Sliding, Overturning, Foundation Failure of the Ground, and Circular Slip Failure

① Verification of the stability of upright wave-absorbing block type breakwaters can be considered equivalent to that for the composite breakwaters. Provided, however, that it is necessary to use the values shown below for the standard partial factors used in the verification of sliding, overturning, and failure of the bearing capacity of the foundation ground.

② Partial factors

(a) As partial factors for standard system failure probabilities for sliding and overturning of the upright section of upright wave-absorbing block type breakwaters and foundation failure of the foundation ground, in the variable situation in respect of the action of waves, the values in **Table 3.5.1** can be used as a reference. The partial factors for the standard failure probability for circular slip failure in the permanent situation are equivalent to those for the composite breakwaters. **Table 3.1.1** of **3.1.4 (6) Performance Verification and Partial Factors for Sliding, Overturning, Bearing Failure of Foundation Ground, and Circular Slip Failure** can be used as a reference.

Table 3.5.1 Standard Partial Factors (Variable situations in respect of Waves)

Target system reliability index β_T			2.04			
Target system failure probability P_{f_T}			2.1×10^{-2}			
Target reliability index β_T ' used in calculation of γ			2.12			
			γ	α	μ/X_k	V
Sliding	γ_f	Friction coefficient	0.83	0.689	1.060	0.150
	$\gamma_{P_H}, \gamma_{P_U}$	Change of water depth: Mild	1.09	-0.708	0.812	0.230
		Change of water depth: Steep	1.22		0.893	0.242
	γ_{wl}	r_{wl}=1.5	1.05	-0.125	1.000	0.200
		r_{wl}=2.0, 2.5	1.11		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	–	–	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.02	0.113	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	–	–	1.020	0.040
Overturning	$\gamma_{P_H}, \gamma_{P_U}$	Change of water depth: Mild	1.20	-0.974	0.812	0.230
		Change of water depth: Steep	1.34		0.893	0.242
	γ_{wl}	r_{wl}=1.5	1.08	-0.182	1.000	0.200
		r_{wl}=2.0, 2.5	1.15		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	–	–	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.01	0.172	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	–	–	1.020	0.040
Bearing capacity of foundation ground	γ_{P_H}	Change of water depth: Mild	1.15	-0.856	0.812	0.230
		Change of water depth: Steep	1.28		0.893	0.242
	γ_q	Surcharge on slice segment	0.90	0.625	0.685	0.074
	$\gamma_{w'}$	Weight of slice segment	1.00	0.050	1.000	0.030
	$\gamma_{\tan\phi'}$	Ground strength: Tangent of angle of shear resistance	0.95	0.324	1.000	0.070
	$\gamma_{c'}$	Ground strength: Cohesion	0.98	0.164	1.000	0.070
	γ_a	Structural analysis factor	0.76	–	–	–

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: RC: reinforced concrete, NC: non-reinforced concrete.
*3: Change of water depth Mild/Steep: Gradient of sea bottom <1/30/≧1/30.
*4: r_{wl} denotes the ratio of the highest high water level (H.H.W.L.) and mean monthly-high water level (H.W.L.).
*5: γ_q is a term which is multiplied by the average value of the surcharge. The average value of the surcharge is obtained using $\bar{q} = \sum \bar{V} / \overline{2b'}$.

3.6 Gravity-type Breakwaters (Wave-absorbing Caisson Type Breakwaters)

Wave-absorbing caisson type breakwaters are one type of deformed caisson breakwaters which use caissons with special shapes. The front section of the caissons has a porous wall and a wave chamber, giving the breakwater a wave-absorbing performance.[45] The performance verification of wave-absorbing caisson type breakwaters is equivalent to that for the composite breakwaters. In addition to **3.1.4 Performance Verification**, the following can be used as a reference.

3.6.1 Principals of Performance Verification

(1) With wave-absorbing caisson type breakwaters, it is necessary to select an appropriate structure, giving due consideration to wave-absorbing performance, etc. Because the hydraulic characteristics of wave-absorbing caisson type breakwaters, including the wave transmission and reflection coefficients, hydraulic conductivity, are still insufficiently understood, it is preferable to perform hydraulic model tests as necessary.

(2) Wave-absorbing caisson type breakwaters have the following features in comparison with the composite breakwaters.

① It can reduce reflected waves.

② It can reduce wave overtopping and transmitted waves.

③ It can reduce wave force. In particular, when the mound is high, there are cases in which powerful impulsive breaking wave force acts on conventional caisson breakwaters; however, with wave-absorbing caisson type breakwaters, there is no remarkable wave force increase.

④ it possesses a sea water aeration function, as the breakwater structure promotes mixing of air bubbles with the water. In addition, the wave chamber has the effect of fish banks.[46), 47)]

(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. As the structural type for wave-absorbing caisson type breakwaters, an appropriate structure should be selected considering the design conditions, use conditions, economy, etc. based on a careful investigation of the wave-absorbing performance, and wave resistance of each structure.

(4) For the structures and their features of various types of wave-absorbing caisson type breakwaters the **Technical Manual of New Type Breakwaters** [48)] can be used as a reference.

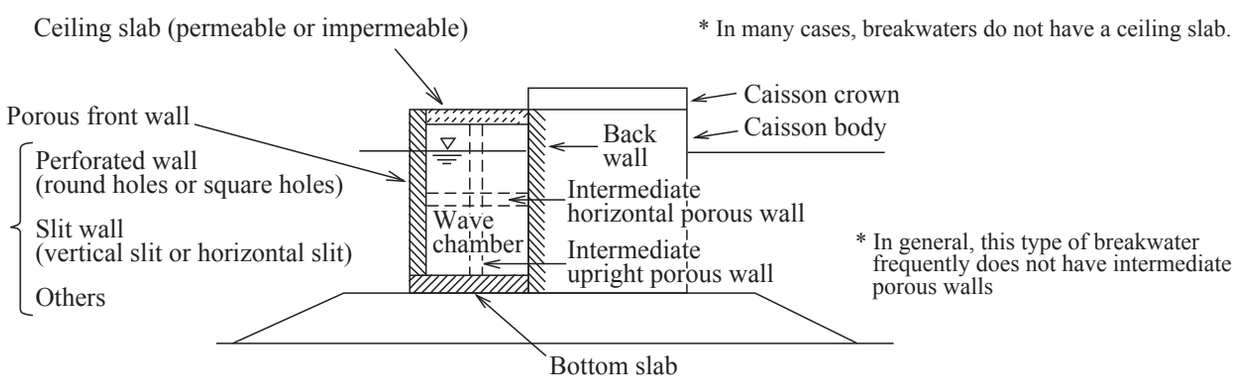


Fig. 3.6.1 Example of Cross Section of Wave-absorbing Caisson Type Breakwater

3.6.2 Actions

(1) The conditions of waves for use in the verification of wave-absorbing performance can be set separately from the conditions of waves for use in the performance verification of the stability of the facilities and the performance verification of the structural members, corresponding to the purpose of wave absorption and the wave conditions.

(2) In many cases, wave-absorbing caissons are generally adopted for the purpose of reducing reflected waves. Consequently, it is preferable to determine the conditions of the waves which are the object of wave-absorption and the target reflection coefficient corresponding to the required wave-absorbing performance. In particular, because the reflection coefficient of wave-absorbing caissons differs remarkably depending on the wave periods,

the conditions of the waves which are the object of wave-absorption should be determined based on an investigation of the characteristics of wave height and wave period.

(3) It is necessary to determine wave force using calculation formulas suitable for the wave-absorbing caisson type breakwaters or hydraulic model tests adapted to the conditions. In particular, in complex structures, in addition to the wave force used in the stability examination of the upright section as a whole, it is preferable also to conduct an adequate examination for the wave forces acting on the structural members. For wave forces acting on wave-absorbing caisson type breakwaters, **Part II, Chapter 2, 4.7.2(7) Wave Forces on Upright Wave-absorbing Caisson** can be used as a reference.

(4) As the wave force used in the performance verification of the structural members, the most severe wave force conditions for each member should be used. For wave forces acting on the structural members of the wave-absorbing caisson type breakwaters, **Part II, Chapter 2, 4.7.2(7) Wave Forces on Upright Wave-absorbing Caisson**, and **1.5.2 Action** of **Chapter 2, 1.5 Upright Wave-absorbing Caissons**, can be used as a reference.

3.6.3 Setting of Basic Cross Section

(1) In wave-absorbing caisson type breakwaters, the required dimensions should be determined appropriately, considering the shape of the structure. In particular, because the transmission coefficient will differ depending on the structure, it is preferable that the crown height be determined appropriately corresponding to the transmission characteristics of the objective structures. In cases where the structure has permeability, it is preferable that the dimensions of the opening section be determined appropriately.

(2) 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 also considering these characteristics.

3.6.4 Performance Verification

Performance verification and partial factors for sliding, overturning foundation, failure, and circular slip failure

① The verification of the stability of wave-absorbing caisson type breakwaters can be considered equivalent to that for the composite breakwaters. Provided, however, that it is necessary to use the values shown below for the standard partial factors used in the verification of sliding, overturning, and foundation failure.

② Partial factors
As the standard system failure probabilities for sliding and overturning of the upright section of wave-absorbing caisson type breakwaters and failure of the bearing capacity of the foundation ground, in the variable situation in respect of the action of waves, the values in **Table 3.6.1** can be used as a reference. The partial factors for the standard failure probability for circular slip failure in the permanent situation are equivalent to those for the composite breakwaters. **Table 3.1.1** of **3.1.4 (6) Performance Verification and Partial Factors for Sliding, Overturning, Foundation Failure, and Circular Slip Failure** can be used as a reference.

Table 3.6.1 Standard Partial Factors (Variable Situations in respect of Waves)

Target system reliability index β_T			2.05			
Target system failure probability P_{f_T}			2.0×10^{-2}			
Target reliability index β_T ' used in calculation of γ			2.10			
			γ	α	μ/X_k	V
Sliding	γ_f	Friction coefficient	0.84	0.661	1.060	0.150
	γ_{P_H}, γ_{P_U}	Change of water depth: Mild	1.07	-0.732	0.799	0.223
		Change of water depth: Steep	1.20		0.882	0.235
	γ_{wl}	r_{wl}=1.5	1.02	-0.053	1.000	0.200
		r_{wl}=2.0, 2.5	1.04		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	0.98	0.059	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.02	0.014	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.01	0.135	1.020	0.040
Overturning	γ_{P_H}, γ_{P_U}	Change of water depth: Mild	1.16	-0.971	0.799	0.223
		Change of water depth: Steep	1.30		0.882	0.235
	γ_{wl}	r_{wl}=1.5	1.03	-0.063	1.000	0.200
		r_{wl}=2.0, 2.5	1.05		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	0.97	0.124	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.02	0.015	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	0.180	1.020	0.040
Bearing capacity of foundation ground	γ_{P_H}	Change of water depth: Mild	1.12	-0.852	0.799	0.223
		Change of water depth: Steep	1.25		0.882	0.235
	γ_q	Surcharge on slice segment	1.01	-0.126	1.000	0.041
	$\gamma_{w'}$	Weight of slice segment	1.00	0.037	1.000	0.030
	$\gamma_{\tan\phi'}$	Ground strength: Tangent of angle of shear resistance	0.96	0.350	1.000	0.060
	$\gamma_{c'}$	Ground strength: Cohesion	0.99	0.075	1.000	0.060
	γ_a	Structural analysis factor	0.92	–	–	–

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: RC: reinforced concrete, NC: non-reinforced concrete.
*3: Change of water depth Mild/Steep: Gradient of sea bottom <1/30/≧1/30.
*4: r_{wl} denotes the ratio of the highest high water level (H.H.W.L.) and mean monthly-high water level (H.W.L.).
*5: γ_q is a term which is multiplied by the average value of the surcharge. The average value of the surcharge is obtained using $\bar{q} = \sum \bar{V} / \overline{2b'}$.

3.7 Gravity-type Breakwaters (Sloping-top Caisson Breakwaters)

Sloping-top caisson breakwaters are one type of deformed caisson breakwaters which uses caissons with special shapes. This type is a breakwater which utilizes the wave force acting on the sloping wall to stabilize the breakwater body simultaneously by reducing the horizontal wave force. The performance verification of the sloping-top caisson breakwaters is equivalent to that for the composite breakwaters. In addition to **3.1.4 Performance Verification**, the following can be used as a reference.

3.7.1 Fundamentals of Performance Verification

(1) Normally the sloping surface of the sloping-top caisson breakwater is set to begin at the still water level. However, with a semi-submerged shape in which the toe end of the sloping surface is set below the still water level, further reduction of wave forces is possible.[50)]

(2) When the upright part at the front of the caissons is armored with wave-dissipating blocks, there are cases in which this causes to the generation of impulsive breaking wave pressure, depending on the crown height of the wave-dissipating works. Furthermore, because the wave-dissipating blocks only extend as high as the still water level, particular caution is needed with regard to the stability of the blocks.

(3) An example of the cross section of a sloping-top caisson breakwater is shown in **Fig. 3.7.1**.

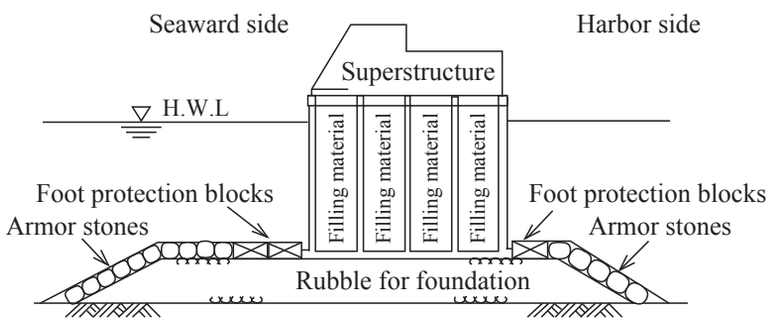


Fig. 3.7.1 Example of Cross Section of Sloping-top Caisson Breakwater

3.7.2 Actions

(1) It is preferable that the wave forces acting on the sloping-top caisson breakwaters be decided based on hydraulic model tests. However, in cases where this would be difficult, **Part II, Chapter 2, 4.7.2(6) Wave Forces on Sloping-top Caisson Breakwaters** can be used as a reference.

(2) There is a study by Sato et al.[51)] on the wave force acting on a sloping-top caisson breakwater covered with wave-dissipating blocks.

3.7.3 Setting of Basic Cross Section

(1) The coefficient of wave transmission of the sloping-top caisson breakwaters is approximately 2 times that of the upright breakwaters of the same crown height, as shown in **Fig. 3.7.2**. Therefore, if the crown height is set on the same level as the significant wave height $H_{1/3}$, it is possible to reduce the transmitted wave height to approximately the same as when the crown height of the upright breakwater is 0.6 times the significant wave height.

(2) With sloping-top caisson breakwaters, as the gradient of the sloping wall becomes steeper, the effectiveness of the structure against wave transmission in the harbor increases, but conversely, wave pressure increases, reducing its effect as a sloping-top breakwater. According to hydraulic metal tests carried out with various slope gradients, no remarkable difference in the coefficient of wave transmission can be observed with gradients of 30°, 45°, and 60°. Therefore, considering the effect in reducing wave pressure and convenience in construction works, it is preferable that the slope gradient be set at 45°.

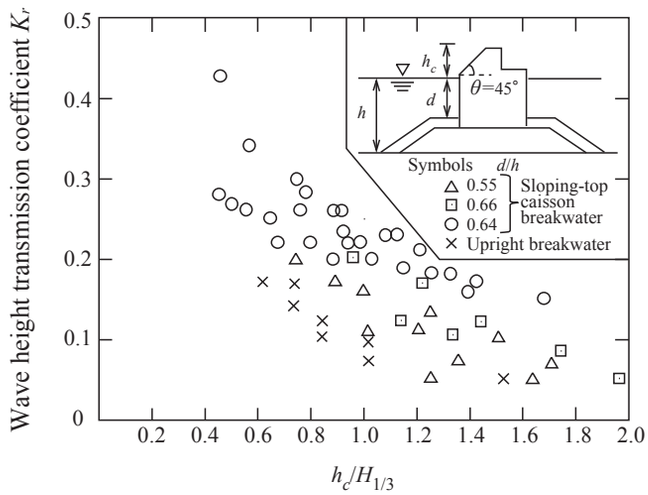


Fig. 3.7.2 Coefficient of Wave Transmission and Relative Crown height

3.7.4 Performance Verification

(1) Performance verification and partial factors for sliding, overturning, foundation failure, and circular slip failure

① The verification of the stability of the sloping-top caisson type breakwaters can be considered equivalent to that for the composite breakwaters. Provided, however, that it is necessary to use the values shown below for the standard partial factors used in the verification of sliding, overturning, and foundation failure.

② Partial factors
As the standard system failure probabilities for sliding and overturning of the upright section of wave-dissipating caisson breakwaters and failure of the bearing capacity of the foundation ground, the values in **Table 3.7.1** can be used as a reference. The partial factors for the standard failure probability for circular slip failure are equivalent to those for the composite breakwaters. **Table 3.1.1** of **3.1.4(6) Performance Verification and Partial Factors for Sliding, Overturning, Foundation Failure and Circular Slip Failure** can be used as a reference. The sloping-top caisson breakwaters covered with wave-dissipating blocks are equivalent to the breakwaters covered with wave-dissipating blocks; therefore, **Table 3.4.1** of **3.4.3(1) Performance Verification and Partial Factors for Sliding, Overturning, Foundation Failure and Circular Slip Failure** can be used as a reference.

Table 3.7.1 Standard Partial Factors (Variable Situations in respect of Waves)

Target system reliability index β_T			2.16			
Target system failure probability P_{f_T}			1.5×10^{-2}			
Target reliability index β_T ' used in calculation of γ			2.24			
			γ	α	μ/X_k	V
Sliding	γ_f	Friction coefficient	0.80	0.727	1.060	0.150
	$\gamma_{P_H}, \gamma_{P_U}$	Change of water depth: Mild	1.05	-0.670	0.777	0.232
		Change of water depth: Steep	1.19		0.868	0.243
	γ_{wl}	r_{wl}=1.5	1.03	-0.058	1.000	0.200
		r_{wl}=2.0, 2.5	1.05		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	0.98	0.027	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.02	0.031	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.01	0.128	1.020	0.040
Overturning	$\gamma_{P_H}, \gamma_{P_U}$	Change of water depth: Mild	1.17	-0.970	0.777	0.232
		Change of water depth: Steep	1.33		0.868	0.243
	γ_{wl}	r_{wl}=1.5	1.04	-0.096	1.000	0.200
		r_{wl}=2.0, 2.5	1.09		1.000	0.400
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	0.98	0.045	0.980	0.020
	$\gamma_{W_{NC}}$	Unit weight of NC	1.02	0.049	1.020	0.020
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	0.214	1.020	0.040
Bearing capacity of foundation ground	γ_{P_H}	Change of water depth: Mild	1.13	-0.872	0.777	0.232
		Change of water depth: Steep	1.28		0.868	0.243
	γ_q	Surcharge on slice segment	0.97	0.309	0.643	0.038
	$\gamma_{w'}$	Weight of slice segment	1.00	0.038	0.643	0.030
	$\gamma_{\tan\phi'}$	Ground strength: Tangent of angle of shear resistance	0.96	0.325	1.000	0.060
	$\gamma_{c'}$	Ground strength: Cohesion	0.99	0.076	1.000	0.060
	γ_a	Structural analysis factor	1.00	–	–	–

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: RC: reinforced concrete, NC: non-reinforced concrete.
*3: Change of water depth Mild/Steep: Gradient of sea bottom <1/30/≧1/30.
*4: r_{wl} denotes the ratio of the highest high water level (H.H.W.L.) and mean monthly-high water level (H.W.L.).
*5: γ_q is a term which is multiplied by the average value of the surcharge. The average value of the surcharge is obtained using $\bar{q} = \sum \bar{V} / \overline{2b'}$.

3.8 Pile-type Breakwaters

Public Notice

Performance Criteria of Pile-type Breakwaters

Article 36
The performance criteria of the pile-type breakwaters under the variable action situations, in which the dominant actions are variable waves and Level 1 earthquake ground motions, shall be as specified in the subsequent items:

(1) The risk that the axial force acting on the piles may exceed the resistance based on 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.

[Commentary]

(3) Performance Criteria of Pile-type Breakwaters

① Pile-type breakwaters
Settings of the performance criteria and the design situations excluding accidental situations of pile-type breakwaters shall be as shown in **Attached Table 19**.

The performance criteria of the superstructure and curtain wall of pile-type breakwaters shall be equivalent to the settings in **Article 23** through **Article 27**, corresponding to the type of members comprising the objective pile-type breakwater.

Attached Table 19 Settings for Performance Criteria and Design Situations (excluding accidental situations) of Pile-type Breakwaters

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating actions	Non-dominating actions		
14	1	2	36	1	1	Serviceability	Variable	Variable waves	Self weight, water pressure	Axial force acting on piles	Resistance based on failure of ground (pushing and pulling)
					2			Level 1 earthquake ground motion	Self weight, water pressure	Yielding of piles	
								Variable waves	Self weight, water pressure	Axial force acting on piles	Design yield stress
								Level 1 earthquake ground motion	Self weight, water pressure	Yielding of piles	

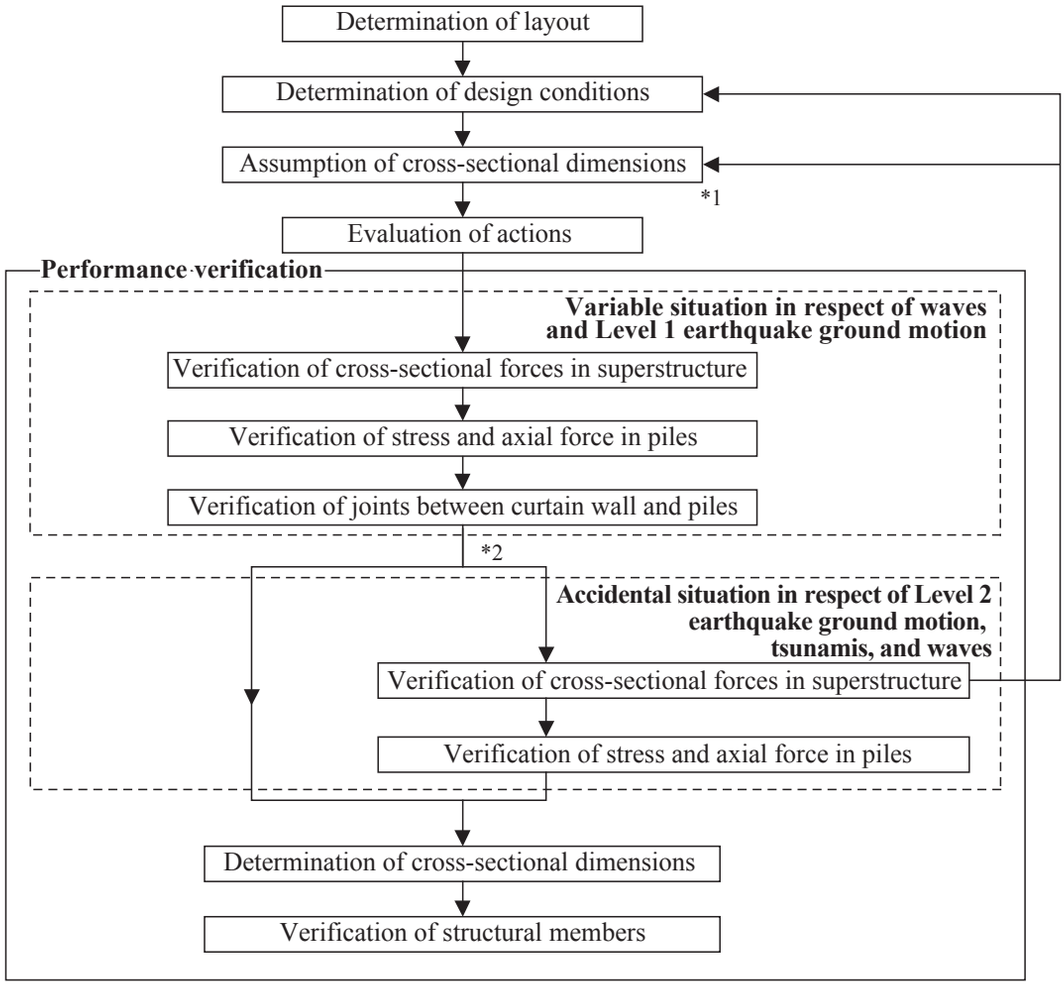
[Technical Note]

3.8.1 Fundamentals of Performance Verification

(1) The pile-type breakwaters can be broadly divided into curtain wall breakwaters and steel pipe pile breakwaters. The curtain wall breakwater is a permeable breakwater and was developed for use in waters with a comparatively low wave height, such as enclosed bays, or locations with soft sea bottom ground. Steel pipe pile breakwater is breakwater in which the curtain section is eliminated and waves are stopped only by the piles.

(2) For curtain wall breakwaters, it is preferable to select an appropriate structure considering the coefficient of wave reflection and transmission, and when necessary, to conduct the performance verification by performing hydraulic model tests.

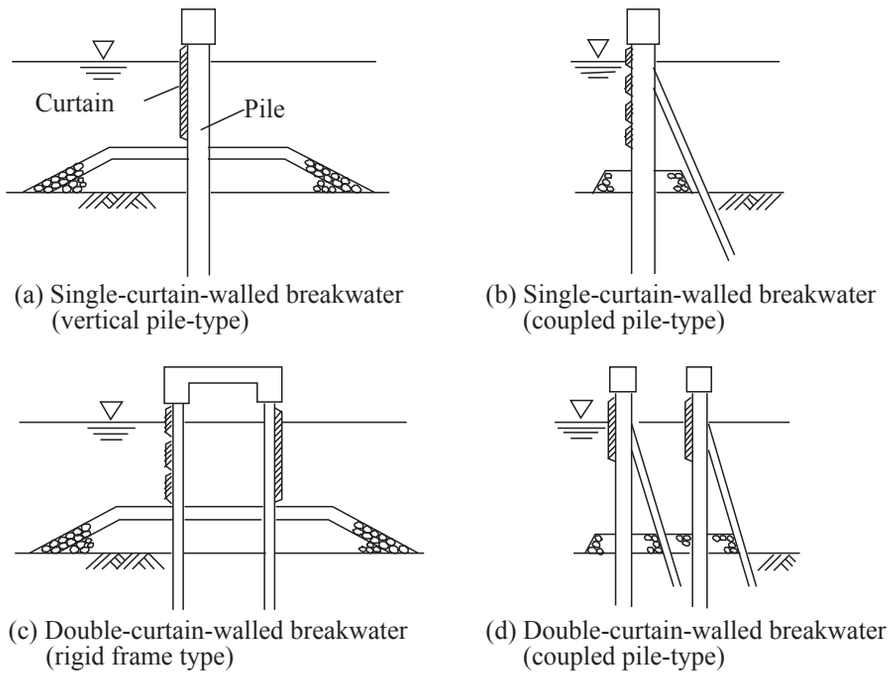
(3) An example of the performance verification procedure for curtain wall breakwaters is shown in **Fig. 3.8.1**.



*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 Performance Verification Procedure for Pile-type Breakwaters

(4) The curtain wall breakwaters can be broadly divided into the single-curtain-walled type and the double-curtain-walled type, depending how the so-called curtain wall such as concrete plates is arranged relative to the direction of wave propagation. Furthermore, a variety of types are conceivable, depending on the shape of the pile structure supporting the curtain wall or the shape of slits provided in the curtain wall. Examples of the cross sections of pile-type breakwaters are shown in **Fig. 3.8.2**.



(a) Single-curtain-walled breakwater (vertical pile-type)

(b) Single-curtain-walled breakwater (coupled pile-type)

(c) Double-curtain-walled breakwater (rigid frame type)

(d) Double-curtain-walled breakwater (coupled pile-type)

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

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

① The reflection coefficient can be reduced so as to the same level as in the breakwaters covered with wave-dissipating blocks or less.

② Exchange of sea water can be expected by tides and waves passing through slits provided in the curtain wall or the gap between the lower edge of the curtain wall and the sea bed.

③ Comparing the single-curtain-walled and the double-curtain-walled breakwaters, because an energy dissipating effect can be expected between the front and the back curtain walls with the double-curtain-walled type breakwater, reflected waves and transmitted waves can be reduced in comparison with the single-curtain-walled breakwaters.

④ Because the velocity of flows passing under the curtain wall is quite high, it is necessary to take appropriate countermeasures to prevent or suppress washing-out of sand.

3.8.2 Actions

It is necessary to set the wave force acting on the curtain wall breakwaters based on the results of hydraulic model tests, numerical analysis, or appropriate calculation formulas. When using the single-curtain-walled breakwater, the result obtained by subtracting the wave pressure distribution acting deeper than the lower edge of the curtain wall from the wave pressure distribution shown in **Part II, Chapter 2, 4.7 Wave Pressure and Wave Force** can be used as the wave force acting on the curtain wall.

3.8.3 Setting of Basic Cross Section

(1) The structural type and the shape of curtain wall breakwaters shall be determined considering the condition of sea states in the area, the target reflection coefficient, the target transmission coefficient and constructability.

(2) In setting the cross section of the curtain wall breakwaters, including the crown height, the depth of the lower end of the curtain and the size of the slits provided in the curtain, and in the case of the double-curtain-walled breakwaters, and the spacing between the curtain walls, it is preferable to set the cross section based on model tests adapted to the conditions. It is preferable that the dimensions of members such as the curtain wall, and piles be determined appropriately considering the spacing between the piles in the direction of the breakwater extension.

(3) Examples of model tests for the single-curtain-walled breakwaters include, for example, model tests by Morihira et al.[57] The depth of the lower end of the curtain wall can be obtained from **Fig. 3.8.3** if the wave transmission coefficient is determined, and the crown height of the curtain wall can be obtained from **Fig. 3.8.4**. Provided,

however, that the crown height of the curtain in **Fig. 3.8.4** was corrected so that $R/H = 1.25$ at $d/h = 1.0$, and does not show a crest capable of completely preventing wave overtopping. In the figure, d is the depth of the lower end of the curtain, h is the water depth, L is the wave length, R is the crown height of the curtain, and H is the wave height. The relationship with the wave reflection coefficient of waves by a single curtain wall is shown in **Fig. 3.8.5**.

(4) In steel pipe pile breakwaters, if the steel pipes are driven with a space between the piles, the structure can function as a permeable type breakwater. According to the research by Hayashi et al., [53] the relationship between the pile spacing/pile diameter ratio b/D and the coefficient of wave transmission γ_T is as shown in **Fig. 3.8.6**.

The moment due to wave force decreases as the spacing between the piles is increased, but this effect reaches to the limit at around $b/D = 0.1$. With this type of breakwater, caution should also be paid regarding scouring of the ground between the piles.

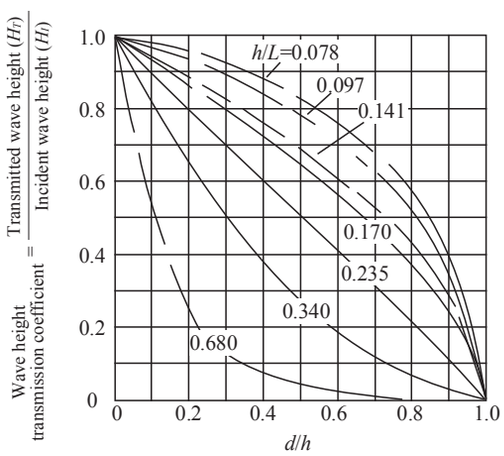


Fig. 3.8.3 Relationship between d/h and Coefficient of Wave Transmission (Single Curtain Wall)

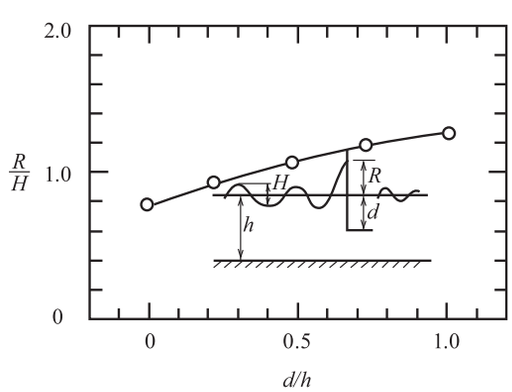


Fig. 3.8.4 Calculated Curve of Crown height (Single Curtain Wall)

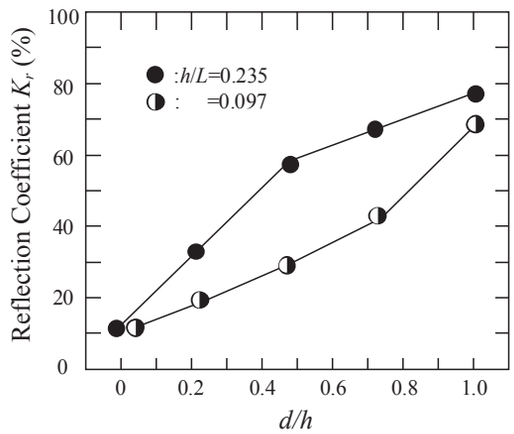


Fig. 3.8.5 Relationship between d/h and Wave Reflection Coefficient (Single Curtain Wall)

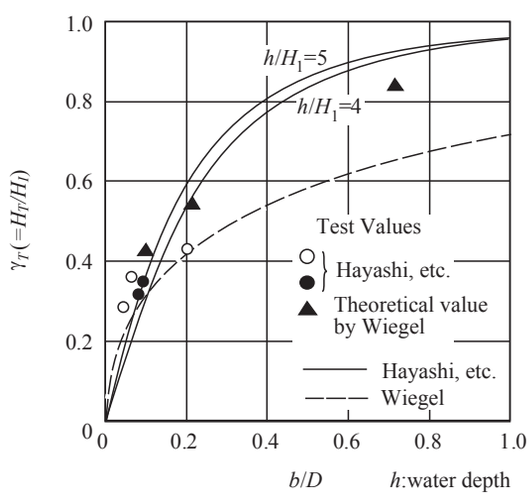


Fig. 3.8.6 Relationship between Ratio of Pile Spacing/Pile Diameter and Coefficient of Wave Transmission [53)]

3.9 Breakwaters with Wide Footing on Soft Ground

[Commentary]

(1) Breakwaters with Wide Footing on Soft Ground (pile foundation)

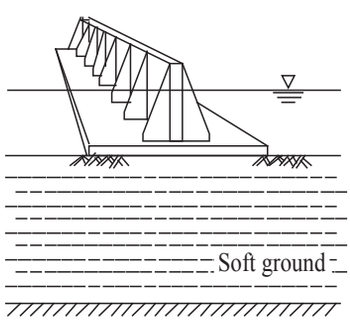
Because breakwaters with a wide footing on soft ground with a pile foundation are a structural type which has the respective structural features of the gravity-type breakwater and the pile-type breakwater, the performance criteria for breakwaters with wide footing on soft ground are equivalent to the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

[Technical Note]

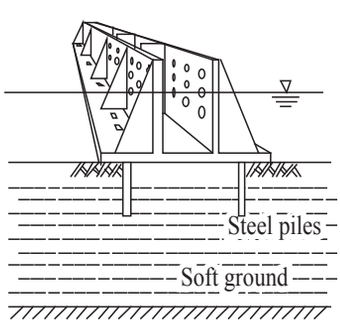
3.9.1 Fundamentals of Performance Verification

(1) Breakwaters with wide footing on soft ground (hereafter, soft landing breakwaters) resist against the horizontal wave force by the piles and the cohesion between the bottom of the breakwater body and the surface layer of the cohesive soil. On the other hand, the bottom slab and footing 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 weight of the breakwater body can be reduced and soil improvement is not required.

(2) Examples of the cross sections of soft landing breakwaters are shown in **Fig. 3.9.1**. 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 type with piles/inverted π type

Fig. 3.9.1 Examples of Cross Sections of Soft Landing Breakwaters

(3) Because the soft landing breakwater is constructed directly on soft ground, it is affected by scouring by waves and water currents in the area around the breakwater body. Therefore, appropriate countermeasures shall be taken as necessary.

3.10 Floating Breakwaters

Public Notice

Performance Criteria of Floating Breakwaters

Article 37
The performance criteria of floating breakwaters under the variable action situation, in which the dominant action is variable waves, shall be as specified in the subsequent 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 lines 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 force acting on the mooring anchor shall be equal to or less than the threshold level.

[Commentary]

(1) Performance Criteria of Floating Breakwaters

① Settings in connection with the performance criteria and the design situation excluding accidental situations of floating breakwaters shall be as shown in **Attached Table 20**.

Attached Table 20 Settings in Connection with Performance Criteria and Design Situations (excluding accidental situations) of Floating Breakwaters

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating actions	Non-dominating actions		
14	1	2	37	1	1	Serviceability	Variable	Variable waves	Self weight, wind, water pressure, water currents	Capsizing of floating body	Limit value for capsizing
					2					Integrity of members	-
					3					Yielding of mooring lines	Design yield stress
					4					Stability of mooring anchor, etc.	Resistance (horizontal and vertical) of mooring anchor

② Stability of mooring anchor (serviceability)
Mooring anchor is a collective term for equipment placed on the surface of the sea bottom to fix the floating body. Concretely, in addition to the mooring anchors, sinkers are also included.

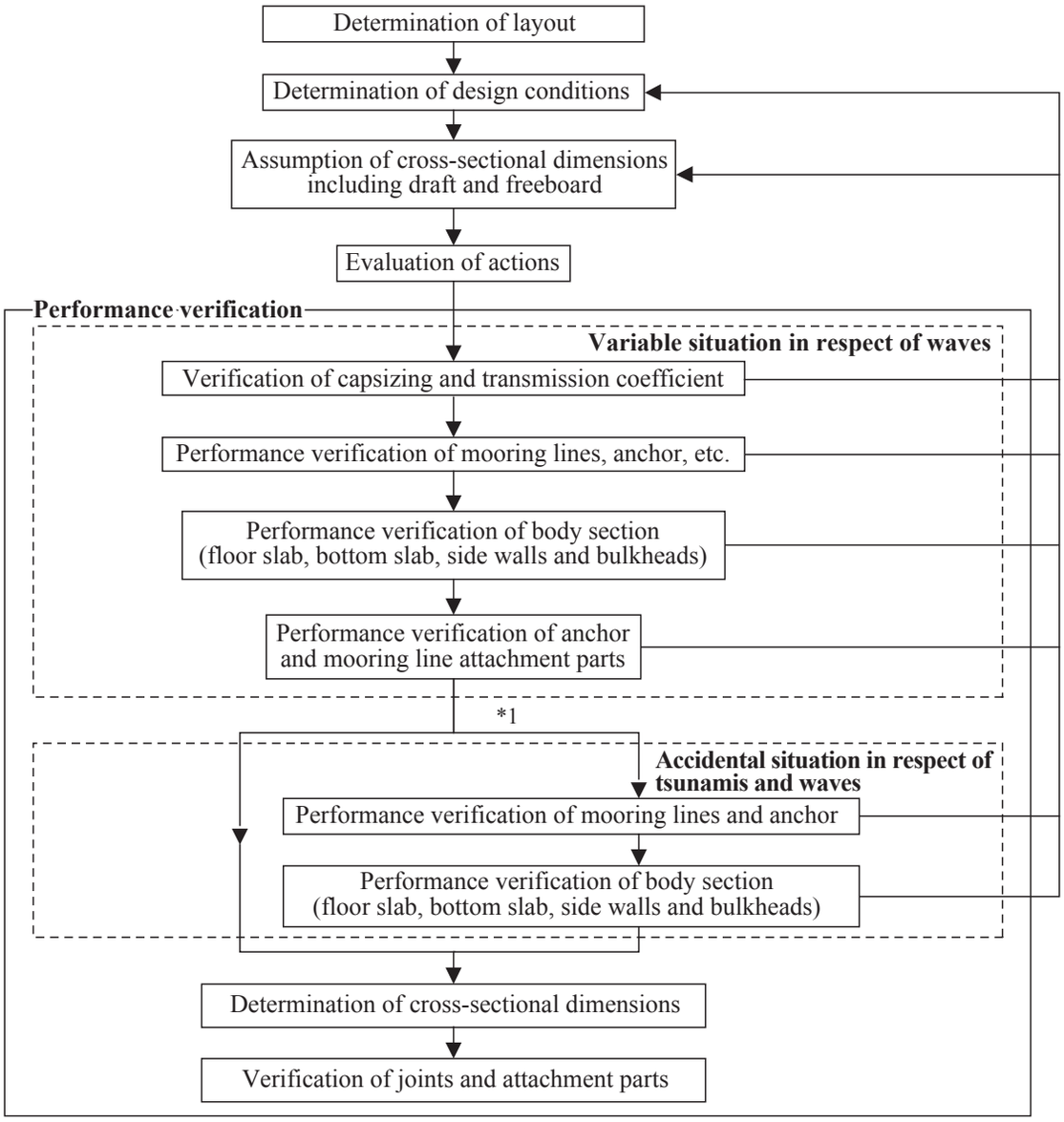
[Technical Note]

3.10.1 Fundamentals of Performance Verification

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

(2) An example of the performance verification procedure for floating breakwaters is shown in **Fig. 3.10.1**.

(3) 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 levels changes or ground conditions, and they are moveable. However, they also have numerous problems, in that they allow large 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 resistance of the anchor system against repeated impulsive actions is not adequately understood. Furthermore, because there is a danger of secondary damage due to drifting of the floating body if the mooring lines break, appropriate measures should be taken.



*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 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. 3.10.1 Example of Performance Verification Procedure for Floating Breakwaters

3.10.2 Setting of Basic Cross Section

The layout and the shape of the floating breakwaters should be set so that the required harbor calmness can be obtained. In determining these settings, it is preferable to measure the wave transmission coefficient by conducting hydraulic model tests. As theoretical analysis methods, Ito et al.[59] proposed an approximation method for the motion of a 2-dimensional rectangular floating body, and Iijima [60] proposed a theory in connection with free floating bodies.

3.10.3 Performance Verification

(1) The performance verification of mooring system can be conducted referring to **Part II, Chapter 2, 4.9 Actions on Floating Body and its Motions**.

(2) Mooring-related design can be divided into two stages:

① First stage in which the tensions that will be exerted on mooring lines and sinkers are determined through static and dynamic analyses by assuming various conditions concerning mooring-related matters such as the mooring method and line length.

② Second stage in which detailed design of the actual mooring lines and sinkers is carried out and the stability is confirmed, based on the tensions and other findings in the first stage above.

(3) Dynamic analysis of the mooring lines consists of determining the fluctuating tension and displacement that arise from the motions of floating body. This analysis can be classified into the following two procedures:

① Methods to analyze these factors based on the static mooring characteristics.

② Methods to analyze these factors based on the dynamic response characteristics of mooring lines.

(4) The performance verification for the mooring anchor is equivalent to that for floating piers. In addition to referring to **Chapter 5, 6.4 Performance Verification**, Reference 62) can also be used as a reference.

(5) The structure of the floating body of a floating breakwater shall possess adequate safety as a whole, and shall also possess adequate local strength. With structures having a relatively long length relative to their width and depth, such as floating breakwaters, it is generally preferable to examine the following points.
Longitudinal strength: The cross-sectional forces such as longitudinal flexural moment, shearing force and torsional moment in the permanent situation and under action of waves shall be obtained for the floating body as a whole.
Lateral strength: The cross-sectional forces such as flexural moment and shearing force in the direction perpendicular to the longitudinal axis under action of waves shall be obtained for the floating body as a whole.
Local strength: The cross-sectional forces such as flexural moment and shearing force generated in individual wall panels and girders shall be obtained.

(6) Longitudinal strength calculation methods are divided into two categories, one of which considers floating body motions, while other that does not. Among calculation methods that do not consider floating body motions, the Muller equation, the Prestressed Concrete Barge Standards, and the Veritus Rule are frequently used. On the other hand, the Ueda's formulae [63)] is used as a calculation method that does take into account the floating body motions. A comparison of the methods of both categories is cited in the References 63), which can be referred to when applying the calculations.

(7) The performance verification for the stability of the floating body is equivalent to that for floating pier. **Chapter 5, 6.4 Performance Verification** can be used as a reference. For other concepts in connection with the verification of stability when inundated, Reference 64) can be used as a reference.

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4 Amenity-oriented Breakwaters

It is necessary to examine the crown height of the amenity-oriented breakwaters which will be visited by the general public from the viewpoint of public use and safety, including spray, and the wave overtopping.

References

1) Coastal Development Institute of Technology : Technical Manual for the Improvement of Port environment, 1991
2) TAKAHASHI, S., Kimihiko ENDOH and Zen-ichirou MURO: Experimental Study on People's Safety against Overtopping Waves on Breakwaters- A study on Amenity-oriented Port Structures (2nd Rept.)-, Rept. of PHRI Vol. 31 No.4, 1992

5 Storm Surge Protection Breakwaters

The performance verification for storm surge protection breakwaters can be considered equivalent to **3 Ordinary Breakwaters.** In addition to this, the following points need to be considered corresponding to the structural type.

5.1 Fundamentals of Performance Verification

(1) In the storm surge protection breakwaters, it is necessary to set the layout, and crown height appropriately, considering the effect of the breakwater in reducing the effects of storm surge.

(2) In the storm surge protection breakwaters, in addition to the stability of the facilities against the action of waves, it is also necessary to secure the stability of the facilities considering the characteristics of attack by storm surges such as the rise in the water level inside the breakwater.

5.2 Actions

In the examination of the stability of the upright section, the rise in the water level inside the breakwater due to the inflow of the storm surge shall be considered. In this case, **Part II, Chapter 2, 4 Waves** and **Part II, Chapter 2, 3 Tidal Level** can be used as a reference for waves and tidal levels, respectively.

5.3 Setting of Basic Cross Section

The crown height of the storm surge protection breakwaters shall be the required height based on appropriate consideration of the waves and tidal levels at the construction site. For waves and tidal levels, **Part II, Chapter 2, 4 Waves** and **Part II, Chapter 2, 3 Tidal Level** can be used as a reference, respectively.

References

1) JSCE: Handbook of coastal facilities (2009 Edition), pp465-468,2000

6 Tsunami Protection Breakwaters

The performance verification for tsunami protection breakwaters can be considered equivalent to **3 Ordinary Breakwaters.** In addition to this, the following points need to be considered, corresponding to the structural type.

6.1 Fundamentals of Performance Verification

(1) It is necessary to set the layout and, crown height of the tsunami protection breakwaters, appropriately, considering the effect of the breakwater in reducing the effects of tsunamis.

(2) In addition to the stability against the action of waves, it is also necessary to secure the stability of the tsunami protection breakwaters considering the characteristics during tsunami attack.

6.2 Actions

(1) For tsunamis, **Part II, Chapter 2, 5 Tsunamis** can be used as a reference.

(2) In the performance verification for tsunamis, it is preferable that the difference in the water level inside and outside the breakwater during action of tsunamis be evaluated appropriately based on a numerical simulation. Attention should be paid to the fact that the water level behind the breakwater will not necessarily be the same as the still water level, depending on inflow and outflow of tsunamis.

(3) In the calculation of tsunami force, **Part II, Chapter 2, 5(7) Tsunami Wave Force** can be used as a reference. However, because many points still require clarification, it is preferable to confirm the wave force by an appropriate method such as hydraulic model tests or the like.

6.3 Setting of Basic Cross Section

It is necessary to set the crown height of the tsunami protection breakwaters to the crown height required against wave overtopping in both cases of action of waves and tsunamis at appropriately set tidal levels.

6.4 Performance Verification

(1) In the performance verification of the tsunami protection breakwaters in the accidental situation in respect of tsunamis, in general, an examination shall be performed for the stability against sliding and overturning of the upright section and the failure due to insufficient bearing capacity of the foundation ground.

(2) In the examination of the stability against sliding and overturning of the upright section for tsunamis, equation **(6.4.1)** and equation **(6.4.2)** can be used. In the following equations, the symbol γ is the partial factor for its subscript, and the subscripts d denote the characteristic value.

① Sliding

$$f_d\left(W_d - P_{B_d} - P_{U_d}\right) \geq \gamma_a P_{H_d} \tag{6.4.1}$$

where

f : friction coefficient between bottom of wall body and foundation
W : weight of body (kN/m)
P_B : buoyancy (kN/m)
P_U : uplift force of tsunami (kN/m)
P_H : horizontal wave force of tsunami (kN/m)
γ_a : structural analysis factor

② Overturning of breakwater body

$$a_1 W_d - a_2 P_{B_d} - a_3 P_{U_d} \geq \gamma_a a_4 P_{H_d} \tag{6.4.2}$$

where

W : weight of body (kN/m)
P_B : buoyancy (kN/m)
P_U : uplift of tsunami (kN/m)
P_H : horizontal wave force of tsunami (kN/m)
a_1–a_4 : arm lengths of actions (see **Fig. 3.1.4** of **3.1 Gravity-type Breakwaters (Composite Breakwaters)**)
γ_a : structural analysis factor

The design values of wave force P_{Hd} and P_{Ud} in equation **(6.4.1)** and equation **(6.4.2)** can be calculated using equations **(5.4)** and **(5.5)** in **Part II, Chapter 2, Section2, 5 Tsunamis**. The design value of the weight of the breakwater body W_d can be calculated using equation **(3.1.4)** in **3.1 Gravity-type Breakwaters (Composite Breakwaters)**. When caissons do not have a footing, equation **(3.1.5)** in **3.1 Gravity-type Breakwaters (Composite Breakwaters)** can be used in calculating the design value of buoyancy P_{Bd}.

(3) The examination for the failure due to insufficient bearing capacity of the foundation ground for tsunamis is equivalent to that for variable situations in respect of waves in composite breakwaters. **3.1.4 Performance Verification** can be used as a reference. Provided, however, that the partial factors used in verification shall be in accordance with the following **(4) Partial factors**.

(4) Partial factors
For the partial factors used in the examination of the stability against sliding and overturning of the upright section and the failure due to insufficient bearing capacity of the foundation ground for tsunami protection breakwaters in the accidental situation in respect of tsunamis, the values in **Table 6.4.1** can be used as a reference. Provided, however, that the values shown in **Table 6.4.1** are the standard values when setting the tsunami force of the largest class as the accidental action expected at the location where the facilities are to be constructed. Here, in cases where uncertainty is expected in calculation of the characteristic value of the tsunami force, there are examples in which 1.2 is set as a structural analysis factor.

Table 6.4.1 Partial Factors for use in Performance Verification of Tsunami Protection Breakwaters

			γ	α	μ/X_k	V
Sliding	γ_f	Friction coefficient	1.00	–	–	–
	γ_{PH}, γ_{PU}	Tsunami force	1.00	–	–	–
	γ_{wl}	r_{wl}=1.5	1.00	–	–	–
		r_{wl}=2.0, 2.5	1.00		–	–
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	1.00	–	–	–
	$\gamma_{W_{NC}}$	Unit weight of NC	1.00	–	–	–
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	–	–	–
	γ_a	Structural analysis factor	1.00 or over	–	–	–
Overturning	γ_{PH}, γ_{PU}	Tsunami force	1.00	–	–	–
	γ_{wl}	r_{wl}=1.5	1.00	–	–	–
		r_{wl}=2.0, 2.5	1.00		–	–
		H.H.W.L.	1.00		–	–
	$\gamma_{W_{RC}}$	Unit weight of RC	1.00	–	–	–
	$\gamma_{W_{NC}}$	Unit weight of NC	1.00	–	–	–
	$\gamma_{W_{SAND}}$	Unit weight of filling sand	1.00	–	–	–
	γ_a	Structural analysis factor	1.00 or over	–	–	–
Bearing capacity of foundation ground	γ_{PH}	Tsunami force	1.00	–	–	–
	γ_q	Surcharge on slice segment	1.00			
	$\gamma_{w'}$	Weight of slice segment	1.00			
	$\gamma_{\tan\phi'}$	Ground strength: Tangent of angle of shear resistance	1.00			
	$\gamma_{c'}$	Ground strength: Cohesion	1.00			
	γ_a	Structural analysis factor	1.00 or over			

*1: α: sensitivity factor, μ/X_k: bias of average value (average value/characteristic value), V: coefficient of variation.
*2: RC: reinforced concrete, NC: non-reinforced concrete.
*3: Change of water depth mild/steep: Gradient of sea bottom <1/30/ Longer than 1/30.
*4: r_{wl} denotes the ratio of the highest high water level (H.H.W.L.) and mean monthly-high water level (H.W.L.).

(5) The tsunami protection breakwaters are frequently constructed in locations where the water is deep. In this case, the height of the breakwater body is also large, and the stability during action of ground motion becomes

a particular problem. Therefore, it is preferable to examine seismic resistance by performing seismic response analyses considering the nonlinearity of the mound materials. In addition, it is also preferable to examine the stability of the mound during action of ground motion. The performance verification of the mound for the stability during action of ground motion is equivalent to that for the composite breakwaters; **3.1.4 Performance Verification** can be used as a reference.

6.5 Structural Details

(1) An experimental study by Tanimoto et al.[1] has confirmed that in the situation where a tsunami flows in through a narrow harbor entrance, the flow velocity will increase and there are produced strong vortices that exert a substantial influence on the stability of the armor material of the submerged mound section of breakwater. Tsunami also exercises strong tractive forces on the bed, which are said to be even greater than those by storm surges. Attention, therefore, must be paid in particular to the reinforcement for the stability of the breakwater section at a harbor entrance and to scour prevention works for the foundation ground.

(2) Because 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 wave transformation on the slope surface of the rubble mound. It will also be necessary to make extra-banking for the rubble mound against large settlement of the rubble mound by its own weight.

6.6 Tsunami Reduction Effect of Tsunami Protection Breakwaters

Regarding the effect of tsunami protection breakwaters, oscillation analysis of Ofunato Bay, Iwate Prefecture, for both states before and after the construction of the tsunami protection breakwater when Tokachi-oki Earthquake Tsunami of May 1968 occurred, was carried out based on records of the tidal levels measured in the bay [2] According to the results, the wave height amplification ratio *M,* amplitude at back of bay/amplitude of incident waves, after the construction is reduced in the low order oscillation frequency with a long period *T* was reduced in comparison with that before the construction, as shown in **Fig. 6.4.1**, confirming that tsunami protection breakwaters demonstrate a tsunami reduction effect.[2] This has also been verified by numerical calculations by Itoh et al.[3]

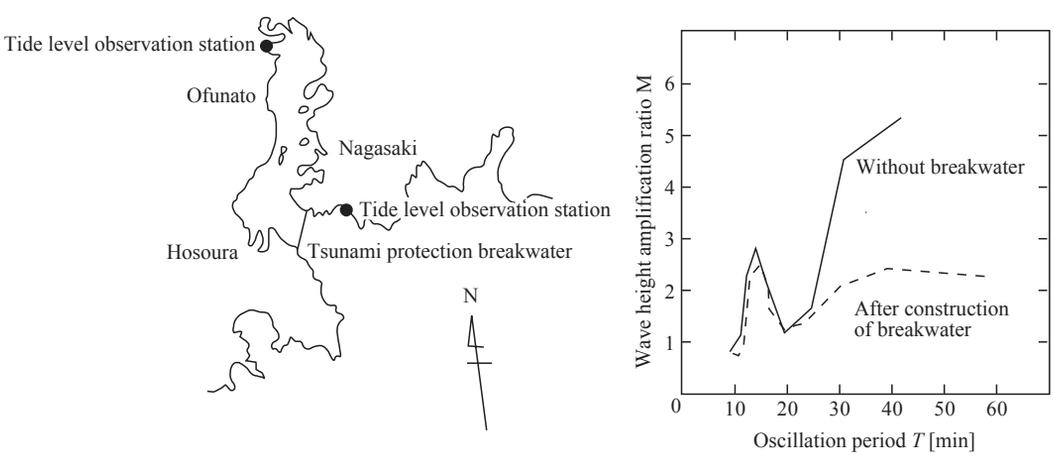


Fig. 6.4.1 Effect of Tsunami Protection Breakwater (Case of Ofunato Bay)

References

1) TANIMOTO, K., Katsutoshi KIMURA and Keiji MIYAZAKI: Study on Stability of Submerged Dike at the Opening Section of Tsunami Protection Breakwaters, Rept. of PHRI Vol. 27 No.4, pp.93-121, 1988

2) Horikawa, K. and H. Nishimura: Performance of Tsunami breakwaters Proceedings of 16th Conference on Coastal Engineering, JSCE, pp.365-369,1969

3) ITO, Y., katsutoshi TANIMOTO and Tsutomu KIHARA: Digital Computation on the Effect of Breakwaters against Long-period Waves (4th Report)- On the Effect of Ofunato Tsunami Breakwater against the Tsunami caused by the Earthquake on May 16, 1968.-, Rept. of PHRI Vol.7 No.4, pp.55-83, 1968

7 Sediment Control Groins

Ministerial Ordinance

Performance Requirements for Sediment Control Groins

Article 15

1 The performance requirements for sediment control groins shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism for the mitigation of siltation in waterways and basins caused by littoral drift through effective control of sediment movement.

2 The provisions of the item (2) of the paragraph 1 of the preceding article shall be applied correspondingly 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 36 shall be applied to the performance criteria of sediment control groins with modifications as necessary 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 these facilities are arranged appropriately so as to enable control of littoral drift, in consideration of the environmental conditions and others to which the facilities concerned are subjected and have the dimensions necessary for their function.

[Commentary]

(1) Performance Criteria for Sediment Control Groins
In the performance verification for sediment control groins, appropriate consideration shall be given to the increase in earth pressure due to the sedimentation by littoral drift and effects due to river currents.

[Technical Note]

7.1 General

(1) Layout of Sediment Control Groins

① Sediment control groins shall be appropriately located by considering the characteristics of sediment transport, so as to exercise the expected function of longshore transport control.

② In general, the sediment control groins on the updrift side of longshore sediment transport, shall be located perpendicular to the shoreline in the surf zone and shallower, and in deeper waters, shall be located so that littoral drift is dispersed to the side opposite the harbor entrance.

③ In cases where sediment control groins are constructed on the downdrift side of longshore sediment transport in order to prevent entrainment of littoral drift into the harbor from the shore on the downdrift side of longshore sediment transport, in general, the groin shall be constructed perpendicular to the coastline and shall also have an appropriate length considering wave direction and wave transformation. Provided, however, that in cases where a sediment control groin also functions as a breakwater, an appropriate layout considering its required functions as a breakwater is necessary.

④ If a sediment control groin in required in places such as the vicinity of waterways inside a harbor, it shall be constructed in an appropriate location in consideration of the natural conditions.

(2) Layout of Updrift Side Breakwaters
It is preferable that the updrift side breakwater is extended beyond the surf zone in the direction perpendicular to the shoreline in order to cause deposition of littoral drift at the updrift side of the breakwater (refer to **Fig. 7.1.1**). When this extension part is short or slanted towards the downdrift side from the shoreline, the efficiency of sediment catchment at the updrift side is reduced and sediment can easily move along the breakwater towards the harbor entrance. When this section is extended with a slant angle towards the downdrift side from the shoreline, it can easily become the cause of local scouring at the updrift side.[1)] In the area deeper than the breaker line, the breakwater shall be slanted so that it simultaneously stops waves and disperses littoral drift toward the updrift side of the harbor entrance with the aid of reflected waves or Mach-stem waves (refer to **Fig. 7.1.1**).

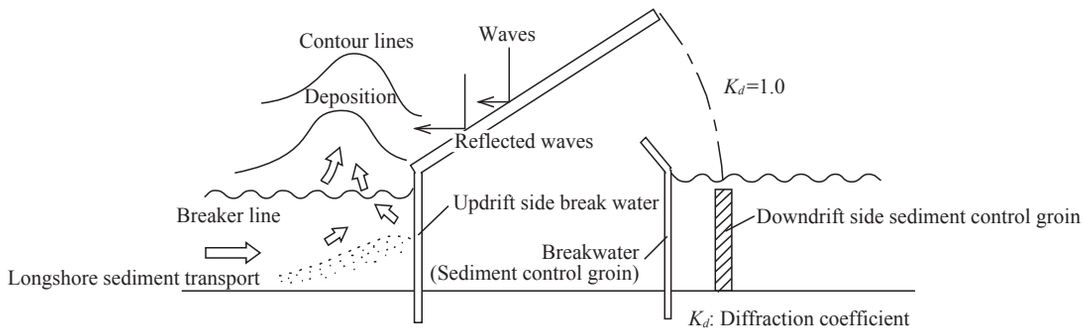
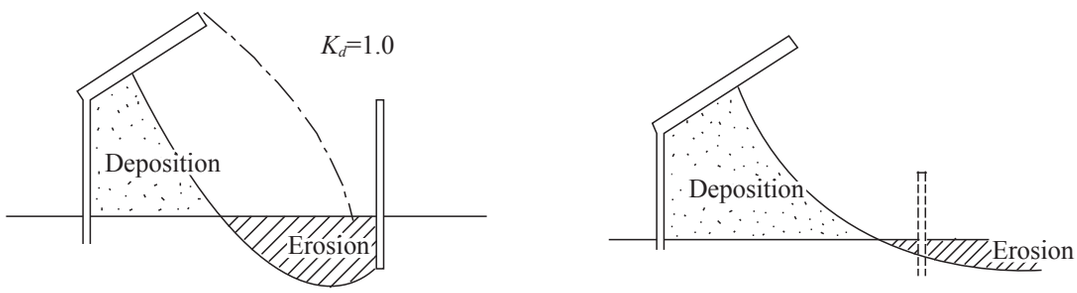


Fig. 7.1.1 Conceptual Layout of Breakwater (Sediment Control Groin)

(3) Position of the Downdrift Side Breakwater and Construction Time

When the updrift side breakwater is extended beyond the extension line of the downdrift side breakwater, deposition will start at the downdrift side of the latter breakwater. Sand bar will then be formed from the shore toward the harbor entrance, and it will cause beach erosion at the far downdrift shore.[2)] If the downdrift side breakwater is extended during construction of the updrift breakwater and the slant section of the latter is not extended enough, remarkable local erosion may be caused at the harbor side of the downdrift breakwater, as shown **Fig. 7.1.2(a)**. Conversely, if the extension of the downdrift breakwater is delayed, it may cause deposition in the harbor and erosion at the downdrift shore as shown in **Fig. 7.1.2(b)**. Very careful attention should therefore be paid to the extension speed of both the updrift and downdrift side breakwaters, and care must be taken to maintain the appropriate balance of extensions.



(a) Case with rapid extension of thedowndrift-side breakwater

(b) Case with slow extension of the downdrift-side breakwater

Fig. 7.1.2 Construction Time of Downdrift Side Breakwater

(4) Length of Breakwater and Water Depth at Tip

Because longshore sediment transport occurs mainly in the surf zone, it is necessary to extend the breakwater offshore beyond the surf zone. In small ports where the water depth at the tip of the breakwater remains in the surf zone during stormy weather, it is difficult to completely prevent littoral drift from entering the port. At major ports in Japan, there are many cases in which the water depth at the tip of updrift side breakwater is approximately equal to the maximum depth of the navigation channels in the port concerned.

(5) Structural Forms of Sediment Control Groin

Because the required function of a sediment control groin is to stop sediment transport firmly, in principal a sediment control groin should have an impermeable structure. Where rubble stones or concrete blocks are used to build a sediment control groin around the shoreline, the core is to be filled with quarry run or small stones of up to 100 to 200 kg; there are also cases where the harbor side of the sediment control groin is covered with impermeable materials such as sand mastic asphalt. In the following situations, it is preferable to adopt the structure of wave-dissipating types.

① When there is a large concern about scouring by currents.

② When there are concerns of shoaling caused by reflected waves or of causing obstruction to the navigation of ships.

7.2 Performance Verification

(1) Crown Height of Sediment Control Groin

Although it is preferable for sediment control groins not to allow overtopping of waves to prevent the inflow of suspended sediment, there are also cases where overtopping is permitted due to structural constrains or by reasons of construction costs. The crown height should be determined by taking the following into considerations:

① Section around shoreline

It is preferable that the crown height of the section around the shoreline of sediment control groins be sufficiently high as to prevent overtopping by running-up waves. Because sand carried by runup waves may overtop the crest of the section around shoreline of the sediment control groin, the crest should be sufficiently high. It is preferable to raise the crown height or extend the groin itself to the landward direction, in view of conditions after construction.

② Sections located shallower than the breaker line depth

The crown elevation of the sediment control groin in the sections located shallower than the breaker line depth may be $0.6H_{1/3}$ above the mean monthly-highest water level (HWL), where $H_{1/3}$ should be the significant wave height around the tip of sediment control groin.

③ Sections located deeper than the breaker line depth

The crown elevation of the sediment control groin in the sections located deeper than the breaker line depth should be a height that is obtained by adding a certain margin to the mean monthly-highest water level. In the water deeper than the breaker zone, the suspended sediment is concentrated near the seabed and overtopping water contains almost no sediment, and therefore overtopping may be permitted.

References

1) Tanaka, N: Transformation of sea bottom and beach near port constructed within the beach, Proceedings of Annual Conference, pp.1-46,1974
2) SATO, S., Norio TANAKA and Katsuhiro SASAKI: The Case History on Variation of Sea Bottom Topography Caused by the Construction Works of Kashima Harbour, Rept. of PHRI Vol. 13 No.4, pp.3-78, 1974
3) Nakase, A., T. Okumura and M. Sawaguchi: Easy-to-understand Foundation works, Kajima Publishing, p.376、1981

8 Seawalls

Ministerial Ordinance

Performance Requirements for Seawalls

Article 16

1 The performance requirements for seawalls shall be as specified in the subsequent items for the purpose of protecting the land area behind the seawall in consideration of its structure type.

(1) Seawalls shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable protection of the land area behind the seawall concerned from waves and storm surges.

(2) Damage due to self weight, earth pressure, variable waves, and Level 1 earthquake ground motions, and/or other actions shall not impair the functions of the seawall concerned and shall not adversely affect its continued use.

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, and/or socioeconomic activity by the damage to the seawall concerned shall include the subsequent items, in consideration of the type of seawall.

(1) The performance requirements for a seawall which is required to protect the land area behind the seawall concerned from tsunamis or accidental waves shall be such that the seawall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable protection of the land area behind the seawall concerned from tsunamis or accidental waves.

(2) Damage due to tsunamis, accidental waves, Level 2 earthquake ground motions, and/or other actions shall not have a serious impact on the structural stability of the seawall concerned, even in cases where the functions of the seawall concerned are impaired. Provided, however, that for the performance requirements for a seawall which requires further improvement of its performance due to environmental, social and/or other conditions to which the seawall concerned is subjected, the damage due to said actions shall not adversely affect the restoration through minor repair work of the functions of the seawall concerned.

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 be applied with modifications as necessary to the performance criteria of seawalls in consideration of the type of structure.

2 In addition to the provisions of the preceding paragraph, the performance criteria of seawalls shall be as specified in the subsequent items:

(1) The seawall shall be arranged appropriately so as to enable control of wave overtopping in consideration of the environmental conditions and others to which the seawalls concerned are subjected and shall have the dimensions necessary for their function.

(2) Under the variable action situation in which the dominant action is water pressure, the risk of losing the stability due to seepage failure of the ground shall be equal to or less than the threshold level.

(3) In the case of the structure having a parapet, the risk of sliding and overturning of the parapet under the variable action situation in which the dominant actions are variable waves and Level 1 earthquake ground motions 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 the seawalls in the place where there is a risk of serious impact on human lives, property, or socioeconomic activity by the damage to the facilities concerned shall be as specified in the subsequent items:

(1) Seawalls which are required to protect the hinterland from tsunamis or accidental waves shall have the dimensions as necessary for protection of the hinterland from tsunamis or accidental waves.

(2) Under the accidental action situation in which the dominant actions are tsunamis, accidental waves, or Level 2 earthquake ground motions, the degree of damage owing to the dominant actions shall be equal to or less than the threshold level corresponding to the performance requirements.

[Commentary]

(1) Performance Criteria for Seawalls

① Common performance criteria for seawalls

The settings in connection with the performance criteria and design situations excluding accidental situations for the stability of the facilities of seawalls shall be as shown in the **Attached Table 21**. In the performance criteria for seawalls, in addition to these provisions, the settings in connection with the **Public Notice, Article 22, Item 3 (**Scouring and Sand Washing Out) and **Article 28 Performance Criteria of Armor Stones and Blocks** shall apply, as necessary, and depending on the type of members comprising the objective seawall, the setting in connection with **Article 23** through **Article 27** shall also apply.

Attached Table 21 Settings for Performance Criteria and Design Situations (excluding accidental situations) of Stability of Facilities Common to Seawalls

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
16	1	2	39	2	2	Usability	Variable	Water pressure	Self weight	Seepage failure of ground	Limit value for seepage failure
					3			Variable waves	Self weight, earth pressure, water pressure	Sliding or overturning of parapet*1)	Limit value for sliding Limit value for overturning
								Level 1 earthquake ground motion	Self weight, earth pressure, water pressure		Limit value for sliding Limit value for overturning

*1): Limited to structures having parapets.

② Seawalls as facilities against accidental incidents

(a) Stability of facilities (safety, restorability)

1) The settings in connection with the performance criteria and design situations limited to accidental situations of seawalls designed as facilities against accidental incidents shall be as shown in the **Attached Table 22**. In performance verification of seawalls as facilities against accidental incidents, among the settings in connection with the performance criteria and design situations for the accidental situations of Level 2 earthquake ground motion, tsunamis, and accidental waves, values shall be set appropriately corresponding to the structural type of the objective seawall and the performance requirements of the objective seawall.

The items safety and restorability are specified in the performance requirements in the **Attached Table 22** because the performance requirements will differ depending on the functions required in the objective seawall designed as facilities against accidental incidents.

As performance criteria in connection with accidental situations for seawalls designed as facilities against accidental incidents, in addition to these provisions, the settings in connection with the **Public Notice Article 22 Performance Criteria Common to Structural Members** shall also apply as necessary.

Attached Table 22 Settings for Performance Criteria and Design Situations limited to Accidental Situations for Seawalls as Facilities against Accidental Incidents

Ministerial Ordinance			Public Notice			Performance requirements	Design situation			Verification item	Index of standard limit value
Article	Paragraph	Item	Article	Paragraph	Item		Situation	Dominating action	Non-dominating action		
16	1	2	39	3	2	Safety, restorability	Accidental	Level 2 earthquake ground motion (Tsunami) (Accidental wave)	Self weight, earth pressure, water pressure	Damage	–

*1): Limited to structures having parapets.

2) Degree of damage
In setting the limit value of the degree of damage for accidental situations in which the dominating actions are Level 2 earthquake ground motion, tsunamis, and accidental waves in the performance verifications of seawalls as facilities against accidental incidents, consideration shall not be limited to the functions of the objective seawall, but shall also include comprehensive considerations of the condition of implementation of the surrounding protective facilities for the harbor and other facilities for protection of the hinterland, and soft countermeasures related to disaster reduction and disaster prevention in the objective region. In seawalls used as facilities against accidental incidents in which restorability is a performance requirement, appropriate consideration shall be given to the allowable restoration period when setting the limit value of the degree of damage.

3) Accidental situation in which dominating action is tsunami
In the performance verifications in connection with tsunamis, in cases where the expected tsunami occurs as a result of an earthquake with a hypocenter located near the objective facilities, appropriate consideration shall be given to the fact that the facilities will be affected by the action of the ground motion caused by the objective earthquake before they are affected by the action of the tsunami. In other words, in cases where the dominating action is the accidental situation associated with tsunamis, it is necessary to conduct the performance verification for tsunamis based on consideration of the effects caused by the action of the ground motion which precedes a tsunami. It should be noted that the ground motion which precedes the tsunami which is expected in this case is not necessarily identical with the Level 2 earthquake ground motion.

References

1) Shore protection facility Technical Committee: Technical standards and commentary for shore protection facilities, Japan Port Association, 2004

9 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 prevention of closure of a river mouth by littoral drift through effective control of sediment transport.

2 The provisions of the item (2) of the paragraph (*1*) of Article 14 shall be applied correspondingly to the performance requirements for training jetties.

Public Notice

Performance Criteria of Training Jetties

Article 40

The provisions of Article 38 shall be applied to the performance criteria of training jetties with modifications as necessary.

[Commentary]

(1) Performance Criteria of Training Jetties

The settings in connection with the **Public Notice, Article 38 Performance Criteria of Sediment Control Groins** shall be applied with the necessary modifications to the performance criteria of training jetties. In the performance verifications of training jetties, appropriate consideration shall be given to the increase of earth pressure due to sedimentation of littoral drift and to waves and river currents.

[Technical Note]

9.1 General

(1) Layout of Training Jetties

Examples of the layout of training jetties in relation to the direction of longshore sediment transport are shown in **Fig. 9.1.1**.[1)] The most preferable one for maintaining the water depth of river mouth 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, usually it is effective to make the training jetty on the downdrift side longer. Bending the updrift training jetty towards the downdrift side will prevent sediment moving into the area between two training jetties and make the sediment transported alongshore pass smoothly to the downdrift side. For actual examples of river mouth improvement, refer to the reference 2).

(2) Water Depth at Tip of Training Jetties

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 equal to or greater water depth than the limiting wave breaker depth.

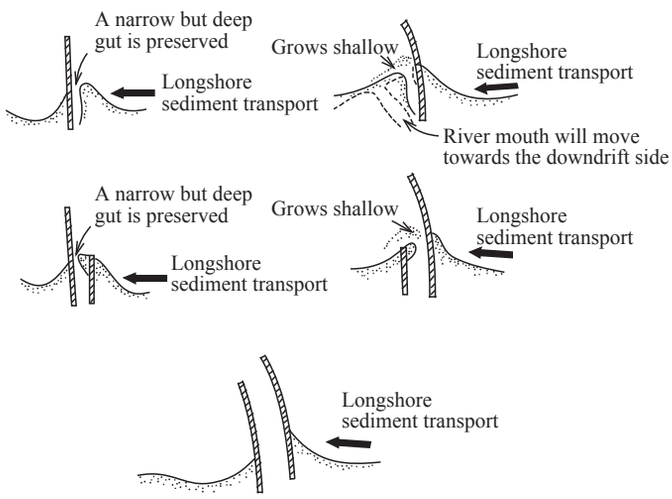


Fig. 9.1.1 Varieties of Training Jetty Layout [1)]

9.2 Performance Verification

Because the training jetty is generally longer than groins and is exposed to intensive wave actions, it is necessary to consider scouring at the tip and sides of a jetty. In addition, it should be considered that the river side of the training jetty will be subject to scouring action by the river current.

References

1) JSCE: Handbook of Civil Engineering, (Vol. 2), pp.2268-2270,1974

10 Floodgates

Ministerial Ordinance

Performance Requirements for Floodgates

Article 18

1 The performance requirements for floodgates shall be as specified in the subsequent items for the purpose of protecting the hinterland of the floodgate from inundation and of draining unnecessary inland water.

(1) Floodgates shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism for prevention of overflow due to storm surges.

(2) Floodgates shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism for protection of the hinterland from inundation and for drainage of unnecessary inland water.

(3) Damage due to self weight, water pressure, variable waves, Level 1 earthquake ground motions, or other actions shall not impair the functions of the floodgate concerned and not affect its continued use.

2 In addition to the provisions of the preceding paragraph, the performance requirements for floodgates which have a risk of having a serious impact on human lives, property, and/or socioeconomic activity by the damage to the floodgate concerned shall include the subsequent items in consideration of the type of floodgate.

(1) In the performance requirements for a floodgate which is required to protect the hinterland of the floodgate concerned from tsunamis or accidental waves, the floodgate shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to enable protection of the hinterland of the floodgate concerned from overflows by tsunamis or accidental waves.

(2) The damage due to tsunamis, accidental waves, Level 2 earthquake ground motions, or other actions shall not have a serious impact on the structural stability of the floodgate concerned, even in cases where the functions of the floodgate concerned are impaired. Provided, however, that as for the performance requirements for floodgates which require further improvement in the performance due to environmental, social, or other conditions to which the floodgates concerned are subjected, the damage due to said actions shall not affect the restoration through minor repair works of the functions of the floodgate concerned.

Public Notice

Performance Criteria of Floodgates

Article 41

1 The performance criteria of floodgates shall be as specified in the subsequent items:

(1) Floodgates shall be located appropriately so as to enable protection of the land behind the facilities from inundation and drainage of unnecessary water accumulated there in consideration of the environmental conditions and others to which the facilities concerned are subjected and shall have the dimensions necessary for their function.

(2) Floodgates shall have the dimensions necessary in consideration of storm surges, waves, and tsunamis.

(3) Under the permanent action situation in which the dominant action is self weight, the risk of impairing the integrity of the members and losing the structural stability shall be equal to or less than the threshold level.

(4) Floodgates shall satisfy the following standards under the variable action situation in which the dominant action is water pressure:

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

(b) The risk of losing the structural stability due to seepage failure of the ground shall be equal to or less than the threshold level.

(5) Floodgates shall satisfy the following standards under the variable action situation in which the dominant actions are variable waves and Level 1 earthquake ground motions:

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

b) The risk of losing the stability of floodgate system shall be equal to or less than the threshold level.

2 In addition to the provisions of the preceding paragraph, the performance criteria of floodgates in which there is a risk of serious impact on human lives, property, or socioeconomic activity by the damage to the facilities concerned shall be as specified in the subsequent items:

(1) Floodgates which are required to protect the hinterland from tsunamis or accidental waves shall have the dimension necessary to control overflows.

(2) Under the accidental action situation in which the dominant actions are tsunamis, accidental waves, or Level 2 earthquake ground motions, the degree of damage owing to the dominant actions shall be equal to or less than the threshold level corresponding to the performance requirements.

[Technical Note]

(1) Layout and Dimensions of Floodgates

① Layout
In setting the layout in the performance verifications of floodgates, it is necessary to give appropriate consideration to installation in a position where the water gate can demonstrate its full water collecting capacity, and to avoiding installation in positions where sediments will tend to accumulate due to the effects of wind, waves, and water currents.

② Structure
In setting the structure of the transitional part of gate in the performance verifications of floodgates, it is necessary to give appropriate consideration to the quality, shape, and dimensions of the materials and to a watertight structure so as to secure the required water-tightness.

③ Cross-sectional dimensions
In setting the height and other dimensions in the performance verifications of floodgates, it is necessary to give appropriate consideration to the dewatering capacity of the objective floodgate, the effects of littoral drift and settlement of the ground, the water levels inside and outside the objective water gate and in the surrounding ground. In floodgates which allow passage of ships, when setting the height, appropriate consideration shall be given to setting a height which will not impede the passage of ships.

④ Ancillary equipment
In the performance verifications of floodgates, it is necessary to examine the installation of ancillary equipment for use in maintenance control, such as control bridges, stairs, handrails, as necessary, so as to enable safe and smooth operation and maintenance control of the gate.

References

1) Shore Protection Facility Technical Committee: Technical standards and commentary for shore protection facilities, Japan Port Association, 2004

11 Locks

Ministerial Ordinance

Performance Requirements for Locks

Article 19

1 The performance requirements for locks shall be as specified by the Minister of Land, Infrastructure, Transport and Tourism for the purpose of enabling the safe and smooth navigation of ships between the water areas having different water levels.

2 The provisions of the items (1) and (3) of the paragraph (*1*) and the paragraph (*2*) of the preceding article shall be applied correspondingly to the performance requirements for locks.

Public Notice

Performance Criteria of Locks

Article 42

1 The provisions of the preceding article shall be applied to locks with modifications as necessary.

2 In addition to the provisions of the preceding paragraph, the performance criteria of locks shall be such that the locks are located appropriately so as to enable ships to navigate safely and smoothly in consideration of the environmental conditions to which the facilities concerned are subjected, the utilization conditions, and others, and the locks have the dimensions necessary for their function.

[Commentary]

(1) Performance Criteria for Locks

① Safe and smooth navigation of ships (usability)

(a) The specifications of locks shall comprise the structure and cross-sectional dimensions of the lock and the ancillary equipment. In setting the layout and dimensions in the performance verifications of locks, the settings in connection with the **Public Notice, Article 41 Performance Criteria of Floodgates** shall be applied; in addition, appropriate consideration shall be given to the necessary conditions for safe and smooth navigation of ships.

(b) Cross-sectional dimensions
In the performance verifications of locks, water depth, width, and length shall be set appropriately considering the respective clearances, based on appropriate consideration of the effects of the dimensions and motion of the design ship and the expected traffic volume.

(c) Ancillary equipment
In the performance verifications of locks, the layout of the ancillary equipment for maintenance control, including emergency equipment, lighting equipment, power-related equipment, monitoring and instrumentation equipment, and maintenance and control equipment shall be examined, as necessary, in order to secure safe and smooth operation of the objective lock.

[Technical Note]

(1) General

① The names of the respective parts of locks shall be as shown in **Fig. 11.1**.

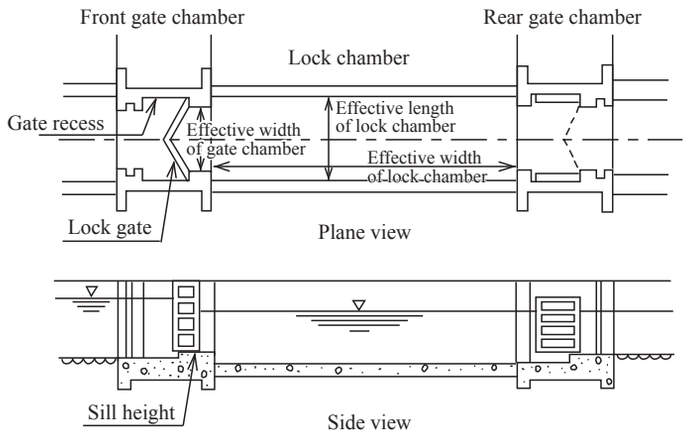


Fig. 11.1 Names of Respective Parts of Lock

② Installation position of locks

(a) There are cases in which locks impose constraints on the functions of the surrounding harbor, for example, by limiting the area of basins, land designated for extension of mooring facilities, and hinder other navigating ships depending on whether the installation position of the lock is appropriate or not. The natural conditions at the installation position also have a large effect on construction costs. Accordingly, it is preferable to use due care in selecting the positions of locks.

(b) It is preferable that installation of locks on soft ground be avoided whenever possible. However, in cases where installation on soft ground is unavoidable, adequate countermeasures should be taken for uneven settlement. Because the functions of the lock will decline by settlement of the gate at locations where ground settlement occurs, it is preferable to raise the crown height in advance in such cases.

(c) Because ship's ingress and egress may become difficult owing to the factors such as winds, waves, tidal currents, and littoral drift, it is optimal to choose a calm water area for the lock location. In cases where the water is not calm, breakwaters should be constructed, or training jetties or guiding jetties should be extended to make the water zone calm in the vicinity of the lock.

(d) The size and number of ships that will pass through the lock are also factors in the selection of the location. That is, the lock must be located at the site where a sufficiently wide area of water can be secured for anchorage and turning basin for use by waiting ships.

(e) In addition to the above, the lock's location must be selected with adequate consideration given to the conditions of land usage or traffic conditions of the inland area.

③ Size and shape of locks

(a) The scale of the lock chamber can generally be set based on equation **(11.1)**. In this case, appropriate values shall be set considering the keel clearance, beam clearance, and length clearance mentioned in the following items, considering the motion of traffic ships.

$$\left.\begin{array}{l}\text{Effective water depth}\\ \quad = \text{Draft of ship passing through lock} + \text{Keel clearance}\\ \text{Effective width}\\ \quad = \text{Beam of ships passing through lock} \times \text{Number of ships in parallel}\\ \quad\quad \times \text{Beam clearance}\\ \text{Effective length}\\ \quad = \text{Length of ships passing through lock} \times \text{Number of ships in one line}\\ \quad\quad \times \text{Length clearance}\end{array}\right\} \qquad (11.1)$$

(b) Generally, the clearances for the various dimensions for locks depend upon the ship size. Fukuda, however, has proposed the following values for locks used by small ships:

Clearance for effective water depth: 0.2–1.0m
Clearance for effective width: 0.2–1.2m
Clearance for effective length: 3–10 m

(2) Performance Verification

① Lock doors

The doors of locks should have a structure which makes it possible to secure the assumed difference in water levels and the required stability against actions due to waves, and should also have a structure which satisfies the following requirements.

1) It shall consider the scale of the lock, time required for opening and closing.
2) It shall be easy to inspect the machinery section and other moving parts.
3) It shall consider wear and prevention of corrosion of members.

② Lock chamber

The lock chamber shall have a structure appropriate to meet the conditions such as the foundation condition, water level difference between inside and outside the lock chamber, the dimensions and number of ships to be accommodated, and the quantity of water changing and discharging of the lock chamber.

References

1) Nishihata, I.: Design of Water Gate and Lock Gate, Ohom Publishing, 2004
2) Fukuda, H.: Lock, Jyo-ritsu Publishing, 1955
3) Planning Division, The third Port Construction Bureau, Ministry of Transport: Storm surge countermeasure works (Improvement of Lock gate)at the coast of Amagasaki, Nishinomiya and Ashiya, Disaster Prevention in Ports and Harbours, Association of disaster Prevention in Ports and coast, pp.41-45,1990

12 Revetments

Ministerial Ordinance

Performance Requirements for Revetments

Article 20

1 The provisions of Article 16 shall be applied correspondingly to the performance requirements for revetments.

2 In addition to the provisions of the preceding paragraph, the performance requirements for revetments to be utilized by an unspecified large number of people shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to secure the safety of the users of the revetment concerned.

12.1 Common Items for Revetments

Public Notice

Performance Criteria of Revetments

Article 43

1 The provisions of Article 39 shall be applied to the performance criteria for revetments with modifications as necessary.

2 In addition to the provisions of the preceding paragraph, the performance criteria for the revetments which are utilized by an unspecified large number of people shall be such that the revetments have the dimensions necessary to secure the safety of users in consideration of the environmental conditions to which the facilities concerned are subjected, and the utilization conditions, and others.

[Commentary]

(1) Performance Criteria of Revetments

① Amenity-oriented revetments (usability)

(a) In setting the structure and dimensions in the performance verifications of amenity-oriented revetments, consideration shall be given to the effects of wave overtopping and spray, prevention of slipping and falling and falling into the water of users, and smooth implementation of rescue activities for users who have fallen into the water. Ancillary equipment such as fences to prevent falling shall be installed appropriately.

[Technical Note]

12.1.1 Fundamentals of Performance Verification

(1) In cases where a reclamation revetment is built adjoining to the 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 aspects when studying the reclamation layout plan and 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 thought that reclamation revetment construction will cause deterioration of the groundwater quality, countermeasures such as construction of a watertight wall must be considered in order to insulate the groundwater of the land from the reclaimed area.

(2) In the case of reclamation where a large water area is enclosed by revetments, the opening becomes smaller with the progress of revetment construction, and a considerable rapid flow occurs at closing sections due to the difference of water levels between the inside and outside of revetments. Therefore, careful consideration is required for structure of revetments at the final closing section, which should have enough stability against the expected flow speed.

The flow velocity at closing sections is controlled by the water area being closed, the cross-sectional area of the closing section, the average water depth and the difference in tidal levels. In closing sections, it is preferable that ground hardening work be conducted at a location with good ground before the flow velocity increases as work progresses. Depending on the flow velocity at the closing section, there are also cases in which a submerged weir or broad-crested weir is used.

12.1.2 Actions

(1) For the ground conditions of landfill soil, **Part II, Chapter 3 Geotechnical Conditions** can be used as a reference.

(2) For actions due to ground motion, **Part II, Chapter 4 Earthquakes** can be used as a reference.

(3) For dynamic water pressure, **Part II, Chapter 5, 2.2 Dynamic Water Pressure** can be used as a reference.

(4) As the water level in reclaimed areas, two water levels are generally set, these being the water level in the reclaimed area and the residual water level. The water level in the reclaimed area is used in seepage calculations and the performance verification of waste water treatment facilities. The residual water level is the water level immediately behind the revetment and is used in examination of the stability of the revetment. Provided, however, that in cases where the water level at positions near the revetment is higher than the residual water level, the danger of circular slip failure may be underestimated if the residual water level is used in the examination of circular slip failure. In such cases, it is necessary to conduct the examination of the stability of the revetment for the water level in the reclaimed area.

① Water level inside reclamation
The water level inside the reclamation area should be established by considering the stability of revetment both during the construction and after completion, and the influence on the surrounding water. Regarding the influence on the surrounding waters, particular caution should be paid in connection with overtopping flows due to waves generated inside revetments during construction. If the water level inside the reclamation area is excessively high in comparison with the water level at the front of the revetment, the water discharge of polluted water from the revetment and foundation ground may increase; therefore, caution is necessary. Furthermore, attention shall also be paid to the fact that the water level inside the reclaimed area will influence the cost of construction of the revetment and the construction and maintenance control costs of waste water treatment facilities.

② Residual water level

(a) For reclamation revetments, the structures with low permeability are often used to reduce to the seepage of contaminated water through revetments. For this reason, the residual water level behind them is generally higher than that behind quaywalls or ordinary revetments.

(b) Reviewing examples of the past construction, in reclamation revetments with gravity-type structures, there are more cases in which permeability is reduced by increasing the layer thickness of the levee-widening earth or the backfilling sand than by reducing the permeability of the revetment body itself. Accordingly, in revetments of this type, the residual water level used in the performance verification of the revetment body should be the same as in ordinary gravity-type revetments, as the water level just behind the revetment body shows behavior similar to that in ordinary gravity-type revetments.

(c) For reclamation revetments using a sheet pile, there are examples where grout material is poured into the sheet pile joint or a double sheet pile structure is used to increase the watertightness. For these cases, the residual water level behind the reclamation revetment tends to be higher than that behind the ordinary sheet pile quaywalls.

(5) In case of reclamation using suction dredgers, there are cases in which suspended soft soil concentrates behind the revetment and 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 phenomena in the performance verifications.

12.1.3 Performance Verification

(1) In the performance verifications of revetments, the following items shall generally be examined.

① The crown height shall be the height to enable preservation and use of the reclaimed land unaffected by waves and storm surges.

② Stability against the actions of waves, earth pressure, etc. shall be secured.

③ The structure shall prevent 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 In amenity-oriented revetments, safe and pleasant use of the structure by users shall be possible.

(2) Setting of Crown Height

① For revetments, an appropriate crown height shall be set considering the wave overtopping quantity, tidal level at high tide so as to enable preservation of the landfill behind the revetment and not hinder use of the revetment or the land behind it.

② In setting the crown height of revetments, the following method [1)] can be used.

(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 corresponding to the importance of the hinterland, or the required crown height h_c' considering ground motion and the crest settlement d_s due to consolidation obtained from the ground conditions.

$$h_d = \max(h_c, h_c') + d_s \tag{12.1.1}$$

(b) The required crown height h_c above the water level in equation **(12.1.2)** shall be a value obtained by adding a height allowance to the calculated crown height for the design wave at the design high water level of the revetment. The required crown height h_c above the water level can be calculated by setting the exceedence probability P for the permissible wave overtopping rate. The exceedence probability P for the permissible wave overtopping rate can be calculated using equation **(12.1.2)**. For the mean value and the standard deviation of h_c/h_{cd}, 1.00 and 0.15 can be used, respectively.

$$P = 1 - \int_0^x \frac{1}{\sqrt{2\pi}\, z\varsigma} \exp\left\{ -\frac{1}{2}\left(\frac{\ln z - \lambda}{\varsigma} \right)^2 \right\} dz \tag{12.1.2}$$

Provided, however, that

$$z = \frac{h_c}{h_{c_d}}$$

where

P : exceedence probability of permissible wave overtopping rate

h_c : required crown height above water level (m)

h_{cd} : calculated crown height for design wave at design high water level of revetment (m)

ζ : standard deviation of $\ln(h_c/h_{cd})$; given by $\zeta = \sqrt{\ln \frac{(1+\sigma^2)}{\mu}}$

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} (= 1.00 can be assumed)

σ : standard deviation of h_c/h_{cd} (= 0.15 can be assumed)

Equation **(12.1.2)** is shown graphically in **Fig. 12.1.1**. For example, assuming the exceedence probability of the permissible wave overtopping rate is 0.01, the required crown height h_c above the water level, which is obtained by adding a height allowance to the calculated crown height h_{cd}, is given as 1.40 times the calculated crown height h_{cd}.

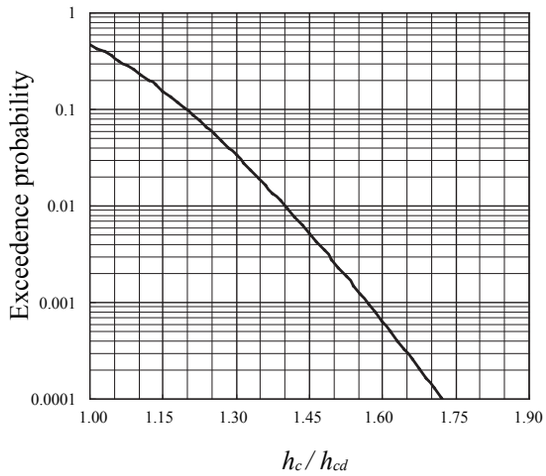


Fig. 12.1.1 Relationship of Exceedence Probability of Permissible Wave Overtopping Rate to h_c/h_{cd}
(Required Crown height above Water Level / Calculated Crown height)

(3) In order to estimate the quantity of seepage of polluted water into the sea from reclamation revetments, 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 below, the cross section of a revetment consists of different materials, including sheet piles and concrete members, and backfilling sand. Furthermore, permeability of sheet piles will differ at the joints and in the sheet piles themselves. For this reason, there are cases in which Darcy's law cannot be applied.

In analysis of seepage flows in this case, 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, applying ingenuity in order to apply Darcy's law in an approximate manner.

In seepage flow analysis, the scope of analysis extends to the point where the water level within the reclaimed area can be considered uniform. However, analysis can be performed by setting the scope corresponding to the required accuracy, considering the structure of the revetment body, and condition of backfilling sand. Provided, however, that caution is necessary when the permeability of the landfill soil deposited in the reclaimed area is itself low, as the water level within the reclaimed area 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. However, it can be applied by using an appropriate equivalent width and the equivalent coefficient of permeability for that width. In addition, because it cannot be assured that a laboratory test could reproduce the joint conditions of the protorype structure in proper scale, it is preferable to use the values measured in-situ.

(b) Reference 11) is available concerning the permeability of steel sheet pile-type structures. It describes the result of analyses taking into account the in-situ measurements on residual water levels at five project sites. In the analyses, it was assumed that the sheet pile wall below the seabed are impermeable and the part of wall above the seabed is equivalent to the permeable layer of 1 m thick to which Darcy's law can be applied. The results obtained for the coefficient of permeability, equivalent coefficient of permeability, were in the range of $1\text{x}10^{-5}$ – $3\text{x}10^{-5}$ cm/s. The results of the similar analysis carried out for two examples of steel pipe pile-type quaywall with diameter of approximately 80 cm yielded a value of $6\text{x}10^{-5}$ cm/s. The coefficient of permeability for backfilling material of the foregoing surveys was in the range of 10^{-2} – 10^{-3} cm/s.

(c) The permeability of sheet pile joint has the following characteristics:
In cases without backfilling material, the sheet pile joint is similar in nature to a narrow orifice of abrupt sectional reduction, and can be expressed in equation (**12.1.3**) with the constant $n = 0.5$ [12), 13)]

$$q = Kh^n \tag{12.1.3}$$

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 : constant

In cases with backfilling material, the property of the backfilling material greatly affects the quantity of seepage through the joint. In the vicinity of the backfilling material behind the sheet pile joint, there are areas at which Darcy's law cannot be applied. There has been an effort to evaluate the permeability as a composite joint that includes a certain thickness of backfill and sheet pile joint. This idea is effective for seepage

analysis. Shoji et al.[14] proposed an empirical equation based on the comprehensive tests considering both the difference in the degree of tensile force in the joint and conditions with or without sand filling. From the results of the tests, for the case that there is backfilling and joints are filled with sand, it was found that the constant n could be given an approximate value of 1.0 and the K value representing the results of the tests was derived.

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 comprising the natural ground. In calculating the coefficients of permeability for each soil layer, **Part II, Chapter 3, 2.2.3 Hydraulic Conductivity of Soil** can be used as a reference. In ground which was formed by natural sedimentation, the coefficient of permeability displays directionality, and in many cases, the coefficient of permeability is larger in the horizontal direction than in the vertical direction.

(b) Permeability of soil improvement sections
In cases where soil improvement is to be carried out as part of construction of a reclamation revetment, in addition to evaluation of the permeability of the natural ground, it is also necessary to examine the changes in permeability due to the soil improvement.

(c) In case that the foundation is made of rocks, careful investigations and consideration of permeability should be required, because the rock foundation may contain cracks or fissures which govern the rate of seepage [16]

12.2 Revetments with Amenity Function

Ministerial Ordinance

Performance Requirements for Revetments

Article 20

2 In addition to the provisions of the preceding paragraph, the performance requirements for revetments to be utilized by an unspecified large number of people shall satisfy the requirements specified by the Minister of Land, Infrastructure, Transport and Tourism so as to secure the safety of the users of the revetment concerned.

Public Notice

Performance Criteria of Revetments

Article 43

2 In addition to the provisions of the preceding paragraph, the performance criteria for the revetments which are utilized by an unspecified large number of people shall be such that the revetments have the dimensions necessary to secure the safety of users in consideration of the environmental conditions to which the facilities concerned are subjected, and the utilization conditions, and others.

[Technical Note]

(1) In amenity-oriented revetments, the cross section of the revetment shall be set considering the danger of users falling into the sea, and ancillary facilities such as fences to prevent falling shall be provided appropriately, as necessary.

(2) In facilities where wave overtopping can be expected to reach parts where people normally walk during even high wave conditions, it is necessary to ensure general public knowledge of the danger by appropriate means such as signs.

(3) When facilities are used by elderly persons, and persons with physical disabilities, efforts must be made to enable safe movement of wheelchairs when designing passages on the revetment, the width and gradient of slopes.

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2) Shibata, K., H. Ueda and K. Ohori: Study on the Dimensions of Embankment and Seawall, Technical Note of PHRI No. 448, 1983
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17) Technical Committee for Coastal protection facilities: Technical standards and commentary of coastal protection facilities, Japan Port Association, 2004
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20) Transport Economy Research Center: Guideline of the facilities for elderly and handicapped people in public transport terminal, 1994

13 Coastal Dikes

Ministerial Ordinance

Performance Requirements for Coastal Dikes

Article 21
The provisions of Article 16 shall be applied correspondingly to the performance requirements for coastal dikes.

Public Notice

Performance Criteria of Coastal Dikes

Article 44
The provisions of Article 39 shall be applied to the performance criteria for coastal dikes with modifications as necessary.

References

1) Technical Committee for Coastal Protection Facilities: Technical standards and comment\ary of coastal protection facilities, Japan Port Association, pp. 3-19- 3-60, 2004

14 Groins

Ministerial Ordinance

Performance Requirements for Groins

Article 22

1 The performance requirements for groins shall be as specified by the Minister of Land, Infrastructure, Transport and Tourism for the purpose of mitigating the influence of littoral drift through effective control of sediment transport.

2 The provisions of the item (2) of the paragraph (*1*) of Article 14 shall be applied correspondingly to the performance requirements for groins.

Public Notice

Performance Criteria of Groins

Article 45

The provisions of Article 38 shall be applied to the performance criteria of groins with modifications as necessary.

[Commentary]

(1) Performance Criteria of Groins

① Application with necessary modifications of performance criteria of sediment control groins

(a) Settings in connection with the **Public Notice, Article 38 Performance Criteria for Sediment Control Groins** shall be applied with the necessary modifications to the performance criteria of groins. In the performance verifications of groins, appropriate consideration shall be given to the effect of increased earth pressure due to accumulation of littoral drift, as necessary, and appropriate consideration shall also be given to the effects of waves and river currents.

(b) Control of littoral drift (usability)
In the layout of groins, in addition to the positions where groins are installed, their direction and the mutual spacing between groins shall be considered. In the dimensions, the structure, crown height, crest width, and length shall be considered. In setting the layout and dimensions in the performance verifications of groins, appropriate consideration shall be given to the predominant direction of waves and water currents, topography, expected conditions of use of the objective groin, and the impact on the natural environment, etc. so that the facilities can demonstrate their required function of controlling littoral drift.

(c) Layout (usability)
In the layout of groins, attention shall be paid to the fact that excessive reduction of longshore sediment transport by installation of groins may increase the possibility of shoreline retreat on the surrounding coast.

References

1) Technical Committee for Coastal Protection Facilities: Technical standards and comment\ary of coastal protection facilities, Japan Port Association, pp.3-77-3-85, 2004

15 Parapets

Ministerial Ordinance

Performance Requirements for Parapets

Article 23
The provisions of Article 16 shall be applied correspondingly to the performance requirements for parapets.

Public Notice

Performance Criteria of Parapets

Article 46
The provisions of Article 39 shall be applied to the performance criteria of parapets with modifications as necessary.

[Technical Note]

16 Siltation Prevention Facilities

16.1 General

(1) In cases where siltation of harbors and waterways is expected, the mode of siltation shall be analyzed based on an adequate investigation of the potential causes of siltation, and appropriate countermeasures shall be taken, considering the various types of effects caused by siltation prevention works, safe navigation of ships and economy.

(2) Causes of Siltation
Causes of siltation are listed below.

① Invasion and accumulation of littoral drift mainly caused by waves or that caused by currents

② Settling and accumulation of river erosion sediments

③ Deposition of wind blown sand

④ Movement of sediments within the objective area and change in location of deposition

⑤ Movement of sediments due to disturbances in the harbor, collapse of slopes in waterways, and formation of sand waves.

16.2 Facilities for Trapping Littoral Drift and River Erosion Sediment

(1) When it is aimed to prevent shoaling due to littoral drift by means of maintenance dredging, an appropriate facility to trap the sediment should be built at a proper location, at which the facility can prevent sediment from invading to waterways or basins. The facility should be able to reduce the wave actions around it and increase the dredging efficiency. The type and layout of these sand trap facilities is preferable to be determined by taking into consideration their capability to trap the sediment, the dredging conditions, and the construction and operational costs, based on adequate investigations and researches.

(2) Facilities to Trap the Sediment Transport
As the method to trap the sediment, provisions to limit sediment deposition area are commonly employed in various countries, by means of building a detached breakwater or partially reducing the crown height of updrift breakwater. There are also sediment traps such as pocket dredging executed in the waterways crossing a large sand bar in the sea floor of straits, which is gradually restored by natural process after dredging. Pocket dredging is also done on the river bed, where shoaling occurs by river discharged sediment.

(3) Proper Positioning of Sediment Trap
The sediment traps may be installed in areas where deposition occurs easily under natural conditions, as shown in **Fig. 16.2.1(a)**, **(b)**, and **(c)**, or artificial conditions may be created to encourage sediments to settle out of flows with a high concentration of littoral drift, as shown in **Fig. 16.2.1(d)**, **(e)**, and **(f)**. To identify suitable locations of this type and capture littoral drift in the most efficient manner, an adequate understanding of the condition and mechanism of sediment transport is indispensable. Furthermore, in selecting the positions for sediment traps, in addition to sediment trapping efficiency, in cases where the trapped sediments will be dredged, it is preferable to give adequate consideration to dredging conditions, in other words, to easily maintaining the water depth necessary for navigation of dredgers and calm conditions during navigation and work.

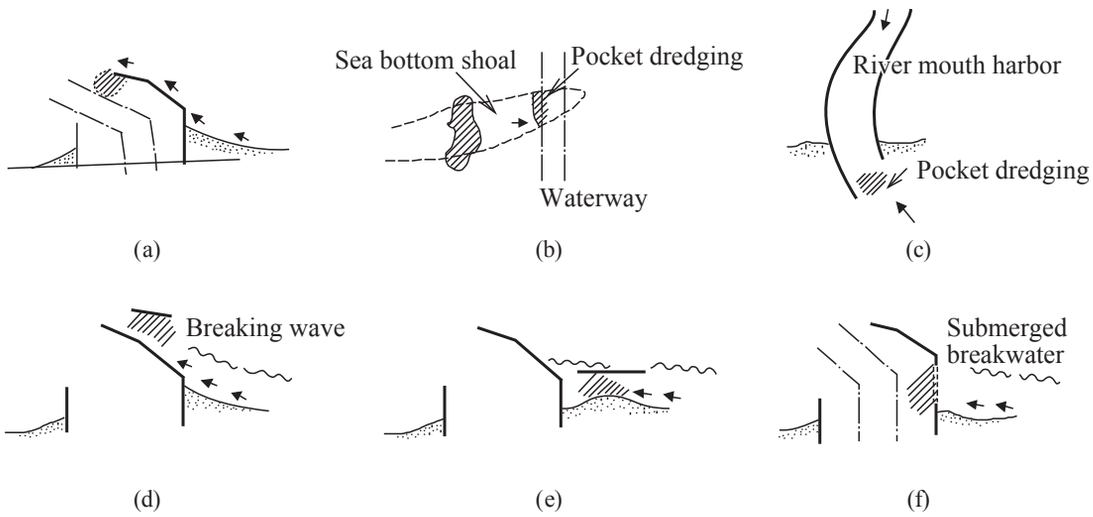

Fig. 16.2.1 Positioning of Sediment Traps

16.3 Wind Blown Sand Prevention Work

16.3.1 General

Wind-blown sand, i.e., sand that is moved by winds, is carried into harbors or waterways where it settles and deposits, and cause shoaling there. In some cases it also accumulates on road surfaces and is dispersed into residential areas, disrupting the daily living of the resident. In particular, there are many instances that open digging of dune or land reclamation cause problems related to wind-blown sand, and thorough countermeasures must be prepared in advance.

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
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
3) JSCE, Civil Engineering Handbook, Vol. II, pp. 2135-2136, 1989