[2] Wave Overtopping Quantity

(1) For structures for which the wave overtopping quantity is an important performance verification factor, the wave overtopping quantity must be calculated by carrying out hydraulic model tests or by using data from hydraulic model tests carried out in the past. In this case, wave irregularity need be considered. If the sea bottom topography is complex, the planar distribution of the wave overtopping rate may be determined by calculating the wave acting on the seawall via wave transformation calculations and then by using the overtopping rate equation.[73)]

(2) The "wave overtopping quantity" is the total volume of overtopped water. The "wave overtopping rate", on the other hand, is the average volume of water overtopping in a unit time; it is obtained by dividing the wave overtopping quantity by the time duration of measurement. The wave overtopping quantity and the wave overtopping rate are generally expressed per unit width.

(3) If the wave overtopping quantity is large, then not only there will be damage to the seawall body itself, but also damage by flooding to the roads, houses and/or port facilities behind the levee or seawall, despite that the levee or seawall is intended to protect them. There is further a risk to users of water frontage amenity facilities that they may be drowned or injured. When verifying performance, it is necessary to set the wave overtopping quantity so that it is equal to or less than the limit value that has been determined in line with the characteristics of structures and the situation with regard to their usage. Furthermore, when estimating the wave overtopping quantity by means of experiments, it is preferable to consider changes in tidal water level, i.e., to carry out experiments for different water levels.

(4) Design Diagram of wave overtopping rate [74)]
For an upright or wave-dissipating type seawall that has a simple form (i.e., that does not have anything like a toe protection mound or a crown parapet), the wave overtopping rate may be estimated using **Figs. 4.3.20** to **4.3.23**. These diagrams have been drawn up based on experiments employing random waves. From the results of a comparison between the experiments and field observations, it is thought that the accuracy of the curves giving the wave overtopping rate is within the range listed in **Table 4.3.4**. The wave overtopping rate for the wave-dissipating type seawall has been obtained under the condition that the upper armor layer at the crown consists of 2 rows of wave-dissipating concrete blocks.

Table 4.3.4 Estimated Range for the Actual Wave Overtopping Rate relative to the Estimated Value

$q\Big/\sqrt{2g\left(H_0'\right)^3}$	Upright seawall	Wave-dissipating type seawall
10^{-2}	0.7–1.5 times	0.5–2 times
10^{-3}	0.4–2 times	0.2–3 times
10^{-4}	0.2–3 times	0.1–5 times
10^{-5}	0.1–5 times	0.05–10 times

Note that when obtaining rough estimates for the wave overtopping rate for random waves using **Figs. 4.3.20** to **4.3.23**, the following should be considered:

① If the actual values of the bottom slope and the deepwater wave steepness do not match any of the values on the diagram, the diagram for which the values most closely match should be used, or alternatively interpolation should be carried out.

② The wave-dissipating concrete blocks in the figures are made up of two layers of tetrapods (upper armor layer at the crown consists of 2 rows). Therefore, even if the same kind of wave-dissipating concrete block is used, if there are differences in the crown width, in the way, or in the form of the toe, then there is a risk that the actual wave overtopping rate may considerably differ from the value obtained by the diagrams.

③ If the number of rows of concrete blocks at the crown is increased, the wave overtopping quantity tends to decrease.[75)]

④ When there are difficulties in applying the diagrams for estimating the wave overtopping rate, the approximate equation of Takayama et al.[76)] may be used.

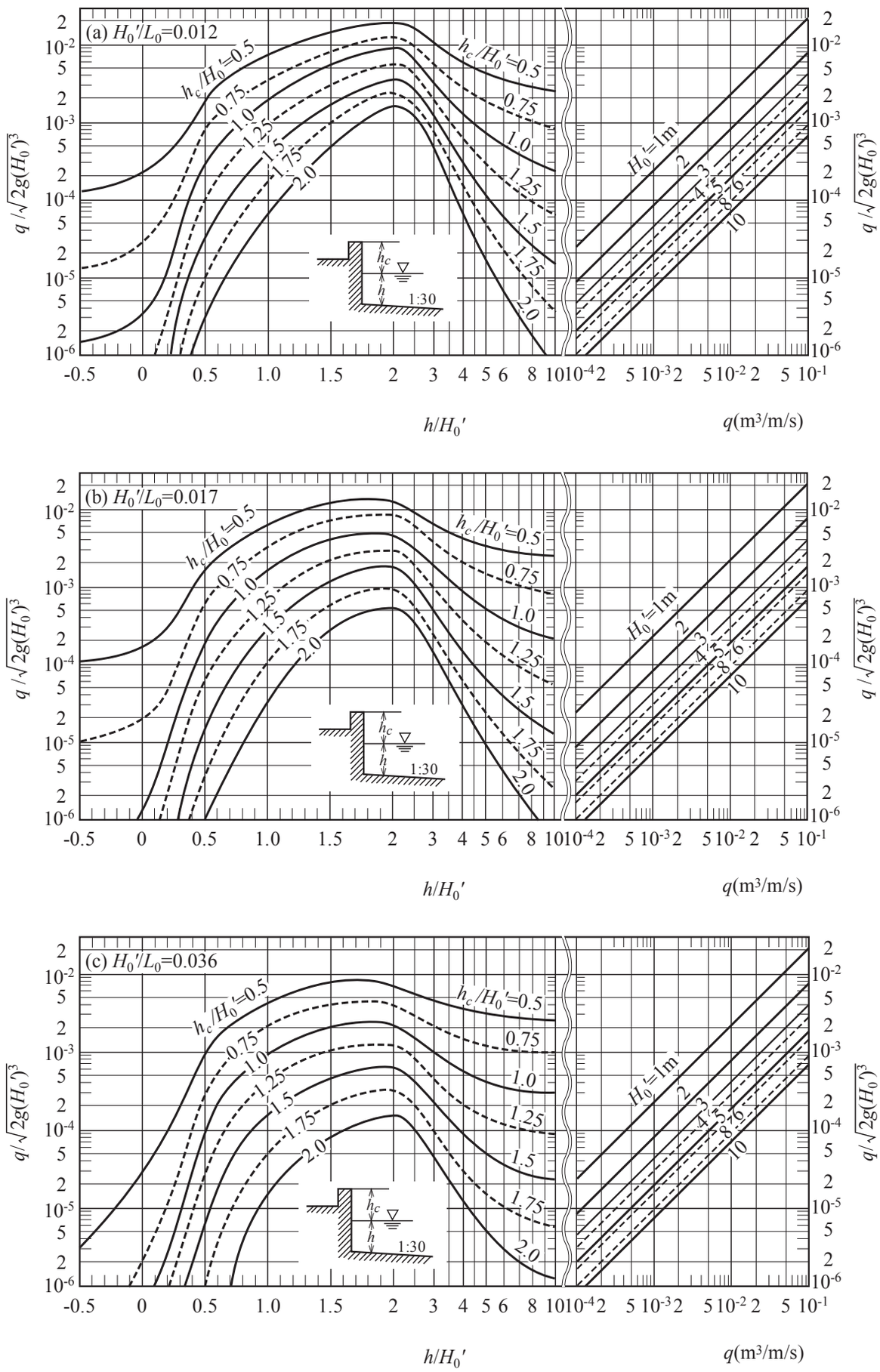


Fig. 4.3.20 Diagrams for Estimating Wave Overtopping Rate for Upright Seawall (Bottom Slope 1/30)

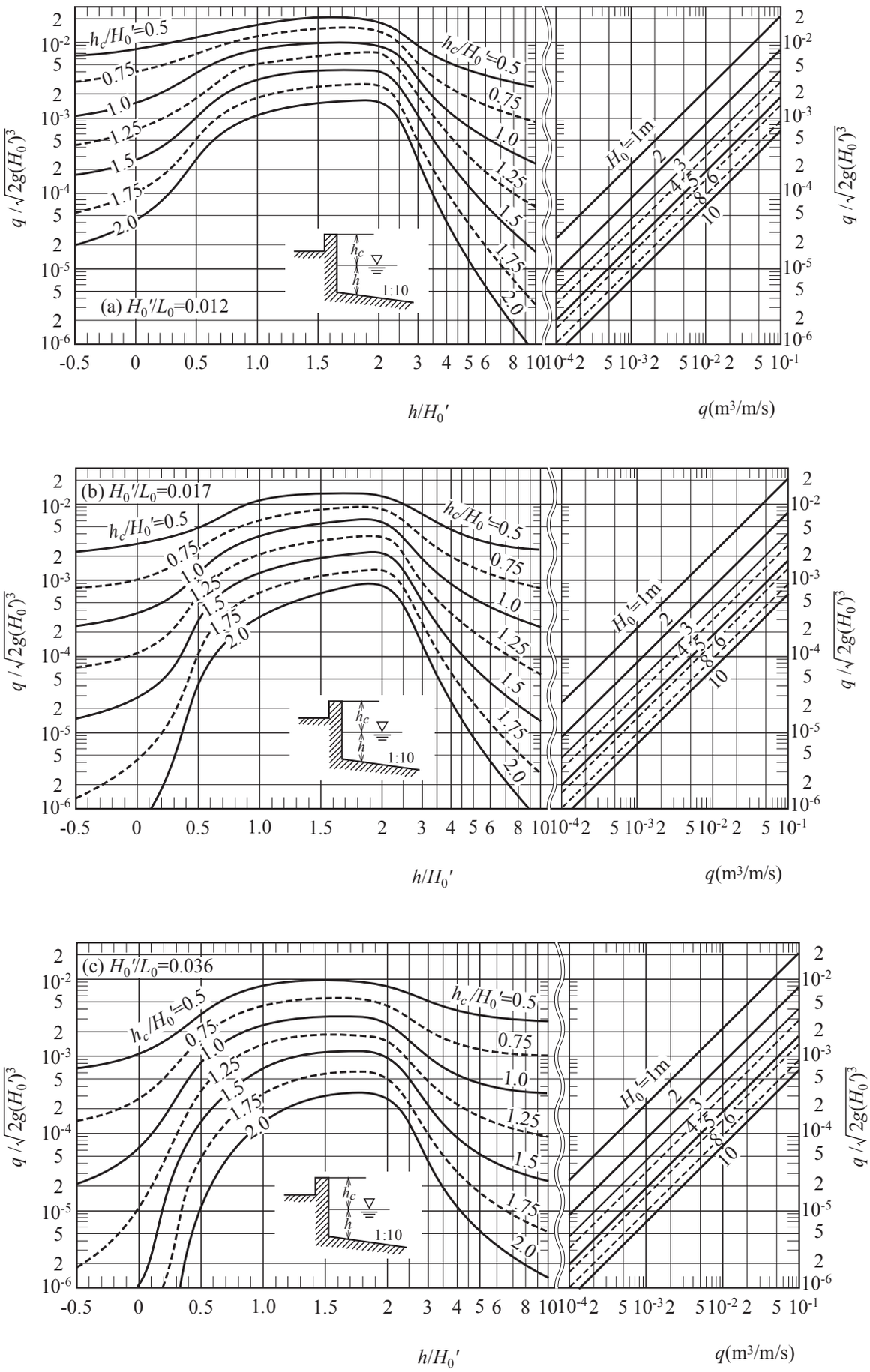


Fig. 4.3.21 Diagrams for Estimating Wave Overtopping Rate for Upright Seawall
(Bottom Slope 1/10)

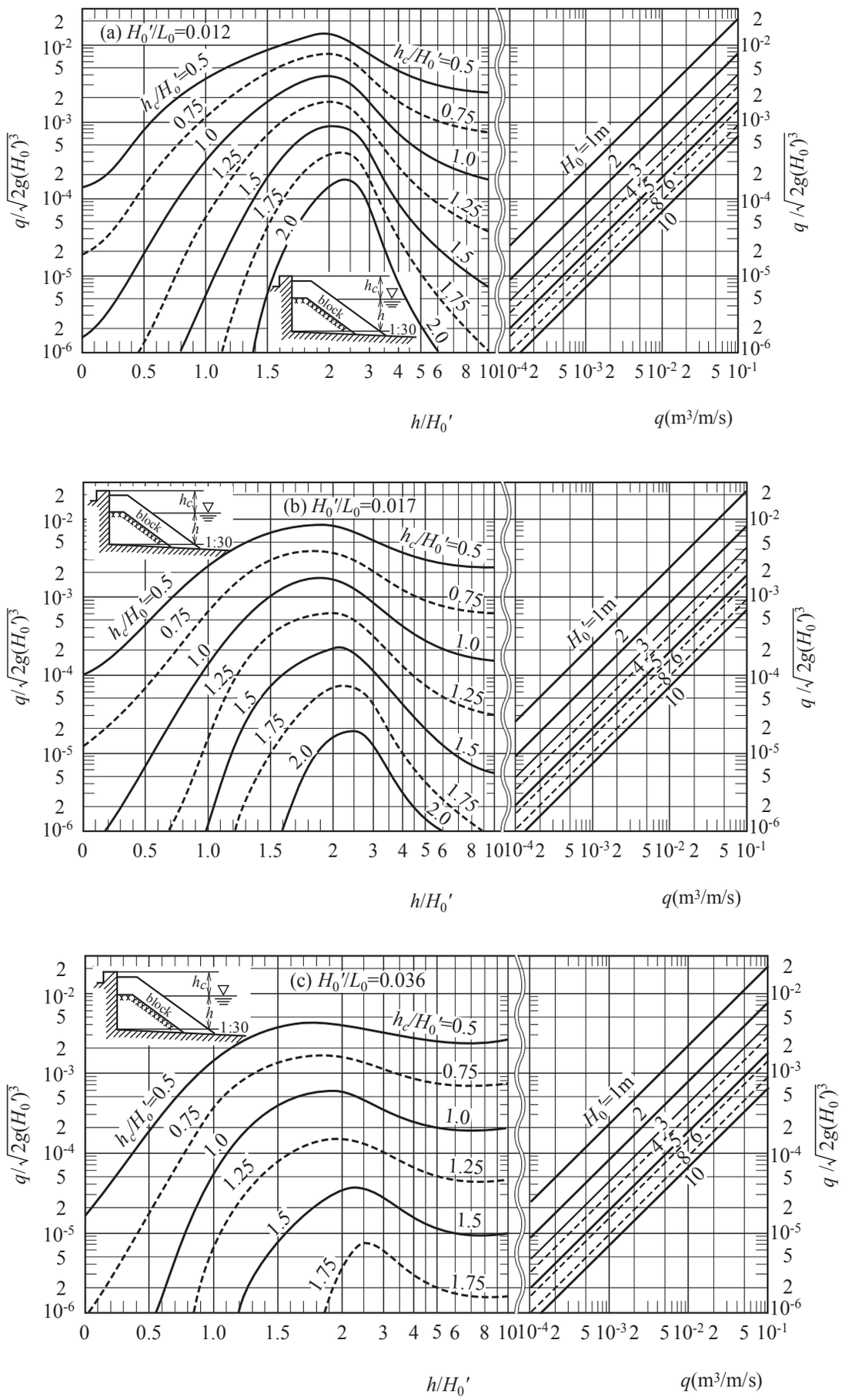


Fig. 4.3.22 Diagrams for Estimating Wave Overtopping Rate for Wave-dissipating Type Seawall (Bottom Slope 1/30)

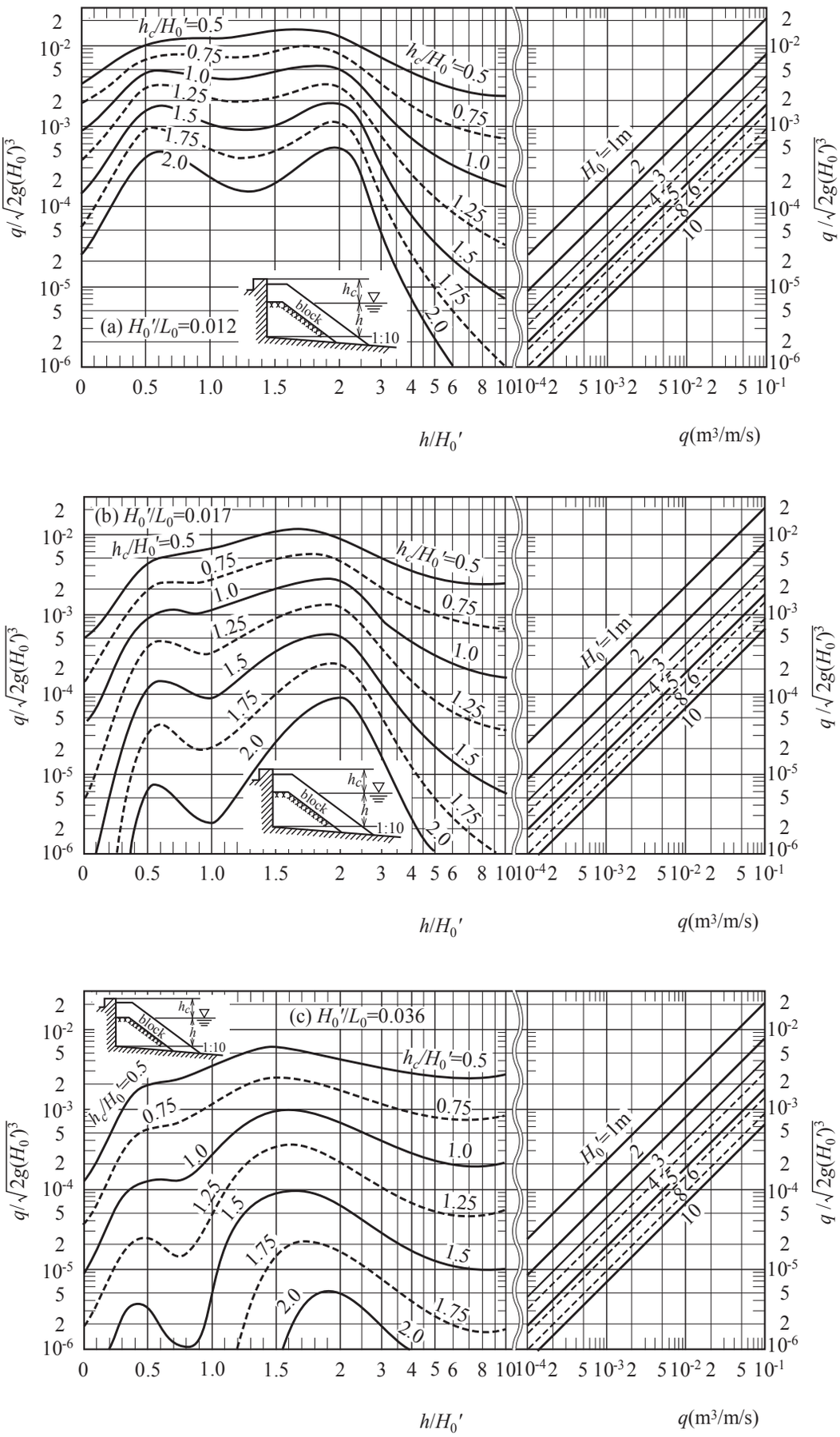


Fig. 4.3.23 Diagrams for Estimating Wave Overtopping Rate for Wave-dissipating Type Seawall (Bottom Slope 1/10)

(5) Allowable Wave Overtopping Rate

The allowable wave overtopping rate depends on factors such as the structural type of the seawall, the situation with regard to land usage behind the seawall, and the capacity of drainage facilities; it needs to be set appropriately depend on the situations. Although it is thus impossible to give one standard value for the permissible wave of overtopping rate, Goda [77] et al. nevertheless gave the values for the threshold rate of wave overtopping for inducing of damage as shown in **Table 4.3.5** based on the past cases of disasters. Also, Fukuda et al.[78] gives the values shown in **Table 4.3.6** as values for allowable wave overtopping rate in view of the land usage behind the seawall. Furthermore, Nagai et al.[79] have considered the degree of importance of the facilities behind the seawall and have come up with the values for the allowable wave overtopping rate as shown in **Table 4.3.7**, using the results of experiments with regular waves. Suzuki et al.[63] have proposed 0.01m^3/m/s as the allowable wave overtopping rate for amenity-oriented revetment When conducting the performance verification, these must be set appropriately by considering the importance of the facilities and the capacity of drainage facilities.

CADMAS-SURF [80] or flooding analysis models such as those that use the MARS method [81] can be used when calculating wave overtopping precisely with the inclusion of items such as the permeability of the soil behind the seawall and the characteristics of wave-dissipating work configurations.

Table 4.3.5 Threshold Rate of Wave Overtopping for Inducing of Damage

Type	Armor Layer	Wave Overtopping Rate (m^3/m/s)
Seawall	Paved behind Not paved behind	0.2 0.05
Levee	Covered with concrete on 3 sides Crown paving/rear slope non constructed Crown not paved	0.05 0.02 0.005 or less

Table 4.3.6 Allowable Wave Overtopping Rate in view of State of Land Use

User	Distance from dike	Wave overtopping rate (m^3/m/s)
Pedestrian	Land right in back (50% degree of safety)	2×10^{-4}
	Land right in back (90% degree of safety)	3×10^{-5}
Automobile	Land right in back (50% degree of safety)	2×10^{-5}
	Land right in back (90% degree of safety)	1×10^{-6}
House	Land right in back (50% degree of safety)	7×10^{-5}
	Land right in back (90% degree of safety)	1×10^{-6}

Table 4.3.6 is a table created with the results where people who watch a wave overtopping observation video make a judgment, and indicates a wave overtopping rate that at least that percentage of people judged to be safe.

Table 4.3.7 Permissible Wave of Overtopping Rate in view of Degree of Importance of Hinterland (m^3/m/s)

Districts where significant damage is expected particularly by the invasion of wave overtopping and spray due to a dense concentration of residential houses and public facilities in the rear.	Around 0.01
Other important districts	Around 0.02
Other districts	0.02 – 0.06

(6) Equivalent Crown Height Coefficient

The equivalent crown height coefficient can be used as a guideline when setting the wave overtopping rate for a seawall upon which wave-dissipating concrete blocks are laid or for a wave-dissipating type seawall with vertical slits. The equivalent crown height coefficient is the ratio of the height of the seawall in question to the height of an imaginary upright seawall that results in the same wave overtopping quantity, where the conditions in terms of waves and the sea bottom topography are taken to be the same for the both cases. If the equivalent crown height coefficient is less than 1.0, this means that the crown of the seawall under study can be lowered below that of an

upright seawall and still give the same wave overtopping quantity; in other words, the seawall under study has a form that is effective in reducing the wave overtopping rate. Below are the reference values for the equivalent crown height coefficient β for typical types of seawall.

Wave-dissipating block type seawall [76] : $\beta = 0.9 – 0.7$
Vertical-slit type seawall [76] : $\beta = 0.6$
Parapet retreating type seawall [75] : $\beta = 1.0 – 0.5$
Stepped seawall [75] : $\beta = 1.7 – 1.0$
When the waves are obliquely incident [82), 83)] :

$$\beta = \begin{cases} 1-\sin^2\theta & |\theta| \le 30^\circ \\ 1-\sin^2 30^\circ = 0.75 & |\theta| > 30^\circ \end{cases}$$

(θ is the angle of incidence of the waves; it is 0º when the waves are incident perpendicular to the seawall faceline)

(7) Effect of Winds on the Wave Overtopping Quantity

In general, winds have a relatively large effect on wave overtopping quantity when it is small, although there is a lot of variation. However, the relative effect of winds decreases as the wave overtopping rate increases. **Fig. 4.3.24** shows the results of an investigation on the wind effect on the wave overtopping quantity based on field observations. The abscissa shows the spatial gradient of the horizontal distribution of the wave overtopping quantity, while the ordinate shows the wave overtopping quantity per unit area. As can be seen from the figure, when the wave overtopping quantity is small, the larger the wind velocity, the smaller the spatial gradient of the horizontal distribution of the wave overtopping quantity becomes. When the wave overtopping quantity is large, the spatial gradient of the horizontal distribution of the wave overtopping quantity increases. This shows that, when the wave overtopping quantity is small, the distance over which a mass of water splash strongly affected by the wind velocity, with a larger distance at a higher wind velocity; however, when the wave overtopping quantity is large, the difference in the splash distance becomes small.

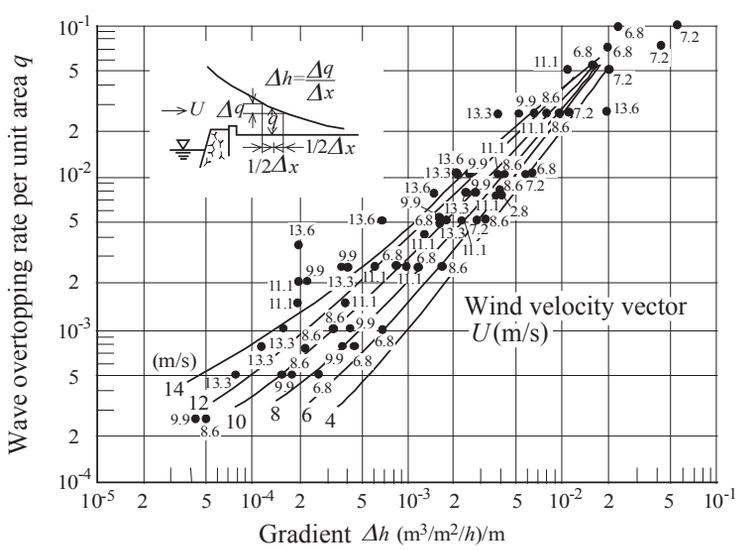


Fig. 4.3.24 Wind Effect on Spatial Gradient of Horizontal Distribution of Wave Overtopping Quantity [78)]

(8) Wave Overtopping of Multi Directional Random Waves

In waters where the multi directionality of waves is well clarified, the wave overtopping rate may be corrected in accordance with S_{max} as in reference 83).

(9) Effects of Parapet

Parapet on a revetment is effective in reducing wave overtopping. Reference 84) can be referred for wave overtopping of sloping dikes with parapet.

[3] Transmitted Waves

(1) It shall be standard to calculate the height of waves transmitted behind a breakwater by overtopping and/or permeation through the breakwater or the foundation mound of breakwater by referring to either the results of hydraulic model tests or the past data.

(2) It is necessary to appropriately estimate the transmitted wave height after waves have overtopped and/or passed through a breakwater, because the transmitted waves affect the wave height distribution behind the breakwater. Transmitted waves include waves that have overtopped and/or overflowed, as well as waves that have penetrated through a sloping breakwater or a foundation mound of composite breakwater. The latter in particular is sometimes referred to as penetrated waves. Recently, several breakwaters have been built with caissons, which are originally not permeable, having through-holes in order to enhance the exchange of the seawater in a harbor. In this case, it is necessary to examine on the wave coefficient of wave transmission, because the coefficient serves as an indicator of the efficiency of the exchange of seawater.

(3) Coefficient of Wave Transmission for Composite Breakwater
Fig. 4.3.25 may be used to calculate the height of waves that are transmitted into a harbor when they overtop a composite breakwater or permeate through a foundation mound. Even when the waves are random, the coefficient of wave transmission agrees pretty well with that shown in **Fig. 4.3.25**. It has also been shown that **Fig. 4.3.25** is valid not only for the significant wave height, but also for the highest one-tenth wave height and the mean wave height.[85)]

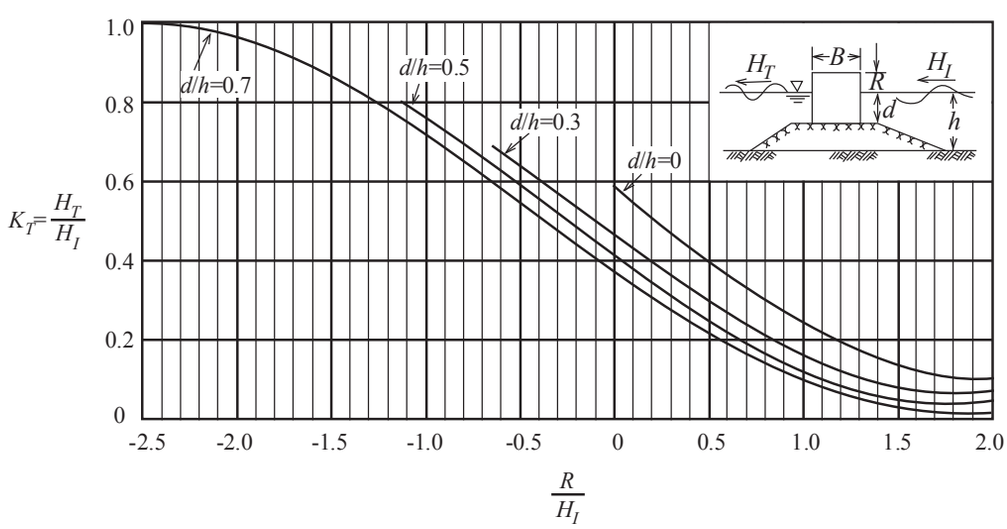


Fig. 4.3.25 Diagram for Calculating Coefficient of Wave Transmission [86)]

(4) Period of Transmitted Waves of Composite Breakwater
The period of the transmitted waves drops to about 50 to 80% of the corresponding incident wave period in both the significant wave and the mean wave.[86)]

(5) For composite breakwaters covered with wave-dissipating concrete blocks, sloping breakwaters covered with wave-dissipating concrete blocks, and other such breakwaters, experiments on the transmitted wave height have been carried out by the Civil Engineering Research Institute of Hokkaido Development Bureau.[87), 88)]

(6) Coefficient of Wave Transmission of Structures

① For a porous and permeable structure such as a sloping breakwater or a wave-dissipating concrete block type breakwater, Kondo's [53)] theoretical analysis may be referred to. The following empirical equation may be used to obtain the coefficient of wave transmission of a typical structure.

Stone breakwater [89)]:

$$K_T = 1 \Big/ \left(1 + k_t \sqrt{H/L}\right)^2 \tag{4.3.31}$$

where

k_t : $k_t = 1.26\left(\frac{B}{d_t}\right)^{0.67}$ in the case of sloping breakwater and $k_t = 1.184\left(\frac{B}{d_t}\right)^{0.895}$ in the case of deformed block type breakwater

B : crown width of the structure

d_t : Nominal diameter of rubble or height of deformed block

H : height of transmitted waves

L : wavelength of transmitted waves

② For a curtain wall type breakwater, the empirical solutions of Morihira et al.[90] may be used.

③ For the coefficient of wave transmission of an upright breakwater of permeable type that has slits in both the front and rear walls, the experimental results [52] are available.

④ Types of breakwater aiming to promote the exchange of seawater include multiple-wing type permeable breakwaters, multiple vertical-cylinder breakwaters, horizontal-plate type permeable breakwaters, and pipe type permeable breakwaters. The coefficient of wave transmissions of these types of breakwater are obtained.[52]

(7) Coefficient of Wave Transmission of Submerged Breakwater

A submerged breakwater is usually made by piling up natural stones or crushed rock to form a mound, and then covering the surface with concrete blocks to prevent underlayers from sucking out. For a submerged breakwater of crushed rock, a diagram [52] showing the relationship between the crown height of the breakwater and the coefficient of wave transmission is available.

4.3.8 Rise of Mean Water Level due to Waves and Surf Beats

[1] Wave Setup

(1) When constructing structures within the breaker zone, it is preferable to consider the phenomenon of wave setup as necessary, which occurs in the breaker zone owing to wave breaking as they approach the coast.

(2) Rise of the mean water level due to breaking waves

The phenomenon where the mean water level near the shoreline rises due to breaking waves, so-called "wave set up", was known long ago through observations at the seashore and so on, but theoretical proof about the causes for the occurrence of this phenomenon has been lacking. In 1962, Longuet-Higgins and Stewart [91] indicated that when a series of waves whose wave height varies approach the shore, this becomes the conveyance of a large momentum at the places where the wave height is large, and it becomes smaller at the places where the wave height is small, so apparent stress ends up being generated, and the mean water level changes. This apparent stress was termed the radiation stress. This radiation stress is an amount proportionate to the square of the wave height, and in that sense is an amount of the same order as the energy of a wave.

(3) Radiation Stress

With the introduction of the concept of radiation stress is introduced, the change in the mean water level can be explained as follows.

When a wave approaching from offshore reaches shallow waters, the wave height increases due to shallow water deformation as the water depth becomes more shallow. When the wave height becomes larger, the conveyance of momentum becomes larger, and the mean water level begins to decline (wave set down). When the wave approaches the place where it is even shallower, it breaks due to the wave height corresponding to the sea bottom slope and water depth, and suddenly the wave height is diminished, and the sudden decline of this wave height causes the conveyance of momentum to decrease suddenly, and the mean water level rises. The rise in the mean water level in the vicinity of the shoreline is viewed as a typical example of a phenomenon caused by such radiation stress.

(4) Diagrams for Estimating the Amount of Wave Setup

The changes in the mean water level by random wave breaking on the bottom slopes of 1/100 and 1/10 as calculated by Goda [47] are shown in **Figs. 4.3.26** and **4.3.27**. The smaller the wave steepness (H_0'/L_0) the faster and larger the rise of mean water level becomes. **Fig. 4.3.28** shows the rise of mean water level at the shoreline. The smaller the wave steepness, and the steeper the bottom slope, the larger the rise of mean water level becomes. When H_0'/L_0 is in the range 0.01-0.05, with the exception of very steep bottom slope, the rise of mean water level near the shoreline is of the order (0.1-0.15)H_0', where H_0' is the equivalent deepwater wave height and L_0 is the wavelength of the deepwater wave. **Fig. 4.3.29** is a diagram for estimating the amount of wave setup that has newly been proposed taking the directional wave spectrum into account. The values are slightly smaller compared to **Fig. 4.3.28** in ranges where the wave steepness is small.[92]

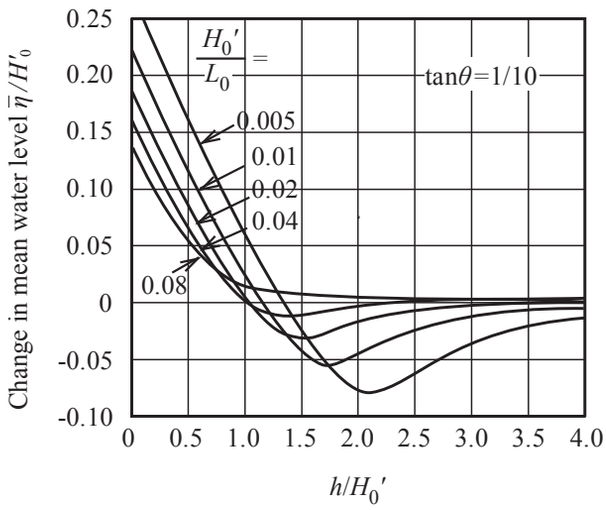


Fig. 4.3.26 Change in Mean Water Level (Bottom Slope 1/10)

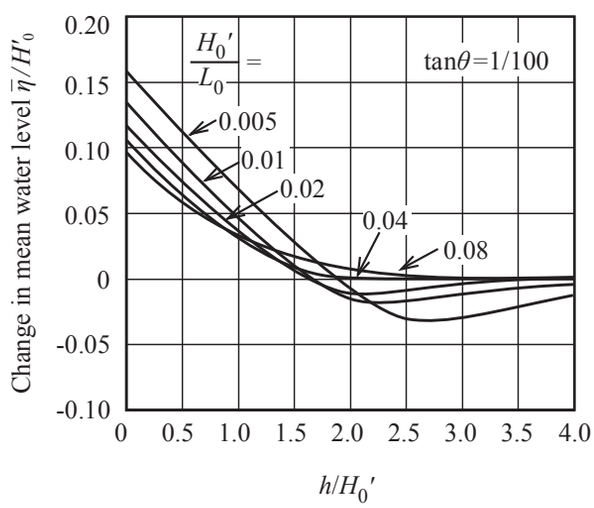


Fig. 4.3.27 Change in Mean Water Level (Bottom Slope 1/100)

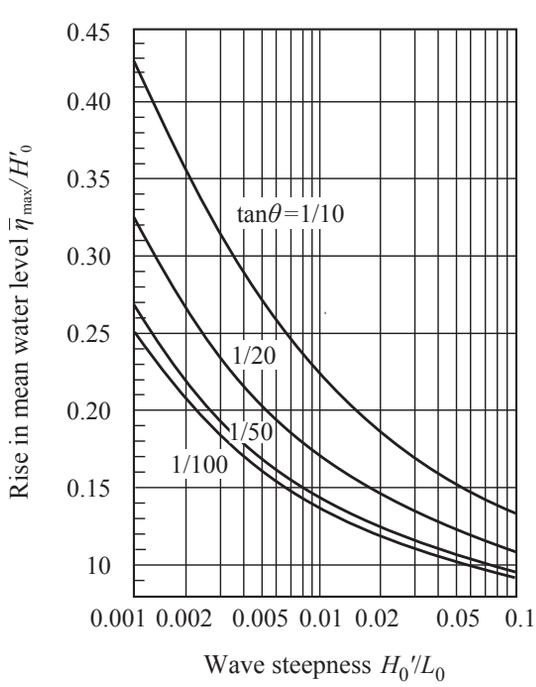


Fig. 4.3.28 Rise in Mean Water Level at Shoreline

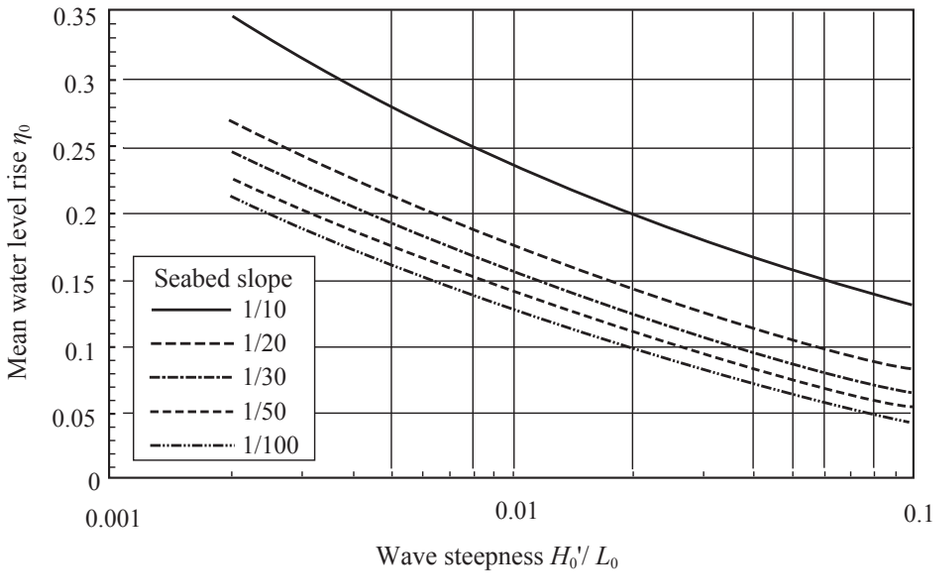


Fig. 4.3.29 Figure Diagram of Water Level Rise at Shoreline considering the Multi Directionality of Waves

(5) Consideration of the rise in mean water level in the performance verification
Since the wave breaking point varies, and the breaking wave height becomes larger, owing to the rise of the mean water level, it is important to consider the rise in mean water level in order to carry out accurate computation of the design wave height in shallow waters.

[2] Surf Beats

(1) Surf beat with a period of one to several minutes, which occurs along with wave deformation in shallow waters, is examined, as necessary.

(2) Random wave height fluctuations lasting one to several minutes in the vicinity of the shoreline are called surf beat, and this has a major effect on the runup height of waves, wave overtopping and stability of beaches at the beach. It is preferred that the size of the surf beat is estimated as appropriate by either Goda's approximation formulas [58] or on-site observations.

(3) Goda's Formulas for Estimating Surf Beat Amplitude
Based on the results of field observations of surf beat, Goda [58] has proposed the following relationship:

$$\zeta_{rms} = \frac{0.04(\eta_{rms})_0}{\sqrt{\frac{H_0'}{L_0}\left(1+\frac{h}{H_0'}\right)}} = \frac{0.01H_0'}{\sqrt{\frac{H_0'}{L_0}\left(1+\frac{h}{H_0'}\right)}} \tag{4.3.32}$$

where

ζ_{rms} : root mean square amplitude of the surf beat wave profile
$(\eta_{rms})_0$: root mean square amplitude of the deepwater wave profile
H_0' : equivalent deepwater wave height
L_0 : wavelength in deepwater
h : water depth

This equation shows that the amplitude of the surf beat is proportional to the deepwater wave height, that it falls as the water depth increases, and that it increases as the deepwater wave steepness (H_0'/L_0) decreases.

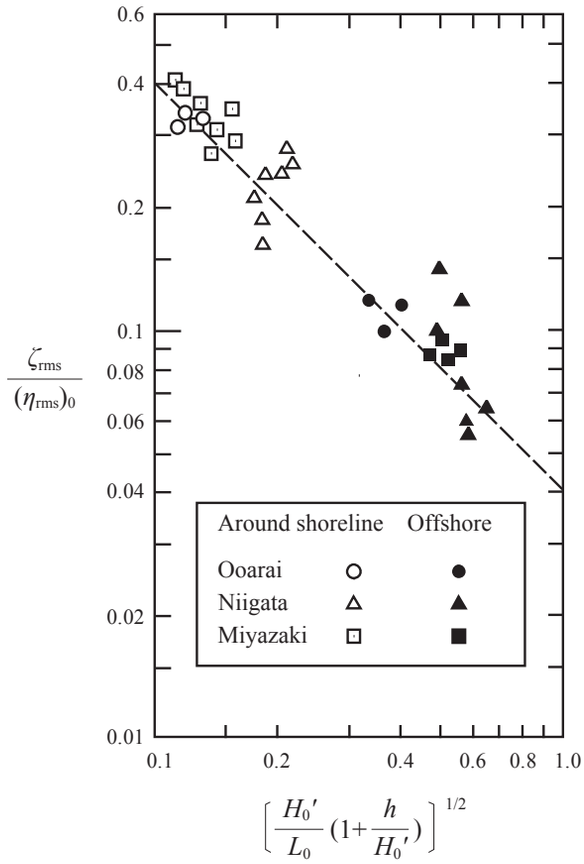


Fig. 4.3.30 Ratio of Surf Beat Amplitude for Deepwater Wave

4.4 Long-period Waves

(1) With regard to long-period waves and seiche in harbors, field observations should be carried out as far as possible, and appropriate measures to control them must be taken based on the results of these observations. Here, long-period waves are defined as waves composed of component waves with periods between 30 seconds to 300 seconds included in the frequency spectrum analyzed from an uninterrupted observation record taken over a period of 20 minutes or more.

(2) Water level fluctuations with the period between 30 seconds and several minutes sometimes appear at observation points in harbors and off the shore. Such fluctuations are called long-period waves. If the period of such long-period waves is close to the natural frequency period of the vibration system made up of a ship and its mooring ropes, the phenomenon of resonance can give rise to a large surge motion even if the wave height is small, resulting in large effects on the cargo handling efficiency of the port. If it is clear from observations that long-period waves of significant wave height 10 - 15 cm or more frequently arise in a harbor, it is advisable to investigate either hard or soft countermeasures.[93)]

When conspicuous water level fluctuations within the period several minutes or longer occur at an observation point in a harbor, it is highly likely that the phenomenon of "seiche" is taking place. This phenomenon occurs when small disturbances in water level generated by changes in air pressure out at sea are amplified by the natural frequency of the harbor or bay. If the amplitude of such seiche becomes significantly large, inundation at the head of the bay or reverse outflow from municipal drainage channels may occur. Also high current velocities may occur locally in a harbor, resulting in breaking of the mooring ropes of small ships. When drawing up a harbor plan, it is thus preferable to give consideration to making the shape of the harbor to minimize the seiche motion as much as possible. At marinas and other small ports, the natural frequency of the port may be close to the frequency of long-period waves and the propagation of long-period waves from the open sea may excite the seiche in the port. The two aspects are therefore highly correlated. If seiche excitation by long-period waves becomes apparent from observations or numerical calculations, it is preferable to deliberate countermeasures while giving thought to these aspects.

(3) Critical Wave Height for Cargo Handling Works Effected by Long-period Waves
It is necessary to give due consideration to the fact that long-period waves in front of a quaywall can induce ship surging with the amplitude of several meters through resonance. The critical wave height for smooth cargo

handling works effected by long-period waves depends on the factors such as the wave period, the dimensions of the ship in question, the mooring situation, and the loading conditions. Nevertheless, according to field observations carried out in Tomakomai Bay, [96] it corresponds to a significant wave height of about 10 - 15 cm.

(4) Calculating Propagation of Long-period Waves
It is preferable to calculate the propagation of long-period waves into a harbor by setting up incident wave boundary out at sea and then using either the Boussinesq equation [97] or a calculation method that uses long linear wave equations.[98]

(5) Standard Spectrum for Long-period Waves
When there are insufficient field observation data of long-period waves out at sea and the long-period waves conditions that determine the external forces are not established, the standard spectrum shown in reference 99) or its approximate expression may be used for the long-period waves performance verifications.

Fig. 4.4.1 shows a comparison between an observed spectrum and an approximate form of the standard spectrum. The term α_ℓ in the diagram is a parameter that represents the energy level of the long-period waves. This shows the relationship between the spectrum peak frequency of short-period wave components and boundary frequency f_b for calculating the energy of long-period waves components. From the past observations, the value is between 1.6 and 1.7. The smaller the value of α_ℓ, the larger the energy of the long-period waves becomes.

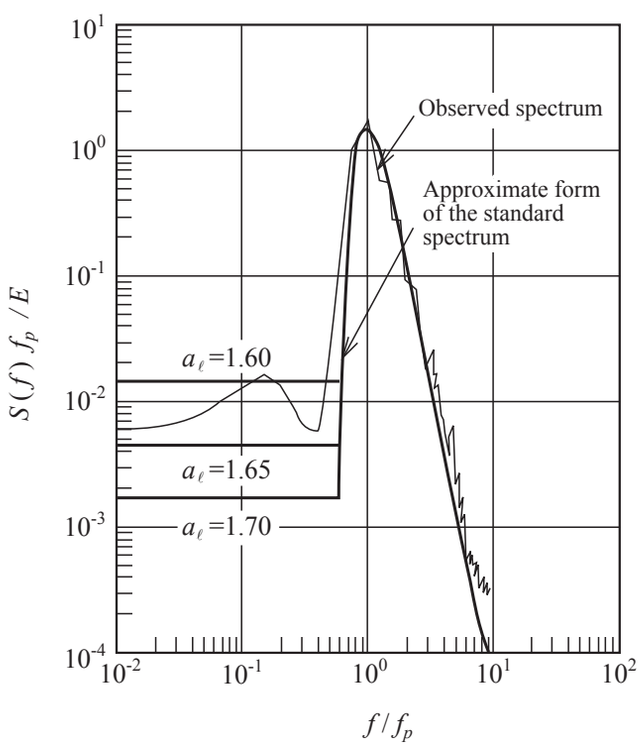


Fig. 4.4.1 Comparison between Standard Spectrum with Long-period Components and Observed Spectrum

(6) Wave Direction of Long-period Waves
In the event of long-period waves are propagated, there are many cases where they overlap with waves reflected from the longshore, and it is difficult to determine the wave direction. However, the energy of the principal long-period waves may propagete according with the principal wave direction of a short-period wave (wind wave).

(7) Method for Calculating Harbor Resonance
See **3.3 Harbor Resonance** for the method for calculating harbor resonance.

(8) Countermeasures for Long-period Waves and Harbor Resonance
In waters where long-period waves are marked, it is preferable to establish a breakwater layout plan in order to inhibit the ingress of long-period waves into the harbor. At this time, in the event that the particle diameter of the mound materials is large, almost all of the energy of the long-period waves is transmitted into the harbor, so it is preferable to undertake appropriate examination on the formation of the breakwater and mound structure.

In order to control the surge oscillation of ships, it is preferable to shift the natural period of the mooring system from the period of the invading long-period waves. To this end, such measures as changing the place where the mooring facilities are installed and the initial tension, as well as an increase in the number of mooring lines using improve the rope materials and new installation of land winches, are desirable, but the results should

be examined beforehand and appropriate measures should be devised based on suitable numerical calculations.

There are many instances where long-period waves are reflected and amplified by the facilities in a harbor, and in particular upright wave-dissipating revetments have almost no wave-dissipating function for long-period waves and swells, so it is necessary to revise the reflection coefficient of the facilities in estimation of the long-period waves height inside harbors. The Environment Assessment Manual of Long-Period Waves in Harbors [100)] can be used as a reference for the rough order of magnitude and calculation method of the reflection coefficient. Long-period wave-dissipating revetments that employ backfilling materials of a two-sided slit caisson wall that has a slit wall on both sides and a gravel material with a large particle diameter allow waves to pass through and dampen long-period waves have been developed as an engineering method that lowers the long-period waves height inside harbors.

The width of the transmitting layer with the gravel material is preferably 50 to 100 meters. It is preferable to set the width of the water transmitting layer, place of installation and installation range so that the maximum effects are achieved in the hydraulic model test and numerical calculations. Since the distribution of long-period waves is not uniform within the harbor, it is preferable to examine as well modifications of the berth location at the planning stage in the event that it is clear that the long-period waves in the target berth exceed the limit values.

(9) Distinction between Long-period Waves and Harbor Resonance

In an ordinary harbor, the period of harbor resonance is longer than that of long-period waves by several minutes, and it is possible to distinguish the two from analysis of the oscillation period. However, the period of harbor resonance may become shorter to 2 to 3 minutes in the case of small craft basins and marinas, and this makes the discrimination difficult. In that case, it is preferable to make a judgment as suitable based on the observation results for offshore waters and the circumstances in the surrounding harbor.

4.5 Concept of Harbor Calmness

(1) Factors of Calmness and Disturbance

① When evaluating the harbor calmness, the factors causing disturbances in the harbor need to be set appropriately.

② The problem of harbor calmness is extremely complex. It involves not only physical factors such as waves, winds, ship motions, and the wind- and wave-resistance of working machinery, but also the factors requiring human judgment, such as the easiness of ships entering and leaving of harbor, ship refuge during stormy weather, and the critical conditions of works at sea. The harbor calmness is further related with the economic factors, such as the efficiency of cargo handling works, the operating rate of ships, and the cost of constructing the various facilities required to improve the harbor calmness. The factors that lead to wave disturbances in harbors, which constitute the basis of the criteria for determining the harbor calmness, include the following:

(a) Waves penetrating through the harbor entrance
(b) Transmitted waves into the harbor
(c) Reflected waves
(d) Long-period waves
(e) Harbor seiche

In large harbors, wind waves generated within the harbor may require attention, and the ship waves by larger ships may cause troubles for small ships.

(2) Points to remember when carrying out harbor calmness calculations

It is necessary to bear the following points in mind when carrying out harbor calmness calculations.

① Set the wave height and period frequency distribution at the port entrance.

② In the event that the depth of navigation channel differs markedly from the surrounding water depth, or shoals may exist inside the harbor, or the water depth changes suddenly in the port entrance, consider the water depth change inside the harbor to the extent possible in the calculation of wave height inside the harbor.

③ Introduce the effects of the period as concerns the permissible value of the wave height in the harbor.

④ Consider the future state of the use of ports as concerns the target value for harbor calmness.

(3) Computation of Harbor Calmness

The harbor calmness can be calculated with the temporal probability of the occurrence of a wave height that does not exceed the critical wave height for cargo handling works or the critical wave height for anchoring. The critical wave height for cargo handling works is the wave height of the limit at which the ships moored at the quaywall or dolphin can safely perform cargo handling activities. The critical wave height for anchoring is the wave height at which anchoring in the basin and buoy mooring as well as mooring at the mooring facilities is possible. Here, the temporal probability of the occurrence of a wave height that exceeds the critical wave height for cargo handling

works is called the cargo handling operating rate, and in general the harbor calmness is assessed as the cargo handling operating rate (see **Fig. 4.5.1**).

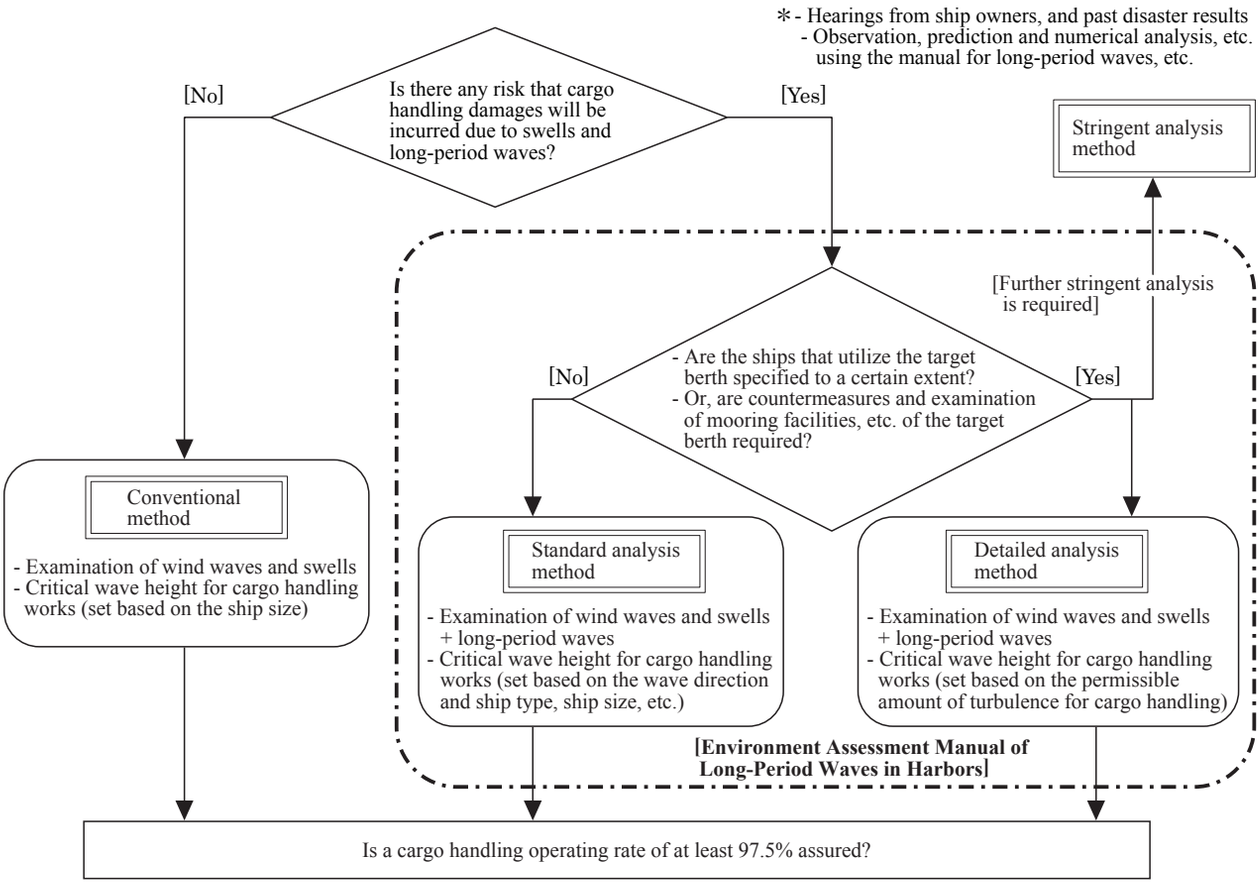


Fig. 4.5.1 Example of Performance Verification Steps Relating to Harbor Calmness

① Harbor calmness for abnormal waves

The harbor calmness for abnormal waves which is equivalent to a probablistic wave with a return period of 50 years, for facilities with a design working life of 50 years in the case of waves as variable action, can be generally assessed by considering the fact that the waves inside the harbor have a major effect on the performance of harbors. The evaluation of the habor calmness can be performed by, setting the critical value of the wave height such that abnormal waves inside the harbor do not cause major damage to the facilities of the harbors, and confirming that the wave height computed by in-harbor wave height calculation does not exceed this critical value.

② Calculation of the cargo handling operating rate for long-period waves

Setting the critical wave height for cargo handling works.

For the setting of the critical wave height for cargo handling works for the long-period waves portion, it is preferable to consider the type of ship and the cargo handling system in question, and to determine the wave heights separately based on a survey of the actual state of cargo handling. The critical wave height for cargo handling works for long-period waves is defined in **Table 4.5.1**.

Table 4.5.1 Critical Wave Height for Cargo Handling Works for Long-Period Waves [102)]

Level of the significant wave height of long-period waves	Assumed conditions	Critical wave height for cargo handling works (m)
1	Ship classes for which the permissible amount of motions in cargo handling is relatively large for surging, or ships whose natural period for surging is less than 1.5 minutes (medium sized ships: 1,000 to 5,000 DWT)	0.20
2	Ship classes for which the permissible amount of motions in cargo handling is moderate for surging, and ships whose natural period for surging is less than 1.5 minutes (general cargo ships: 5,000 to 10,000 DWT)	0.15
3	Ship classes for which the permissible amount of motions in cargo handling is small for surging, or ships whose natural period for surging is 2-3 minutes and under (container ships, mineral ore ships: 10,000 to 70,000 DWT)	0.10

4.6 Ship Waves

(1) It is preferable to consider ship waves during ship navigation in canals and waterways.

(2) Ship waves are caused when ships navigate. The large the ship is and the faster its speed is, the greater is the wave height of ship waves. When the propagation distance of ship waves becomes larger they end up attenuating, so they cause no serious problems in wide water areas. However, there are cases when they cause motions in small ships under anchor, floating docks, etc. inside harbors, in narrow. In addition, there are also cases where they have an effect on wave overtopping at revetments on both sides of a waterway, scouring and stability of armored blocks.

(3) Pattern of Ship Waves
If ship waves are viewed from air, it appears as shown in **Fig. 4.6.1**. It is composed of two groups of waves. One group of waves spread out in a shape like "ノ乀" from a point slightly ahead of the bow of the ship. The other group of waves is behind the ship and is such that the wave crest is perpendicular to the ship's navigation line. The former waves are termed the "divergent waves", while the latter are termed the "transverse waves". The divergent waves form concave curves; the closer to the navigation line, the smaller the gap between waves. On the other hand, the transverse waves are approximately arcshaped, with the gap between waves being constant. In deep water, the area over which the ship waves extend is limited within the area bounded by the two cusplines with the angles ±19º28' from the navigation line and starting from the origin lying somewhat in front of the bow of the ship. The divergent waves cross the transverse waves just inside the cusplines; this is where the wave height is largest. The wave steepness is smaller for the transverse waves than for the divergent waves, implying that the transverse waves often cannot be distinguished from an aerial photograph.

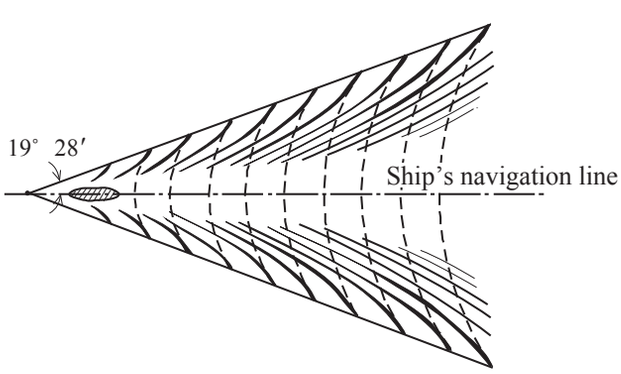


Fig. 4.6.1 Plan View of Ship Waves
(Solid lines show divergent waves and dotted line show transverse waves)

(4) Wavelength and Period of Ship Waves
The wavelength and period of ship waves differ for the divergent waves and the transverse waves, with the latter having both a longer wavelength and a longer period. Amongst the divergent waves, the wavelength and period are both longest for the first wave and then become progressively shorter.

① The wavelength of the transverse waves can be obtained by the numerical solution of the following equation, which is derived from the condition that the celerity of the transverse waves must be the same as the velocity at which the ship is navigating forward.

$$\frac{gL_t}{2\pi}\tanh\frac{2\pi h}{L_t} = V^2 \text{ : (provided } V = \sqrt{gh}\text{):} \tag{4.6.1}$$

where

L_t : wavelength of transverse waves (m)
h : water depth (m)
V : ship's navigation speed (m/s)

Note however that when the water is sufficiently deep, the wavelength of the transverse waves is given by the following equation:

$$L_0 = \frac{2\pi}{g}V^2 = 0.169{V_k}^2 \tag{4.6.2}$$

where

L_0 : wavelength of transverse waves at places where the water is sufficiently deep (m)
V_k : ship's navigation speed (kt); $Vk = 1.946V$

② The period of the transverse waves is equal to the period of progressive waves with the wavelength L_t in water of depth h. It is given by equation **(4.6.3)** or **(4.6.4)**.

$$T_t = \sqrt{\frac{2\pi}{g}L_t\coth\left(\frac{2\pi h}{L_t}\right)} = T_0\coth\left(\frac{2\pi h}{L_t}\right) \tag{4.6.3}$$

$$T_0 = \frac{2\pi}{g}V = 0.330V_k \tag{4.6.4}$$

where

T_t : period of transverse waves in water of depth h (s)
t_0 : period of transverse waves at places where the water is sufficiently deep (s)

③ The wavelength and period of the divergent waves are given by equations **(4.6.5)** and **(4.6.6)**, which are derived from the condition that the component of the ship's speed in the direction of travel of the divergent waves must be equal to the celerity of the divergent waves.

$$L_d = L_t\cos^2\theta \tag{4.6.5}$$

$$T_d = T_t\cos\theta \tag{4.6.6}$$

where

L_d : wavelength of divergent waves as measured in the direction of travel (m)
T_d : period of divergent waves (s)
θ : angle between the direction of travel of the divergent waves and the navigation line (º)

According to Kelvin's theory of wave-generation at places where the water is sufficiently deep, the angle of travel θ of the divergent waves can be obtained as shown in **Fig. 4.6.2**, as a function of the position of the place under study relative to the ship. Note however that for actual ships the minimum value of θ is generally about 40º, and θ is usually about 50º - 55º for the point on a particular divergent wave at which the wave height is the maximum. Note also that, as shown in the illustration in the figure, the angle θ directs the location of the source point Q from where the divergent wave has been generated; α is the angle between the cuspline and the navigation line.

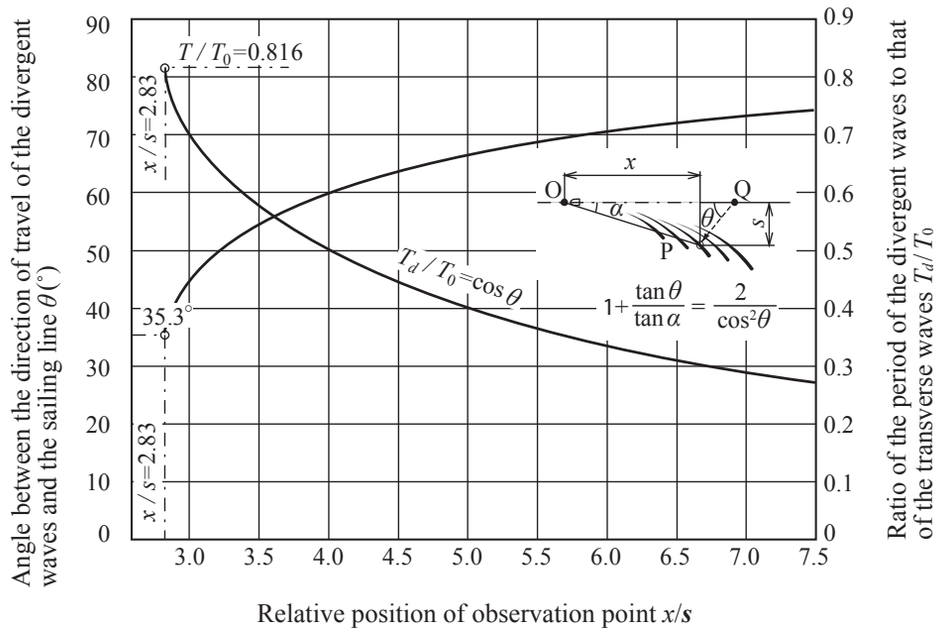


Fig. 4.6.2 Wave Height and Period of Divergent Waves at Places Where the Water is Sufficiently Deep

(5) Shoaling Effect on Ship Waves

As common with waves in general, ship waves are affected by the water depth, and their properties vary when the water depth decreases relative to the wavelength of ship waves. This shoaling effect on ship waves may be ignored if the following condition is satisfied:

$$V \leq 0.7\sqrt{gh} \tag{4.6.7}$$

The critical water depth above which ship waves may be regarded as deepwater waves is calculated by equation **(4.6.7)**, as listed in **Table 4.6.1**. As can be seen from this table, the waves generated by ships in normal conditions can generally be regarded as deepwater waves. Situations in which they must be regarded as shallow water waves include the following cases: a high-speed ferry travels through relatively shallow waters, a motorboat travels through shallow waters, and ship waves propagate into shallow waters. Note that ship waves in shallow water have a longer wavelength and period than those generated by the ship navigating in deep water at the same speed.

Table 4.6.1 Conditions under which Ship waves can be regarded as Deepwater Waves

Speed of vessel Vk (kt)	5.0	7.5	10.0	12.5	15.0	17.5	20.0	25.0	30.0
Water depth h (m)≥	1.4	3.1	5.5	8.6	12.4	16.9	22.0	34.4	49.6
Period of transverse waves T_0 (s)	1.7	2.5	3.3	4.1	5.0	5.8	6.6	8.3	9.9

(6) Height of Ship waves

The ship wake wave research committee of the Japan Association for Preventing Maritime Accidents has proposed the following equation for giving a rough estimate of the height of ship waves:

$$H_0 = \left(\frac{L_s}{100}\right)^{1/3}\sqrt{\frac{E_{\mathrm{HPW}}}{1620 L_s V_K}} \tag{4.6.8}$$

where

H_0 : characteristic wave height of ship waves (m), or the maximum wave height observed at a distance of 100 m from the navigation line when a ship is navigating at its full-load cruising speed

L_s : length of the ship (m)

V_K : full-load cruising speed (kt)

E_{HPW} : wave-generation horsepower (W)

The wave-generating horsepower E_{HPW} is calculated as follows. Refer to Reference 104) for the ship dimensions.

$$E_{HPW} = E_{HP} - E_{HPF} \tag{4.6.9}$$

$$E_{HP} = 0.6S_{HPm} \tag{4.6.10}$$

$$E_{HPF} = \frac{1}{2}\rho S V_0{}^3 C_F \tag{4.6.11}$$

$$S = 2.5\sqrt{\nabla L_s} \tag{4.6.12}$$

$$C_F = 0.075 \Big/ \left(\log\frac{V_0 L_s}{\nu} - 2\right)^2 \tag{4.6.13}$$

where

S_{HPm} : continuous maximum shaft power (W)
ρ_0 : density of seawater (kg/m³) ρ_0=1030(kg/m³)
V_0 : full-load cruising speed (m/s) V_0= 0.514V_K
C_F : frictional resistance coefficient
ν : coefficient of kinematic viscosity of water (m²/s) $\nu \fallingdotseq 1.2\times10^{-6}$ (m²/s)
∇ : full-load displacement of ship (m³)

Equation **(4.6.13)** has been obtained by assuming that the energy consumed through wave-generation resistance is equal to the propagation energy of ship waves, while the values of the coefficients have been determined as averages from the data from ship towing tank tests. The characteristic wave height H_0 varies from ship to ship, although for medium and large-size ships it is about 1.0 ~ 2.0 m. Tugboats sailing at full speed produce relatively large ship waves.

It is considered that the wave height attenuates in proportional to $S^{-1/3}$, where S is the distance of the observation point from the navigation line. It is also considered that the wave height is proportional to the cube of the cruising speed of the ship. Accordingly equation **(4.6.13)** has been obtained by assuming that the energy consumed through wave-generation resistance is equal to the propagation energy of ship waves, while the values of the coefficients have been determined as averages from the data from ship towing tank tests. The characteristic wave height H_0 varies from ship to ship, although for medium and large-size ships it is about 1.0 ~ 2.0 m. Tugboats sailing at full speed produce relatively large ship waves.

It is considered that the wave height attenuates in proportional to $S^{-1/3}$, where S is the distance of the observation point from the navigation line. It is also considered that the wave height is proportional to the cube of the cruising speed of the ship. Accordingly:

$$H_{max} = H_0\left(\frac{100}{S}\right)^{1/3}\left(\frac{V_k}{V_K}\right)^3 \tag{4.6.14}$$

where

H_{max} : maximum height of ship waves at any chosen observation point (m)
S : distance from the observation point to the navigation line (m)
V_k : actual cruising speed of the ship(kt)

Equation **(4.6.14)** cannot be applied if S is too small. However, equation **(4.6.14)** can be applied when either the ship length Ls or 100 m, whichever is the smaller.

The upper limit of the height of ship waves occurs when the wave steepness of the maximum wave of the divergent wave reach to the breaking criterion of H_{max}/L_t = 0.14. If the angle between the wave direction and the navigation line is assumed to beθ= 50º at the point on a divergent wave where the wave height becomes largest, the upper limit of the wave height at any given point is given by equation **(4.6.15)**. However, the conditions for deepwater waves shall be satisfied.

$$H_{limit} = 0.010V_k{}^2 \tag{4.6.15}$$

where

H_{limit} : upper limit of the height of ship waves as defined by the wave breaking conditions (m)

(7) Propagation of Ship Waves

① Among two groups of ship waves, the transverse waves propagate in the direction of ship's navigation line, and continue to propagate even if the ship changes course or stops. In this case, the waves have a typical nature of regular waves with the period being given by equation **(4.6.3)**, and they propagate at the group velocity, undergoing transformation such as refraction and others. Takeuchi and Nanasawa [105] gave an example of such transformations. Note however that as the waves propagate, the length of wave crest spreads out, and even when the water is of uniform depth, the wave height attenuates in a manner inversely proportional to the square root of the distance traveled.

② The direction of propagation of a divergent wave varies from point to point on the wave crest. According to Kelvin's theory of wave-generation, the angle between the direction of propagation and the navigation line is $\theta = 35.3^{\circ}$ at the outer edge of a divergent wave. As one moves inwards along the wave crest, the value of θ approaches 90°. The first wave arriving at a any particular point has the angle $\theta = 35.3^{\circ}$, while θ getting gradually larger for subsequent waves. This spatial change in the direction of propagation of the divergent waves can be estimated using **Fig. 4.6.2**.

③ The propagation celerity of a divergent wave at any point on the wave crest is the group celerity corresponding to the period T_d at that point (see equation **(4.6.6)**). In the illustration in **Fig. 4.6.2**, the time needed for a component wave to propagate at the group celerity from the point Q at wave source to the point P is equal to the time taken for the ship to travel at the speed V from the point Q to the point O. Since each wave profile propagates at the wave celerity (phase velocity), the waves appear to pass beyond the cuspline and vanish one after the other at the outer edge of the divergent waves.

(8) Generation of Solitary Waves

When a ship navigates through shallow waters, solitary waves are generated in front of the ship if the cruising speed Vk (m/s) approaches $\sqrt{gh}$. Around the river mouths, there is a possibility of small ships being affected by such solitary waves generated by other large ships.[106]

4.7 Wave Pressure and Wave Force

4.7.1 General [107), 108)]

(1) Wave Force Calculation

The wave force acting on port facilities is generally determined using appropriate hydraulic model tests, numerical calculations or methods described in **4.7.2 Wave Force on Upright Wall**, with the waves determined by the procedures described in **Chapter 4 Waves**. However, in the event of an increase in wave height or an increase in wave force due to impulsive breaking waves, appropriate consideration needs to be given to these aspects depending on the shape and structural characteristics of the breakwater.

(2) Structure Type and Wave Forces

Wave forces can be generally classified by the type of structure as follows:

① Wave force acting on a wall-type structure

② Wave force acting on armor stones or concrete blocks

③ Wave force acting on submerged members

④ Wave force acting on structures near the water surface

The wave forces are different for each type of structure. It is thus necessary to use an appropriate calculation method in accordance with the conditions including structural type. For some types of structures with a few experiences of construction, their wave actions have not been sufficiently resolved, and therefore it is preferable to carry out studies including hydraulic model tests for such structures.

Wave forces and resistance forces acting on armor stones and concrete blocks differ greatly depending on their shapes and positions in addition to conditions of the waves acting on them. Therefore, when verifying performance, the required mass for the armor stones and concrete blocks are usually determined directly from wave conditions rather than calculating the acting wave force (see **Part III, Chapter 2, 1.7.2 Required Mass of Armor Stones and Blocks in Composite Breakwater Foundation Mound against Waves**).

(3) Wave Irregularity and Wave Force

Sea waves are irregular with the wave height and period varying from wave to wave. Depending on the water depth and the topography of the sea bottom, wave forces such as non-breaking, breaking or after-breaking act on the structure. When calculating the wave force, it is important to include the waves that cause the severest effect on the structure. It is necessary to give sufficient consideration to wave irregularity and to the characteristics of the wave force in accordance with the type of structure.

In general, it may be assumed that the larger the wave height, the greater the wave force becomes. It is thus acceptable to focus on the wave force of the highest wave among a train of random waves attacking the structure. However, with regard to the stabilities of concrete blocks or armor stones on the slope and wave force acting on the floating structures and cylindrical structures with small rigidity, it is preferable to consider the effect of the successive action of the random waves.

(4) Examination of Wave Force by Hydraulic Model Tests
When examining wave force by hydraulic model tests, it is necessary to give sufficient consideration to the failure process of the structure and to use an appropriate measurement method. It is also preferable to give sufficient consideration to the irregularity of waves. In particular, when carrying out experiments using regular waves, an examination against the highest wave should be included.

(5) Examination of Wave Force by Numerical Calculation
A great deal of labor and expense is required for examining the wave force due to model tests, and usually there are limits to the experimental case and the measurement items. On the other hand, in recent years it has become possible to employ numerical calculations when computing the wave force acting on structures. When the numerical calculations are employed in actual design, it is necessary to verify the appropriateness of the calculation results by comparing them with on-site observations and model tests, but once the suitability is confirmed, computation of wave force with less labor and expense becomes possible. The CADMAS-SURF [109] is a numerical computation program developed for the purpose of assisting the structually resistive design against wave action and with it is possible to examine the interactions of waves, ground and structures and the impulsive breaking pressure.

(6) Design Values for Wave Force
The partial factors differ depending on the structural type as far as the design values for the wave force are concerned. The wave force as used here in principle indicates the characteristic value of the wave force. The partial factor for wave force can be referred to the respective structure type.

4.7.2 Wave Force on Upright Walls [110), 111), 112), 113)]

(1) General Characteristics of Wave Force on Upright Walls

① The major parameters that affect the wave force acting on an upright wall are wave period, wave height, wave direction, water level, water depth, bottom slope, water depth of the crown of the foundation mound, the front berm width of foundation mound, slope of foundation mound, the crown height of upright wall, and water depth at base of the upright wall. In addition, it is also necessary to consider the effect of the wall alignment. The wave force on an upright wall with a concaved alignment may be larger than that on an upright, straight wall of infinite length. Furthermore, if the front of upright wall is covered with wave-dissipating concrete blocks, the characteristics of these blocks and the crown height and width will affect the wave force.

② Types of wave force
The wave force acting on an upright wall can be classified according to the type of waves such as a standing wave force, a breaking wave force, or a wave force after breaking. It is considered that the changes of wave forces are continuous. A standing wave force is produced by waves whose height is small compared with the water depth, and the change in the wave pressure over time is gradual. As the wave height increases, the wave force also increases. In general, the largest wave force is generated by the waves breaking just a little off the upright wall. Accordingly, with the exception of very shallow water conditions, the force exerted by waves breaking just in front of an upright wall is larger than the wave force by higher waves that have already broken. It is necessary to note that when breaking waves act on an upright wall on a steep seabed, or on an upright wall set on a high mound, a very strong impulsive breaking wave force may appear.

(2) Wave Forces of Standing Waves or Breaking Waves when the Peak of Waves is on the Wall Surface

① Goda's formula

(a) It is standard to calculate the maximum horizontal wave force acting on an upright wall and the simultaneous uplift using Goda's formula as shown below. Goda's formula is that proposed by Goda taking into consideration wave pressure experiments and results of application of the formula to the existing breakwaters and modified to include the effects of wave direction. Its single-equation formula enables one to calculate the wave force from the standing to breaking wave conditions without making any abrupt transition. However, where the upright wall is located on a steep seabed, or built on a high mound, and is subjected to a strong impulsive wave pressure due to breaking waves, the formula may underestimate the wave force. It should therefore be carefully applied with consideration of the possibility of occurrence of impulsive wave pressure due to breaking waves (see **4.7.2 (4) Impulsive Breaking Wave Force**).

The wave pressure given by Goda's formulas takes the hydrostatic pressure at the still water condition as

the reference value. Any hydrostatic pressure before wave action should be considered separately. Further, the formula aims to examine the stability of the whole body of an upright wall. When breaking wave actions exist, the formula does not necessarily express the local maximum wave pressure at the respective positions; thus such should be considered during examination of the stress of structural members.

(b) Wave pressure on the front face according to the Goda's formulas

The wave pressure on the front face of an upright wall in the Goda's formula is a linear distribution. Wave pressure is 0 at the height expressed as η^* in equation (**4.7.1**), maximum value expressed as p_1 in equation (**4.7.2**) at still water level, and expressed as p_2 in equation (**4.7.3**) at the sea bottom. The formula considers wave pressure from the bottom to the crown of the upright wall (see **Figs. 4.7.1** and **4.7.2**).

$$\eta^* = 0.75(1+\cos\beta)\lambda_1 H_D \tag{4.7.1}$$

$$p_1 = 0.5(1+\cos\beta)(\alpha_1\lambda_1 + \alpha_2\lambda_2\cos^2\beta)\rho_0 g H_D \tag{4.7.2}$$

$$p_2 = \frac{P_1}{\cosh(2\pi h/L)} \tag{4.7.3}$$

$$p_3 = \alpha_3 p_1 \tag{4.7.4}$$

In this equation, η^*, p_1, p_2, p_3, ρ_0, g, β, λ_1, λ_2, h, L, H_D, α_1, α_2 and α_3 respectively represent the following values:

η^* : height above still water level at which intensity of wave pressure is 0 (m)
p_1 : intensity of wave pressure at still water level (kN/m^2)
p_2 : intensity of wave pressure at sea bottom (kN/m^2)
p_3 : intensity of wave pressure at toe of the upright wall (kN/m^2)
$\rho_0 g$: unit weight of water (kN/m^3)
β : angle between the most dangerous direction within the range of ±15° from the main wave direction and the line perpendicular to the faceline of the upright wall (°)
λ_1, λ_2 : wave pressure correction factor (1.0 is the standard value)
h : water depth in front of the upright wall (m)
L : wavelength at water depth h used in calculation as specified in the item (d) below (m)
H_D : wave height used in calculation as specified in the item (d) below (m)
α_1 : value expressed by the following equation:

$$\alpha_1 = 0.6 + \frac{1}{2}\left\{\frac{4\pi h/L}{\sinh(4\pi h/L)}\right\}^2 \tag{4.7.5}$$

α_2 : smaller value of $\dfrac{h_b - d}{3h_b}\left(\dfrac{H_D}{d}\right)^2$ or $\dfrac{2d}{H_D}$

$$\alpha_2 = \min\left\{\frac{h_b - d}{3h_b}\left(\frac{H_D}{d}\right)^2, \frac{2d}{H_D}\right\} \tag{4.7.6}$$

α_3 : value expressed by the following equation:

$$\alpha_3 = 1 - \frac{h'}{h}\left\{1 - \frac{1}{\cosh(2\pi h/L)}\right\} \tag{4.7.7}$$

In this equation, h_b, d and h' respectively represent the following values:

h_b : water depth at an offshore distance of 5 times the significant wave height from the upright wall (m)
d : water depth at the crest of either the foot protection works or the mound armoring units of whichever is higher (m)
h' : water depth at toe of the upright wall (m)

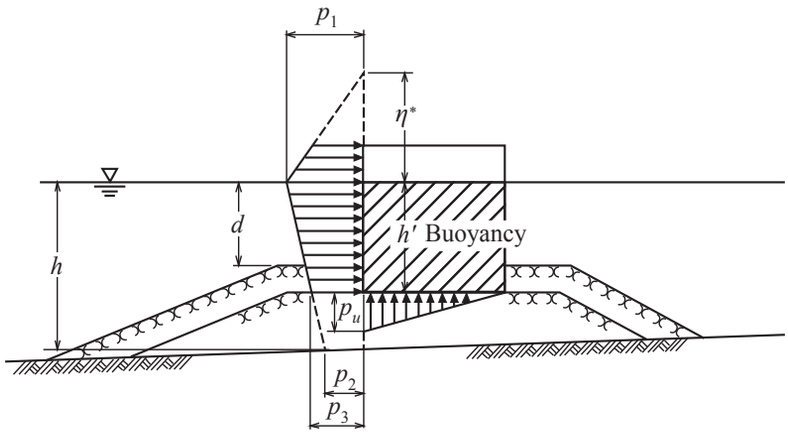


Fig. 4.7.1 Wave Pressure Distribution Used in Design Calculation

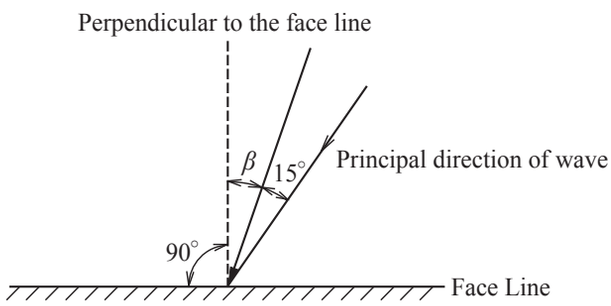


Fig. 4.7.2 Way of Obtaining Incident Wave Angle β

(c) Uplift on the bottom of upright wall

In Goda's formulas, the uplift acting on the bottom of an upright wall is described by a triangular distribution, with the pressure intensity at the front toe p_u given by the following equation and 0 at the rear toe.

$$p_u = 0.5(1+\cos\beta)\alpha_1\alpha_3\lambda_3\rho_0 g H_D \tag{4.7.8}$$

In this equation, p_u and λ_3 respectively represent the following values:

p_u : uplift pressure acting on the bottom of the upright wall (kN/m2)
λ_3 : uplift pressure correction factor (1.0 is the standard value)

(d) Wave height and wavelength used in the wave pressure calculation

In Goda's formulas, the wave height H_D and the wavelength L are the height and wavelength of the highest wave. The wavelength of the highest wave is that corresponding to the significant wave period, while the height of the highest wave is as follows:

1) When the highest wave does not have effect of wave breaking:

$$H_D = H_{\max} = 1.8H_{1/3} \tag{4.7.9}$$

In this equation, $H_{\max}$ and $H_{1/3}$ respectively represent the following values:

$H_{\max}$: highest wave height of incident waves as a progressive wave at the water depth of the upright wall (m)
$H_{1/3}$: significant wave height of incident waves as a progressive wave at the water depth of the upright wall (m)

2) When the highest wave has effect of wave breaking:

H_D: maximum wave height considering transformation due to the breaking of random waves (m)

(e) Highest wave

Since Goda's formulas represents the wave force on individual wave, in the breakwater performance

verifications in general, it is necessary to use the wave parameters of the largest wave force from a wave group. The highest wave shall be subject to consideration. The occurrence of the highest wave in a random wave group is probabilistic, and so it is not possible to determine the parameters of the wave explicitly. Nevertheless, after examination of the results of applying the current method to breakwaters in the field, it is standard to use 1.8 times the significant wave height as the height of the highest wave where no transformation of breaking wave is observed. It has also become standard to use the wavelength corresponding to the significant wave period as the wavelength of the highest wave.

In order to determine whether or not the highest wave is subject to wave breaking, the diagrams for determining the highest wave height (**Fig. 4.3.10 (a)–(e)** in **4.3.6 Wave Breaking**) should be used by referring to the location of the peak wave height in the zone in the onshore side of the 2% attenuation line. It is acceptable to consider that the highest wave is not subject to wave breaking when the water is deeper than that at the peak height, but that it is subject to wave breaking when the water is shallower than this. If the highest wave height is to be obtained using the approximate equation (**4.3.23**) in **4.3.6 Wave Breaking**, h_b should be substituted as h in the first term in the braces { } on the right-hand side of the equation.

If using a value other than 1.8 as the coefficient on the right-hand side of equation (**4.7.9**), it is necessary to conduct sufficient examinations into the occurrence of the highest wave and then choose an appropriate value (see **4.1 Basic Matters Relating to Waves**).

(f) Wave pressure correction factor λ_1, λ_2, λ_3

Equations (**4.7.1**) – (**4.7.8**) are the generalized version of Goda's formulas. It contains three correction factors so that it can be applied to walls of different conditions. For an upright wall, the correction factors are of course 1.0. The wave pressure acting on other types of wall such as a caisson covered with wave-dissipating concrete blocks or an upright wave-dissipating caisson may be expressed using the generalized Goda's formulas with appropriate correction factors (see **4.7.2 (5) Wave Force Acting on Upright Wall covered with Wave-dissipating Concrete Blocks** and **4.7.2 (7) Wave Force Acting on Upright Wave-absorbing Caisson**).

(g) Features and application limits of the Goda's formulas

The first feature of Goda's formulas is that the wave force from standing waves to breaking waves can be calculated continuously, including the effect of surrounding conditions. The parameter α_1 given by equation (**4.7.5**) expresses the effect of the period (strictly speaking h/L); it takes the limiting values of 1.1 for shallow water waves and 0.6 for deepwater waves. The effect of period also appear when determining the maximum wave height to be used in the calculation; for a constant deepwater wave height, the longer the period, the larger the maximum wave height. Since Goda's formulas incorporates the effect of period on the wave pressure as well as on the maximum wave height, it is necessary to take sufficient care when determining the period in the design conditions.

Another feature of Goda's formulas is that the change in the wave force with the foundation mound height and the bottom slope is considered by means of the parameter α_2. As can be seen from equation (**4.7.6**), as the foundation mound height is gradually increased from zero (i.e., $d = h$), α_2 gradually increases from zero to its maximum value. After reaching its limit value, α_2 then decreases until it reaches zero again when $d = 0$. The limit value of α_2 is 1.1; combining this with the limit value of α_1 of 1.1, the intensity of the wave pressure p_1 at the still water level is given $2.2\rho_0 g H_D$.

With regard to the effect of the bottom slope, h_b within the equation for α_2 is taken as the water depth at the distance of 5 times the design significant wave height from the upright wall. Because of this artifice, a steep bottom slope results in the same effect as having a high foundation mound. The effect of the bottom slope also appears when determining the maximum wave height to be used in the calculation. In the wave breaking zone, the steeper the bottom slope, the larger the wave height, because the wave height used in the calculation is the maximum wave height at a distance $5H_{1/3}$ offshore from the upright wall. The bottom slope thus has a strong influence on the wave force, and so care must be taken when setting the bottom slope in the design conditions.

As explained above, Goda's formulas consider the effects of the foundation mound height and the bottom slope on the wave pressure. Nevertheless, for an upright wall on a high mound or a steep seabed, a large impulsive breaking wave force may act, and under such conditions the Goda's formulas may underestimate the wave force. When applying the Goda's formulas, it is thus preferable to pay attention to the risk of an impulsive breaking wave force arising. In particular, with a high mound, it is necessary to consider not only α_2 in equation (**4.7.6**) but also the impulsive breaking wave force coefficient α_I by Takahashi et al.[116] (see **4.7.2 (4) ⑥ Impulsive breaking wave forces acting on composite breakwater**), and to use α_I in place of α_2 when α_I is the larger of the two.

One more problem with Goda's formulas concerns its applicability to extremely shallow waters, for example near to the shoreline. It is difficult, however, to clearly define where the limit of applicability lies. For cases such as the wave force acting on an upright wall near the shoreline, it is advisable to use other calculation equations together with the Goda formula. (See **4.7.2 (10) Wave Force Acting on Upright Wall Located Considerably Toward the Landside from the Breaker Line**).

(h) Effect of wave direction in Goda's formula

Although results from a number of experiments on the effect of wave direction on the wave force are available, there are still many points that are unclear. Traditionally, for standing waves, no correction has been made for wave direction to the wave force. The effects of wave direction have been considered only for breaking waves, by multiplying the wave force by $\cos^2\beta$. However, this has resulted in the irrational situation whereby the breaking wave force is assumed to decrease as the wave angle increases, reaching zero at the limiting value $\beta = 90°$, and yet standing waves are assumed to remain at the perfect standing wave condition. In other words, because actual breakwaters are finite in extension, when the incident angle is large (i.e., oblique wave incidence), it takes a considerably large distance from the tip of breakwater until the wave height becomes two times the incident height. At the limiting value of $\beta = 90°$, it becomes an infinite distance. In this case, it is appropriate to consider that the wave pressure of progressive waves acts on the upright wall. Furthermore, even in cases where the breakwater can be taken to extend infinitely, when using second-order approximation finite amplitude wave theory, the wave pressure form oblique incident waves decreases slightly in comparison to incidence at right angles and its degree becomes proportionate to the wave steepness. Considering these points and application to the breakwaters in the field, equation (**4.7.2**) for wave direction has been corrected by multiplying α_2 which represents mound effects with $\cos^2\beta$, and then multiplying the whole term by $0.5(1+\cos\beta)$.

② Application of other theory and formulas

Goda's formula enables continuous determination of wave forces with considerable precision from standing waves to breaking waves without categorizing them by their application limits. But when the ratio of the wave height to the water depth is small and a standing wave force is obviously exerted on an upright wall, a high-accuracy standing wave theory may be applied. In this case, however, it is necessary to give sufficient consideration to the irregularity of waves in the field, and preferable to examine the force for the highest wave. Moreover, the Sainflou formula [117] and Hiroi's formula [118] may also be used for wave force calculations. When applying these methods, adequate care is needed in determining applicability.

③ Wave force and significant wave period for waves composed of two wave groups with different periods

Examples of two wave groups with different periods being superimposed are such a case that waves enter a bay from the outer sea and another group of waves are generated within the bay. Another case is the superposition of diffracted waves coming from the entrance of a harbor and waves transmitted by wave overtopping. In such cases, the spectrum is bimodal (i.e., having two peaks), and there are actual cases of such observations in the field. [120] Tanimoto, Kitamura et al.[121]) carried out experiments on the wave force acting on the upright section of a composite breakwater by using waves with a bimodal spectrum, and verified that Goda's formulas can be applied even in such a case. They also proposed a method for calculating the significant wave period to be used in the wave force calculation (see **4 Waves**). If each frequency spectrum of the two wave groups before superimposition can be considered to be a Bretschneider-Mitsuyasu type, the significant wave period after superimposition may be obtained using the method by Tanimoto et al. Then this significant wave period may be used in wave force calculation.

④ Wave force for low crested upright wall

When applying Goda's formulas on breakwaters, if the crown height of the upright wall is low, the reduction in resistance force due to the fall in weight becomes greater than the reduction in wave force resulting from the decrease in the range of wave pressure acting on the wall. Therefore, in general, the wall needs to be widened. However, the stability of an upright wall does tend to increase as the crown height is reduced. Nakata, Terauchi et al.[122] have proposed a method for calculating the wave force for a breakwater with a low crown height. In the method, the front wave pressure and the uplift from the Goda's formulas are multiplied by a modification factor λ_h, thus reducing the wave force.

⑤ Wave force for high crested upright wall

When the crown of the upright wall is considerably higher than that for a normal breakwater, there will be no wave overtopping, meaning that the wave force may be larger than that given by Goda's formulas. Mizuno, Sugimoto et al.[123] carried out experiments into the wave force acting on a breakwater with a high crown. This result may be referred.

⑥ Wave force on inclined walls

When the wall is slightly inclined, such as a trapezoidal caisson, the horizontal wave force is more-or-less the same as that for an upright wall. However, it is necessary to consider the vertical component of the wave force acting on the inclined surface, along with the reduction in uplift. Tanimoto and Kimura [124] have carried out experiments on the wave force for slightly inclined walls, and have proposed a method for calculating the wave force.

⑦ Uplift on caisson with footing

When a caisson has a footing, a wave force acts downwards on the upper surface of the footing on the seaside, and an uplift of p_u' acts at the front toe, while the uplift at the rear toe is zero. Nevertheless, in general the

resultant force is not significantly different to that without the footing. It is thus acceptable to ignore the footing, and to assume that the uplift has a triangular distribution as shown in **Fig. 4.7.3**, with the uplift p_u at the front toe being given by equation **(4.7.8)**, and the uplift at the rear toe being zero. If the footing is extremely long, however, it is necessary to calculate the uplift appropriately, considering the change in the uplift p_u' at the front toe of the footing.

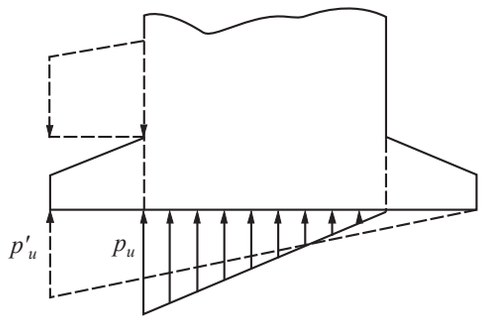


Fig. 4.7.3 Uplift when there is a Footing

⑧ In case of wide mound berm in front of upright wall
The wave force acting on the upright wall of a composite breakwater varies not only with the mound height but also with the berm width and the front slope of foundation mound (see **4.7.2 (4) Impulsive Breaking Wave Force**). As explained, of these three factors, Goda's formulas incorporates only the effect of the mound height. Consequently, if the width and/or slope of the foundation mound are considerably different from normal, it is preferable to carry out examination using hydraulic model tests. Note however that if the berm is sufficiently wide, it may be considered as a part of the topography of the sea bottom. Even with the standard formula, if the width is more than one half of the wavelength, it is possible to use the water depth on the mound for calculation of both the wave height and the wavelength.

⑨ Wave force acting on an upright wall comprised of vertical cylinders
Nagai, Kubo et al.[125] as well as Hayashi, Karino et al.[126] have carried out studies on the wave force acting on an upright wall comprised of cylinders such as a pile breakwater. Through their researches, it has been verified that the wave force is not greatly different from that acting on an upright wall with a flat face. It is thus acceptable to treat an upright wall comprised of cylinders as having a flat face and calculate the wave force using Goda's formulas.

(3) Negative Wave Force of Wave Troughs on Wall Surfaces

① General
When the trough of a wave is at a wall, a negative wave force acts corresponding to the trough depth of the water surface from the still water level. A negative wave force is a wave force that obtained through suitable hydraulic model tests or through appropriate calculations. It is a force directed seaward and may be comparable in magnitude to a positive wave force when the water is deep and the wavelength is short.

② Negative wave pressure distribution
The negative wave pressure acting on an upright wall at the wave trough can be approximately estimated as shown in **Fig. 4.7.4**. Specifically, it can be assumed that a wave pressure acts toward the sea, with the magnitude of this wave pressure being zero at the still water level and having a constant value of p_n from a depth $0.5H_D$ below the still water level right down to the toe of the wall. Here, p_n is given as follows:

$$p_n = 0.5\rho_0 g H_D \tag{4.7.10}$$

where

p_n : intensity of wave pressure in constant region(kN/m^2)
$\rho_0 g$: Unit weight of seawater (kN/m^3)
H_D : wave height used in performance verification (m)

In addition, the negative uplift acting on the bottom of the upright wall can be assumed to act as shown in **Fig. 4.7.4**. Specifically, it can be assumed that an uplift acts downwards with its intensity being p_n as given by equation **(4.7.10)** at the front toe, zero at the rear toe, and having a triangular distribution in-between. Incidentally, it is necessary to use the highest wave height as the wave height H_D used in the performance verification.

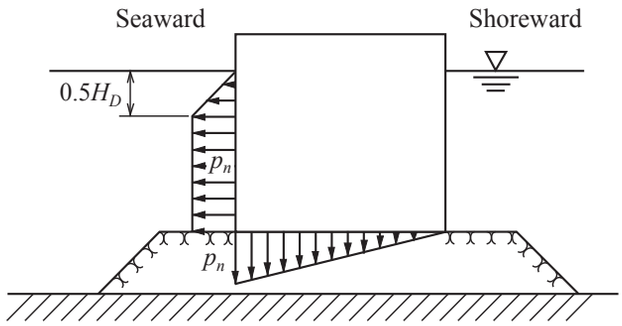


Fig. 4.7.4 Negative Wave Pressure Distribution

③ Negative wave force by finite amplitude wave theory
Goda and Kakizaki [127] have carried out a wave force calculation based on the fourth order approximate solutions of a finite amplitude standing wave theory, and presented calculation diagrams for negative wave pressure. It has been verified that their calculation results agree well with experimental results. When the water is deep and standing waves are clearly formed, it is acceptable to use the results of this finite amplitude standing wave theory of higher order approximation. It should be noted that, for a deepwater breakwater, the negative wave force at the wave trough may become larger than the positive wave force at the wave crest, and that the upright wall may slide toward offshore.

(4) Impulsive Breaking Wave Force

① General
An impulsive breaking wave force is generated when the wave front of a breaking wave strikes a wall surface. It has been shown from model tests that under certain conditions the maximum wave pressure may rise as much as several tens of times the hydrostatic pressure corresponding to the wave height ($1.0\rho_0 gH_D$). However, such a wave pressure acts only locally and for a very short time, and even slight changes in conditions lead to marked reduction in the wave pressure. Because of the impulsive nature of the wave force, the effects on stability and the stress in structural elements vary according to the dynamic properties of the structure. Accordingly, when there is a risk of a large impulsive breaking wave force due to breaking waves being generated, it is necessary to take appropriate countermeasures by understanding the conditions of the impulsive breaking wave force generation and the wave force characteristics by means of hydraulic model tests. It is preferable to avoid the use of cross-sectional shapes and structures that may give rise to a strong impulsive breaking wave force. Where generation of strong impulsive breaking wave force is unavoidable due to steep sea bottom or other reasons, it would be preferable to arrange ways of mitigating wave forces such as by installing appropriate wave-dissipating works.

② Conditions of Generation of Impulsive Breaking Wave Forces
A whole variety of factors contribute to generation of an impulsive breaking wave force, and so it is difficult to describe the conditions in general. Nevertheless, based on the results of a variety of experiments, it can be said that an impulsive breaking wave force is liable to occur in the following cases when the wave incident angle β (see **Fig. 4.7.2**) is less than 20º.

(a) In the case of steep bottom
When the three conditions, such that the bottom slope is steeper than about 1/30; there are waves that break slightly off the upright wall; and their equivalent deepwater wave steepness is less than 0.03, are satisfied simultaneously, then an impulsive breaking wave force is liable to be generated.

(b) In the case of high foundation mound
Even if the bottom slope is mild, the shape of the rubble mound may cause an impulsive breaking wave force to be generated. In this case, in addition to the wave conditions, the crown height, the berm width and the slope gradient of the mound all play a part, and so it is hard to determine the conditions under which such an impulsive breaking wave force will be generated. In general, an impulsive breaking wave force will be generated when the mound is relatively high, the berm width is relatively wide or the slope gradient is gentle, and breaking waves form a vertical wall of water at the slope or at the top of the mound.[128] When the seabed slope is gentler than about and the ratio of the depth of water above the top of the mound including armor units to the water depth above the seabed is greater than 0.6, it may be assumed that a large impulsive breaking wave force will not be generated.

③ Countermeasures
If a large impulsive wave force due to breaking waves acts on an upright wall, the wave force can be greatly

reduced by sufficiently armoring the front with wave-dissipating concrete blocks. In particular, with a high mound, a sufficient covering with wave-dissipating concrete blocks can prevent the generation of the impulsive breaking wave force itself. In some cases the action of an impulsive wave force can also be avoided by using special caissons such as perforated-wall caissons or sloping-top caissons.[128] The wave direction also has a large effect on the generation of an impulsive breaking wave force, and therefore, one possible countermeasure is to ensure that the wave direction is not perpendicular to the breakwater alignment.

④ Examining wave force using model tests
When examining the wave force using hydraulic model tests for the case that an impulsive breaking wave force due to breaking wave acts, it is necessary to give consideration to the response characteristics of the structure. For example, the examination of the stability of upright wall as a whole is preferably conducted by sliding experiment and the strength of members such as parapets by stress measurement experiment.

⑤ Impulsive breaking wave force due to breaking waves acting on an upright wall on a steep seabed.

(a) Water depth of upright wall inducing maximum wave pressure and the mean intensity of wave pressure
Mitsuyasu, [129] Hom-ma, Horikawa et al.[130], Morihira, Kakisaki et al.[131], Goda and Haranaka,[132] Horikawa and Noguchi, [133] and Fujisaki, Sasada et al.[134] have all carried out studies on the impulsive breaking wave force due to breaking waves acting on an upright wall on a steeply sloping sea bottom. In particular, Mitsuyasu carried out a wide range of experiments using regular waves whereby he studied the breaking wave force acting on an upright wall on uniform slopes of gradient 1/50, 1/25, and 1/15 for a variety of water depths. He investigated the change in the total wave force with the water depth at the location of the upright wall, and obtained an equation for calculating the water depth h_M at the upright wall for which the impulsive wave force is largest. When the Mitsuyasu equation is rewritten in terms of the deepwater wavelength, it becomes as follows:

$$\frac{h_M}{H_0} = C_M \left(\frac{H_0}{L_0} \right)^{-1/4} \tag{4.7.11}$$

where

$$C_M = 0.59 - 3.2 \tan\theta \tag{4.7.12}$$

H_0 : deepwater wave height (m)
L_0 : deepwater wavelength (m)
$\tan\theta$: gradient of uniform slope

Hom-ma and Horikawa et al.[135] have proposed a slightly different value for C_M based on the results of experiments with a gradient of 1/15 and other data. In any case, the impulsive breaking wave pressure is largest when the structure is located slightly shoreward of the wave breaking point for progressive waves.

Fig. 4.7.5 shows the total wave force when the impulsive breaking wave force is largest for a number of slope gradients, as based on the results of Mitsuyasu's [129] experiments. In this figure, the mean intensity of the wave pressure *p,* determined by assuming that wave pressure acts from the sea bottom to the height of 0.75 times limiting breaker height H_b above the still water surface, has been obtained and then divided by $p_0 g H_b$ to make it dimensionless; it has then been plotted against the deepwater wave steepness. It is possible to gain an understanding of the overall trend from this figure. Specifically, it can be seen that the smaller the wave steepness, the larger the impulsive breaking wave force is generated. Also, as the slope gradient becomes smaller, the intensity of the maximum impulsive breaking wave force decreases.

(b) Conditions for generation of impulsive breaking wave force
The conditions for the occurrence of an impulsive breaking wave force on a steep seabed, as described in **4.7.2 (4) ② (a) in the case of steep sea bottom**, have been set by primarily employing **Fig. 4.7.5** as a gross guideline. For random waves in the sea, the equivalent deep water wave steepness can be calculated as the ratio of the equivalent deepwater wave height corresponding to the highest wave height $H_{\max}$ to the deepwater wavelength corresponding to the significant wave period: where the wave height $H_{\max}$ is to be calculated at the distance $5H_{1/3}$ from the upright wall taking into account of wave transformation due to random wave breaking. One may refer to **Fig. 4.7.5** in order to obtain an approximate estimate of the mean intensity of the wave force for this equivalent deepwater wave steepness. In this case, H_b should be taken to be the aforementioned $H_{\max}$. One can also envisage an installation of a breakwater at a place where the risk of impulsive breaking wave force generation is not large for the design waves. However, when placing an upright wall closer to the shore where waves already broken act upon, it becomes important to carry out examination for waves with a height lesser than that of the design waves.

(c) Impulsive breaking wave force acting on an upright wall on a horizontal floor adjoining a steep slope
Takahashi and Tanimoto et al.[135] have carried out studies on the impulsive breaking wave force acting on an upright wall on a horizontal floor joining to a steep slope. They employed a horizontal berm connected to a slope of gradient 1/10 or 3/100 in a water tank, and then measured the wave pressure that acts on an upright wall at a variety of positions with regular waves. They have proposed an equation valid for certain wave conditions for calculating the upright wall position at which the wave force is largest and the maximum wave force in that condition.

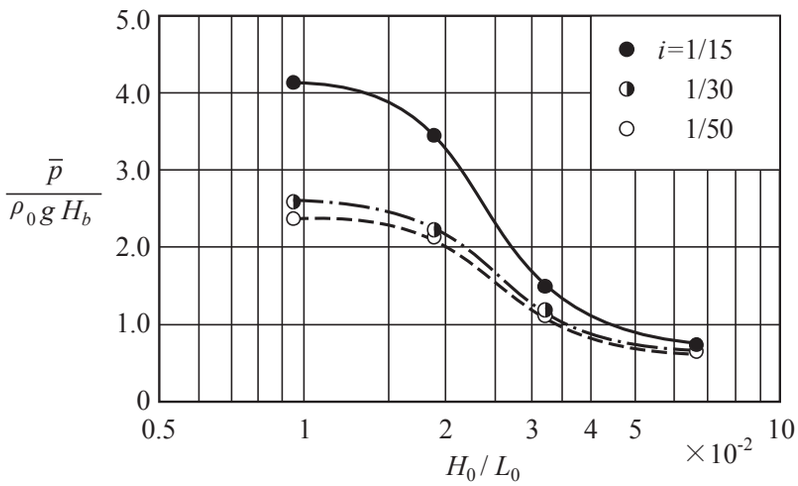


Fig. 4.7.5 Mean Intensity of Wave Force for the Severest Wave Breaking (Upright Wall on a Steep Slope)

⑥ Impulsive breaking wave force acting on composite breakwater

(a) Effect of the mound shape (impulsive breaking wave pressure coefficient)
Takahashi et al.[116] have proposed, based on the results of sliding experiments, [128] the impulsive breaking wave pressure coefficient α_I. This is a coefficient that represents the extent of the impulsive breaking wave force due to breaking waves when the foundation mound is high. It is expressed as the function of the ratio of the wave height to the depth of water above the mound in front of the caisson H_D/d, the ratio of the depth of water above the mound to the original water depth at the upright wall d/h, and the ratio of the berm width of the mound to the wavelength at this place B_M/L. Note that the wave height H_D is the design wave height, namely highest wave height. The impulsive breaking wave pressure coefficient α_I is expressed as the product of α_{I0} and α_{I1} as in the following equations:

$$\alpha_I = \alpha_{I0}\alpha_{I1} \tag{4.7.13}$$

$$\alpha_{I0} = \begin{cases} H_D/d & (H_D/d \le 2) \\ 2 & (H_D/d > 2) \end{cases} \tag{4.7.14}$$

Fig. 4.7.6 shows the distribution of α_{I1}. It attains the maximum value of 1 when d/h is 0.4 and B_M/L is 0.12. The impulsive breaking wave pressure coefficient α_I takes values between 0 and 2; the larger the value of α_I, the larger the impulsive breaking wave force is. When calculating the wave force using Goda's formulas, among α_I and α_2, whichever larger shall be used. The equation for α_I has been formulated based mainly on the results of sliding experiments when H_D/h is relatively large and may be used when examining the sliding of an upright wall on the condition of $H_D/h \ge 0.5$. When $H_D/h<0.5$, $h=2H_D$ may be used, for the sake of convenience, in the calculation of α_{I1}.[136]

(b) Effect of the crown height of the upright wall
The higher the crown height, the greater the risk of an impulsive breaking wave force being generated. This is because the steep front of a breaking wave often takes a nearly vertical cliff of water above the still water level, and if there is an upright wall at this place, the impact of the wave front results in the generation of an impulsive force. For example, Mizuno et al.[123] have pointed out the tendency that, when the crown is high, an impulsive breaking wave force is generated even when the mound is relatively low.

(c) Effect of the wave direction
According to the results of the sliding experiments of Tanimoto et al.[127], even if conditions are such that a large impulsive breaking wave force is generated when the wave angle β is 0º, there is a rapid drop in the

magnitude of the wave force as β increases to 30º or 45º. When the alignment of breakwater is oblique at the direction of incident waves, the impulsive breaking wave force will not generate or actually be neglected because of the weak effect of it against sliding, even if generated. By considering the fluctuation in the wave direction, it is reasonable to assume that the condition for the generation of an impulsive wave force is that β is less than 20º.

(d) Dynamic response of the upright section to an impulsive breaking wave force and the sliding of upright section

When an impulsive breaking wave force due to breaking waves acts on an upright section, the instantaneous local pressure can rise up to several tens of times the hydrostatic pressure corresponding to the wave height, although the duration time of the impulsive breaking wave force is very short. It is necessary to evaluate the contribution of the impulsive breaking wave force to sliding in terms of the dynamic response, considering deformation of the mound and the subsoil. Goda [137] as well as Takahashi and Shimosako, [138] have carried out calculations of the shear force at the bottom of an upright section using dynamic models. Judging from the results of these calculations and the results of various sliding experiments, it would seem reasonable to take the mean intensity of the wave pressure statically equivalent to the sliding of the upright wall on the mound to be (2.5 – 3.0) $\rho_0 gH$. The impulsive breaking wave force coefficient has been introduced based on the results of sliding experiments with consideration of such dynamic response effects.

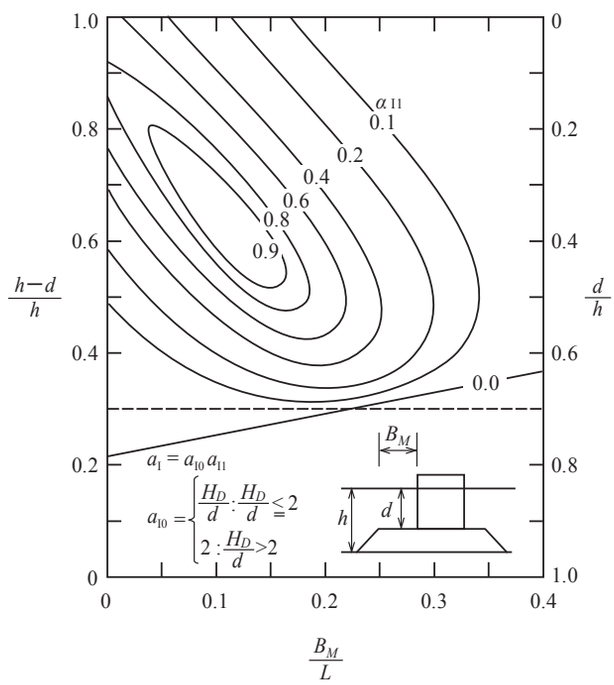


Fig 4.7.6 Impulsive Breaking Wave Pressure Coefficient

(5) Wave Force Acting on Upright Wall Covered with Wave-dissipating Concrete Blocks

① General

If the front of an upright wall is covered with wave-dissipating deformed concrete blocks, the features of wave force acting on the wall varies. The extent of this variation depends on the characteristics of incident waves, along with the crown height and width of the wave-dissipating work, the type of wave-dissipating concrete blocks used, and the composition of the wave-dissipating work including the presence or non-presence of core materials such as rubble. In general, when standing waves act on an upright wall, the variation in wave force on the upright wall is not large. However, when a large impulsive breaking wave force acts, the wave force can be reduced significantly by covering the upright wall with wave-dissipating blocks. Nevertheless, such a reduction in the wave force is only achieved when the wave-dissipating work has a sufficient width and crown height; in particular, it should be noted that if the crown of the wave-dissipating work is below the design water level, the wave-dissipating work often causes an increase in the wave force.

② Wave force calculation formula for upright wall sufficiently covered with wave-dissipating concrete blocks

The wave force acting on an upright wall covered with wave-dissipating concrete blocks varies depending on the composition of the wave-dissipating work, and therefore it should be evaluated using the results of model tests corresponding to the design conditions. However, if the crown elevation of the wave-dissipating work is as high as the crown of the upright wall and the wave-dissipating concrete blocks are sufficiently stable

against the wave actions, the wave force acting on the upright wall may be calculated by applying the extended Goda's formulas. In this method with the standard formula given in **4.7.2 (2) Wave Forces of Standing Waves or Breaking Waves when the Peak of Waves is on the Wall Surface**, the values of η^*, p_1, and p_u given by equations (**4.7.1**), (**4.7.2**), and (**4.7.8**) are used respectively, but it is necessary to assign appropriate values to the wave pressure correction factors λ_1, λ_2 and λ_3 in accordance with the design conditions.

③ Wave pressure correction factors to the extended Goda's formulas

The method using the extended Goda's formulas can be applied for not only breaking waves but also non-breaking waves by assigning appropriate wave pressure correction factors λ_1, λ_2 and λ_3. Studies about the wave pressure correction factor have been carried out by Tanimoto et al.[139), 140)], Takahashi et al.[141)], Sekino, Kakuno et al.[142)], and Tanaka, Abe et al.[143)] They have revealed the following:

(a) Wave-dissipating concrete blocks results a considerable reduction in the breaking wave pressure, and so it is generally acceptable to set the breaking wave pressure correction factor λ_2 to zero.

(b) The larger the wave height, the smaller the correction factor λ_1 for standing wave pressure and the correction factor λ_3 for uplift become.

(c) The larger the ratio of the width of covering block section to the wavelength, the smaller the correction factors λ_1 and λ_3 become.

(d) If even a small portion of the upper part of the upright section is left uncovered, there is a risk of the wave force here becoming an impulsive breaking wave force. Based on such experimental results, Takahashi et al.[141)] have proposed that in general, when the upright wall is sufficiently covered with wave-dissipating concrete blocks, the wave pressure reduction coefficient λ_2 may be taken to be zero, while the values of λ_1 and λ_3 depend primarily on the wave height H (the highest wave height). They have thus proposed the following equations:

$$\lambda_1 = \begin{cases} 1.0 & (H/h \le 0.3) \\ 1.2 - 2(H/h)/3 & (0.3 < H/h \le 0.6) \\ 0.8 & (H/h > 0.6) \end{cases} \qquad (4.7.15)$$

$$\lambda_3 = \lambda_1$$

$$\lambda_2 = 0$$

In the breaker zone, where breakwaters covered with wave-dissipating concrete blocks are generally used, the above equations give $\lambda_1 = \lambda_3 = 0.8$.

④ Wave force on the discontinuous part of wave-dissipating block covering

In those cases where wave-dissipating blocks are placed partially at corners of breakwater alignment, a discontinuous part of a wave-dissipating block covering appears at the end of wave-dissipating works. In cases where the crest height of a wave-dissipating work is lower than the design tide level, care is required since the wave force may increase greatly when it is not armored, and a similar large increase in the wave force may occur also at the discontinuous part of the wave-dissipating block covering.[144)]

Shiomi, Yamamoto, et al.[145)] have conducted a hydraulic experiment for the wave force at the discontinuous part of the wave-dissipating block armoring and examined the following calculation method. The target range for wave force calculation of the discontinuous part is set at as from the slope toe end of the wave-dissipating work to the point where H.W.L. crosses the slope. The armored length is divided into unit lengths l. For each divided water depth of the wave-dissipating work is assumed to be the water depth d on the mound armored work crest, and the wave-dissipating work crest width is assumed to be the mound crest width B_M, shown in **Fig. 4.7.6** and the wave pressure and uplift intensity is calculated by Goda's formulas employed the impulsive breaking wave pressure coefficient α_I) in **4.7.2 (2) Wave Forces of Standing Waves ot Breaking Waves when the Peak of Waves is on the Wall Surface**, and the wave pressure of each divided section is determined. The wave force is calculated such that the mean wave pressure intensity (p_1, p_3, p_4) and the uplift pressure intensity (p_u) of the one caisson act on the entire caisson located in the discontinuous portion. The division length l is determined such that the full wave force over the length of one caisson becomes maximum, but in general it is set at 1/4 to 1/1 of the partition wall interval of the caisson.

⑤ Morihira's formula

The formula proposed by Morihira, et. al.[131)] may be used for the breakwater which is located in surf zone, where there is a significant wave height decrease by the effect of wave breaking, and is covered sufficiently with wave-dissipating concrete blocks.

⑥ Wave force acting on the superstructure of a sloping breakwater covered sufficiently by wave-dissipating blocks

Tanimoto and Kojima [146)] have proposed a calculation equation for the wave pressure correction factor λ for

cases where the foundation ground exists near the still-water surface, and where it is covered sufficiently with wave-dissipating blocks similar to the superstructure of a sloping breakwater.

⑦ Block load due to wave action
Wave force as the direct action of waves and the action due to the leaning of the blocks acts on an upright wall that is covered with wave-dissipating blocks. The latter is called the block load. Research on the block load has been carried out by Hiromoto, Nishijima, et al.[147], Tanaka, Abe, et al.[143], and Takahashi, Tanimoto, et al.[141], and the results have been summarized as follows.

(a) The block load when waves are not acting is small immediately after installation, but increases along with the action of waves, and approaches the constant value. It is possible to consider the same distribution as the earth pressure for that load, but the value differs depending on the wave forces that act on.

(b) The block load during wave action can be ignored in ordinary cases. This is because the upright wall is displaced, albeit slightly, by the action of the waves, and the block load decreases, and becomes almost 0 when the wave height becomes larger. However, in the event that the wave height is small, or when the water depth is large and the block load in still water is large, it can no longer be ignored.

⑧ Impact force of wave-dissipating blocks
Immediately after installation of the blocks or in the event that settlement of the blocks has occurred, when they are subjected to the action of waves in a state where the interlocking between blocks is loose, there are cases where the blocks move due to the waves, and strike the upright wall. In particular, when a wave-dissipating block is large, a powerful impact force acts, and punching sheer failure may occur in the wall surface. Arikawa et al.[148], Yamaguchi et al.[149] have examined this kind of impact force of blocks, and reference can be made to their work.

(6) Wave Force on Sloping-top Caisson Breakwaters

① The wave force on sloping-top caisson breakwaters should be calculated based on the model test results that are suited to the conditions. However, it is possible to use the following calculation equations, if the conduct of the model test is difficult.[150] (See **Fig. 4.7.7**)

$$F_X = F_{SH} + F_V = \lambda'_{SL} F_1 \sin^2\alpha + \lambda_V F_2 \tag{4.7.16}$$

$$F_Z = -F_{SV} + F_U = -\lambda'_{SL} F_1 \sin\alpha\cos\alpha + 0.5 p_u B \tag{4.7.17}$$

$$\lambda'_{SL} = \min\left[\max\left\{1.0, -23(H/L)\tan^{-2}\alpha + 0.46\tan^{-2}\alpha + \sin^{-2}\alpha\right\}, \sin^{-2}\alpha\right] \tag{4.7.18}$$

$$\lambda_V = \min\left[1.0, \max\left\{1.1, 1.1 + 11 d_c/L\right\} - 5.0(H/L)\right] \tag{4.7.19}$$

Here,

F_X : total horizontal wave force acting on the sloping-top breakwater (kN/m)
F_Z : total vertical wave force acting on the sloping-top breakwater (kN/m)
F_{SH} : horizontal component of the wave force acting on the sloping part (kN/m)
F_{SV} : vertical component of the wave force acting on the sloping part (with the upwards direction being positive) (kN/m)
F_V : wave force acting on the upright part (kN/m)
F_U : uplift acting on the bottom surface (kN/m)
F_1 : component corresponding to the sloping part out of the horizontal wave force acting on the upright wall calculated by Goda's formulas (kN/m)
F_2 : component corresponding to the upright part out of the horizontal wave force acting on the upright wall calculated by Goda's formulas (kN/m)
λ_{SL}' : correction factor for the wave force acting on the sloping part
λ_V : correction factor for the wave force acting on the upright part
α: angle of the sloping part (°)
p_U : uplift pressure at the front toe of an ordinary caisson calculated by Goda's formulas
B : caisson width of a sloping-top breakwater (m)
H : wave height (m)
L : wavelength (m)
d_c : height from the still-water surface to the lower end of the slope (with a case where it is located above the still-water surface taken to be positive) (m)

λ_{SL}' is defined by the following three areas.

(a)Where H/L is relatively small

$\lambda_{SL}' = \sin^{-2}\alpha$, that is, $F_{SH} = F_1$, $F_{SV} = F_1 \cdot \tan^{-1}\alpha$,

(b)Where H/L is large

$\lambda_{SL}' = 1.0$, that is, $F_{SH} = F_1 \cdot \sin^2\alpha$, $F_{SV} = F_1 \cdot \sin^2\alpha$,

(c)Where H/L is between (a) and (b)

λ_{SL}' decreases as H/L becomes larger

In addition, with respect to λ_V, $\lambda_V = 1.0$ when H/L is relatively small, and λ_V decreases as H/L becomes larger. However, this wave force calculation equation is applied in the cases where the water depth is relatively deep and the period of the design wave is long, and the value of λ_V should be set at a lower limit of around 0.75.[136] Before this calculation equation was proposed, it had been calculated as $\lambda_{SL}' = \lambda_V = 1.0$ as a convenient and simple method.[151] In this case, the calculated results are somewhat on the safe side in those cases other than when H/L is relatively small.

② Wave force acting on sloping-top caisson breakwaters covered with wave-dissipating concrete blocks

The research of Sato, et al.[152] can be referenced as concerns the wave force acting on sloping-top caisson breakwaters covered with wave-dissipating blocks. In addition, Katayama, et al.[153] have proposed a wave force calculation equation for the semi-submerged type, as when the lower end of the sloping part is under the water surface.

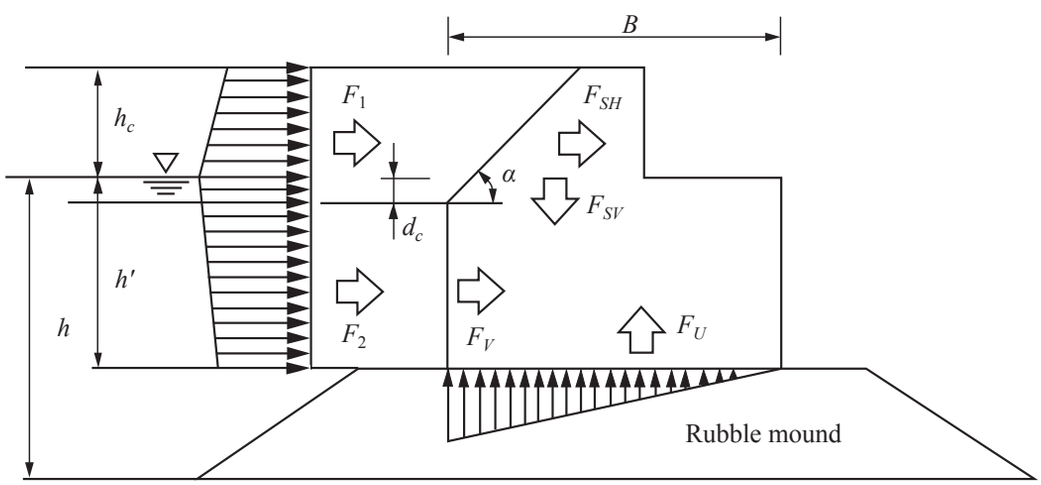


Fig. 4.7.7 Wave Force Acting on Sloping-top Caisson Breakwater

(7) Wave Force Acting on Upright Wave-absorbing Caisson

① General

The wave force acting on an upright wave-absorbing caisson varies in a complex way. Specifically, it varies with the wave characteristics, the water level, the water depth, the topography of sea bottom and the shape of the foundation mound as with the case of an ordinary upright wall, but it also varies with the structure of the wave-dissipating structure. It is thus difficult to designate a general calculation method that can be used in all cases. Consequently, if the calculation method that is sufficiently reliable for the structure in question is not proposed, it is necessary to carry out studies using hydraulic model tests matched to the individual conditions. It is preferable to sufficiently examine not only the wave force to be used in the stability examination but also the wave force acting on structural members. Moreover, it should be noted that the wave force varies significantly according to whether or not the top of wave chamber is covered with a ceiling slab.

② Wave force without a ceiling slab in the wave chamber

The wave force acting on an upright wave-dissipating caisson varies depending on the structural conditions of the wave-dissipating structure, and so it is not possible to calculate this wave force for all general cases involved. Nevertheless, for the ordinary case where there is no ceiling slab in the wave chamber, one can apply the extended Goda's formulas to calculate the wave force. Takahashi, Shimosako, et al.[154] have carried out experiments on a vertical-slit wall caisson, and have presented a method for calculating the wave pressure acting on the slit and rear walls for four representative phases, where the wave pressure given by the Goda's formulas is multiplied by a correction factor λ. They give specific values for the correction factor for the slit and rear walls for each phase. This method can be used to give not only the wave force that is severest in terms of the sliding or overturning of the caisson, but also the wave force that is severest in terms of the performance verification of the elements for each wall. Note, however, that the experiments which form the basis for this calculation method

were conducted under limited structural conditions. Discretion should therefore be exercised in the scope of application for this method.[136)]

③ Simplified method to examine the stability of a wave chamber without a ceiling slab
A simpler form of the Goda's formulas can similarly be applied when examining the stability of a caisson. In this method, it is assumed that the wave pressure acts on the main body of the caisson disregarding the wave-dissipating structure (see **Fig. 4.7.8**), and then the wave force is calculated using η^* obtained using equation **(4.7.1)**, p_1 from equation **(4.7.2)** and p_u from equation **(4.7.8)**, as described in the Goda's formulas in **4.7.2 (2) Wave Force of Standing Waves or Breaking Waves when the Peak of Waves is on the Wall Surface**. In this case, with respect to the wave-dissipating structure, buoyancy of the entire section should be taken into account. With regards the main body of the caisson, on the other hand, buoyancy under the still water should be considered. However, the wave pressure correction factors λ_1, λ_2 and λ_3 should be assigned appropriately according to structural conditions. There are examples of examinations [128)] on the correction factors λ_1 and λ_2 on curved-slit caissons, [155)] perforated-wall caissons and vertical-slit wall caissons; on an average, λ_1=λ_3=1.0 and λ_2=0 can be applied to wave-dissipating caissons.

④ Wave force with a ceiling slab in the wave chamber
When the top of the wave chamber is closed off with provision of a ceiling slab, an impulsive breaking wave force is generated at the instant when the air layer in the upper part of the wave chamber is trapped in by the rise of water surface. It is thus necessary to give consideration to this impulsive breaking wave force in particular with regard to the wave pressure used in the performance verification of structural elements. This impulsive breaking wave force can be reduced by providing suitable air holes. However, it should be noted that if these air holes are too large, the rising water surface will directly strike the ceiling slab without air cushion, meaning that the wave force may actually increase.[156), 157)]

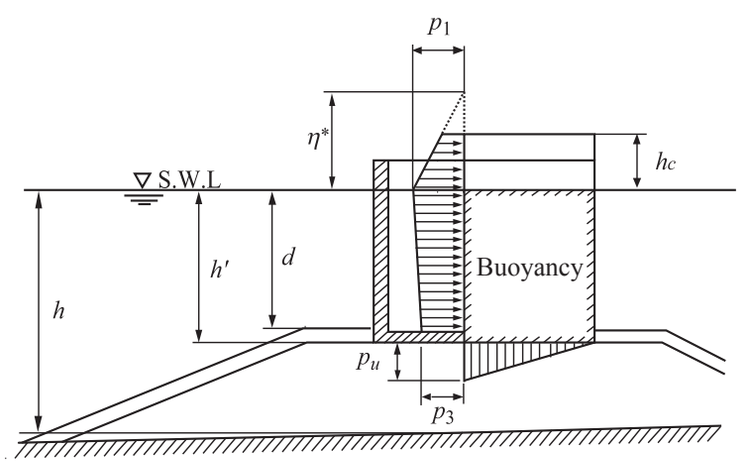


Fig. 4.7.8 Wave Pressure Distribution Employed for Examining Stability
(In case no ceiling slab is installed for wave chamber)

(8) Calculation of Wave Force considering Effect of Alignment of Breakwater

① General
When the alignment of breakwater is discontinuous, the distribution of the wave height along the alignment of breakwater becomes non-uniform due to the effects of wave reflection and diffraction. Ito and Tanimoto [158)] have pointed out that most damaged breakwaters having been struck by storm waves show a pattern of meandering distribution of sliding distance. They have termed this "meandering damage", and pointed out that one of the causes of this type of damage is the differences in the local wave forces induced by the non-uniform wave height distribution. The variation of wave heights along the breakwater is particularly prominent when the breakwater alignment contains a corner that is concave with respect to the direction of wave incidence (see **4.3.4 [3] Transformation of Waves at Concave Corners, near the Heads of Breakwaters, and around Detached Breakwaters**). This should be considered in the calculation of the wave forces.[159), 160)] Variations in wave heights along the breakwater alignment may also occur near the head of the breakwater. In particular, for a detached breakwater that extends over a short length only, diffracted waves from the two ends may cause large variations in wave heights.[161)] These aspects should be considered in the calculation of the wave forces, as necessary.

② Wave force calculation method taking increase in wave height into consideration
Wave force calculation methods that consider the effects of the shape of the breakwater alignment have not

reached to the level of reasonable reliability yet. It is thus preferable to carry out an examination using hydraulic model tests. Nevertheless, there is a good correlation between the increase in the wave height owing to the shape of the breakwater alignment and the increase in the wave force. It is thus acceptable to increase the wave height for the performance verifications in accordance with the extent of the effect of the shape of the breakwater alignment as in equation **(4.7.20)**, and then calculate the wave force based on the standard calculation equation.

$$H_D' = \min\{K_c H_D, K_b\ H_b\} \tag{4.7.20}$$

where

H_D' : wave height to be used in the wave force calculation in consideration of the effect of the shape of breakwater alignment (m)

K_c : coefficient for the increase in wave height due to the effect of the shape of breakwater alignment; $K_c \geq 1.0$

K_{cb} : limit value of increase coefficient for limiting breaker wave; $K_{cb} \fallingdotseq 1.4$

H_D : wave height used in the wave force calculation when the effects of the shape of breakwater alignment are not considered (m)

H_b : breaking wave height at the offshore location with the distance of 5 times the significant wave height of progressive waves from the breakwater (m)

The wave height increase coefficient *Kc* in equation **(4.7.20)** is generally expressed as in equation **(4.7.21)**. It can be appropriately determined based on the distribution of the standing wave height (see **4.3.4 [3] Transformation of Waves at Concave Corners, near the Heads of Breakwaters, and around Detached Breakwaters**) along the alignment of breakwater as determined under the condition that the waves do not break.

$$K_c = H_s / \{H_I (1 + K_R)\} \tag{4.7.21}$$

where

H_s : standing wave height along the wall of breakwater (m)

H_I : incident wave height (m)

K_R : reflection coefficient for the breakwater in question

If the waves are treated as being of regular waves, then the coefficient for wave height increase varies considerably along the breakwater. Moreover, the height increase coefficient is very sensitive to the period of the incident waves and the direction of incidence. It is thus reasonable to consider the irregularity of the period and the direction of incident waves. It should be noted that the value of *Kc* obtained in this way varies along the breakwater and that there may be regions where $Kc < 1.0$. However, the wave height to be used in the performance verifications must not be reduced by applying $Kc < 1.0$.

The second term in the braces { } on the right-hand side of equation **(4.7.20)** was introduced in view of the fact that the increase in wave height from the effects of the shape of the breakwater alignment is limited by the water depth. The height of limiting breaker wave H_b can be taken to be the highest wave height $H_{\max}$ in **4.3.6 Wave Breaking** when there is an upright wall in a region where the highest wave would be affected by breaking waves. If it is further offshore, values provided in the breaker index diagram (see **Fig. 4.3.15**) in **4.3.6 Wave Breaking** can be applied. The limit value K_{cb} of increase coefficient for limiting breaker waves has not been clarified in details. Nevertheless, it may be considered to be about 1.4 based on experimental results up to the present time.

(9) Wave Force acting on Upright Wall in Abrupt Depth Change

For an upright wall located in a place where the water depth changes abruptly owing to the presence of reefs and others, impulsive breaking wave force or wave force after breaking act on the upright wall in accordance with the conditions such as the location of the breakwater. It is preferable to calculate the wave force acting on the upright wall based on hydraulic model tests, by taking the rapid transformation of waves into consideration. Ito et al.[162] have carried out experiments on the wave force acting on an upright wall located on or behind a reef where the water depth is more-or-less uniform, with the offshore slope of the shoal having a gradient of about 1/10.

(10) Wave Force acting on Upright Wall Located Considerably Toward the Landside from the Breaker Line

① Wave force acting on an upright wall located at the seaside of the shoreline

(a) General

When the changes in wave force due to the installation depth of an upright wall on a uniform slope are examined under conditions of the specified waves, in general the wave force reaches a maximum when the upright wall is located somewhat to the shore side from the breaker point as the progressive wave, and the wave forces decreases as the installation depth becomes shallower than that. Given such a tendency, it is

considered that the wave force due to the smaller waves that break somewhat at the offing of the upright wall is greater than wave force after the breaking of a large wave that breaks considerably toward the offing from the upright wall, when it has a certain degree of water depth.

Goda's formula, which are stipulated in **4.7.2 (2) Wave Force of Standing Waves or Breaking Waves when the Peak of Waves is on the Wall Surface**, provide a wave force based on the waves breaking somewhat in the offing of such an upright wall. However, in those places where the water depth in the vicinity of the shoreline is shallow, not only does the breaking wave height vary greatly depending on the changes in water level due to surf beat and so on, but also the breaking wave force varies greatly due to the sea bottom gradient, the wave steepness of offshore waves and the irregularity of the waves, so it is not appropriate to employ Goda's formulas, and it should be calculated with an equation suited to the conditions or the results of a hydraulic model test. In addition, the fact that the water depth itself changes due to the littoral drift, or that the effects of storm surge are great, should also be taken into consideration.

(b) Calculation method of wave force acting on an upright wall at the seaward side of shoreline

A number of different wave force formulas have been proposed for upright walls near the shoreline. It should be necessary to carry out an appropriate wave force calculation in line with the design conditions. Very roughly speaking, the standard formula in **4.7.2 (2) Wave Forces of Standing Waves and Breaking Waves when the Peak of Waves is on the Wall Surface** are applicable in the regions where the seabed slope is mild and the water is relatively deep. The formula of Tominaga and Kutsumi [163] is applicable in the regions near the shoreline. The formula of Hom-ma, Horikawa and Hase [130] is applicable in the regions where the seabed slope is steep and the water is of intermediate depth.

When applying Goda's formula to the places where the water depth is less than one half the equivalent deepwater wave height, it may be preferable to use the values for the wavelength and wave height at the water depth equal to one half the equivalent deepwater wave height in the calculation.

② Wave force acting on an upright wall located at the land side of the shoreline

(a) General

Since the wave force acting on an upright wall located at the land side of the shoreline varies greatly depending on the rise in the water level due to surf beat or the runup of the waves, it should be calculated with an equation suited to the conditions or the results of a hydraulic model test. In addition, the fact that the topology in the vicinity of the shoreline changes due to the littoral drift, or that the effects of storm surge are great, should also be taken into consideration.

(b) Calculation method of wave force acting on an upright wall at the landward side of shoreline

For an upright wall located on the landward side of the shoreline, the formula by the US Army Coastal Engineering Research Center (CERC) [164] is available. Moreover, one may refer to the research that has been carried out by Tominaga and Kutsumi on the wave force acting on an upright wall located on the landward side of the shoreline.

4.7.3 Wave Force Acting on Submersed Members and Isolated Structures

(1) Wave Force Acting on Submersed Members [165]

① Morison's Formula

(a) General

Structural members such as piles that have a small diameter relative to the wavelength hardly disturb the propagation of waves. The wave force acting on such members can be obtained using the Morison's formula as shown in equation **(4.7.22)**, in which the wave force is expressed as the sum of a drag force that is proportional to the square of the velocity of the water particles and an inertia force that is proportional to the acceleration.

$$\vec{f}_n = \frac{1}{2} C_D \rho_0 |\vec{u}_n| \vec{u}_n D \Delta S + C_M \rho_0 \vec{a}_n A \Delta S \tag{4.7.22}$$

where

$\vec{f}_n$: force that acts on a small length ΔS (m) in the axial direction of the member, where the direction of this force lies in the plane containing the member axis and the direction of motion of the water particles and is perpendicular to the member axis (kN)

$\vec{u}_n$, $\vec{a}_n$: components of the water particle velocity (m/s) and acceleration (m/s^2), respectively, in the direction perpendicular to the member axis that lies within the plane containing the member axis and the direction of motion of the water particles (i.e., the same direction as $\vec{f}_n$) (these components are for incident waves that are not disturbed by the presence of member)

$|\vec{u}_n|$: absolute value of $\vec{u}_n$ (m/s)

C_D : drag coefficient

C_M : inertia coefficient
D : width of the member in the direction perpendicular to the member axis as viewed from the direction of $\vec{f}_n$ (m)
A : cross-sectional area of the member along a plane perpendicular to member axis (m^2)
ρ_0 : density of seawater (normally 1.03t/m^3)

Equation (**4.7.22**) is a generalized form of the equation presented by Morison et al.[166], to give the wave force acting on a section of a very small length ΔS of a member orientated in any given direction. The arrows on top of symbols indicate that the force, velocity and acceleration are the components in the direction perpendicular to the member. The first term on the right-hand side represents the drag force, while the second term represents the inertia force. The water particle velocity and acceleration components in the equation both vary in time and space. It is preferable to pay sufficient attention to these variations, and to examine the distribution of the wave force that is severest to the member or structure in question. It should be also necessary to appropriately evaluate the drag coefficient and the inertia coefficient by model tests or field measurement results.

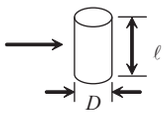
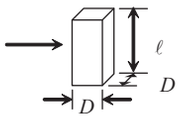
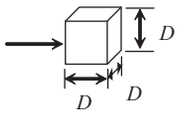
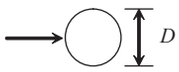
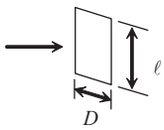
(b) Water particle velocity and acceleration components
The components of water particle velocity and acceleration ($\vec{u}_n, \vec{a}_n$) in equation (**4.7.22**) represent the component of the water particle motion at the center axis of the member. These components are in the direction perpendicular to the member axis, and are evaluated under the assumption that waves are not disturbed by the presence of the structure in question. When calculating the wave force, it is necessary to estimate these components as accurate as possible, based on either experimental data or theoretical prediction. In particular, the water particle velocity component contributes to the wave force with its square, meaning that when the wave height is large, an approximation using small amplitude wave theory becomes insufficient to yield reliable estimate. Moreover, when the member extends above the water level, it is necessary to give sufficient consideration to the range over which the wave force acts, i.e., the elevation of wave crest. When calculating these terms using theoretical values, it is preferable to use the finite amplitude wave theory that agrees with the characteristics of the design waves, based on **4.1 Basic Matters Relating to Waves**. Note also that it is necessary to take full account of wave irregularity with regard to the wave height and period used in the wave force calculation, and to study the wave characteristics that are severest to the safety of member or structure in question. In general, the highest wave height and the significant wave period may be used in the analysis for rigid structures.

(c) Drag coefficient C_D
In general, the drag coefficient for steady flow can be used as the drag coefficient C_D for wave force. Note however that the drag coefficient varies with the shape of the member, the surface roughness, the Reynolds number *Re*, and the separation distance between neighboring members. It also varies with the Keulegan-Carpenter number (*KC* number) because the flow is of oscillating nature. It is necessary to consider these conditions when setting the value of drag coefficient. For a circular cylindrical member, it is standard to set $C_D = 1.0$ if the finite amplitude properties of the waves are fully considered. For an unmanned structure, a lower value may be used if its value is based on the results of model tests matched to the conditions. Even in this case, however, C_D should not be set below 0.7. Note also that when estimating the water particle velocity by an approximate equation, it is preferable to use a value for the drag coefficient that has been adjusted for the estimation error in the water particle velocity. If the velocity of the water particle motion can be calculated accurately, drag coefficient values for steady flow in **Table 6.5.1** in **6.5 Fluid Force due to Current** may be used.

(d) Inertia coefficient C_M
The value by the small amplitude wave theory may be used for the inertia coefficient C_M. Note, however, that the inertia coefficient varies with the shape of the member and other factors such as the Reynolds number, the *KC* number, the surface roughness, and the separation distance between neighboring members. The value of the inertia coefficient should be set appropriately in line with the given conditions.

When the diameter of the object in question is no more than 1/10 of the wavelength, it is standard to use the value listed in **Table 4.7.1** for the inertia coefficient C_M However, when estimating the water particle acceleration by an approximate equation, it is necessary to adjust the value of C_M. for the error in the estimate of water particle acceleration. The value of inertia coefficient shown here is mostly from the study by Stelson and Mavis.[167] According to the experiments of Hamada, Mitsuyasu et al.[168], the mass coefficient for a cube is in the range of 1.4 to 2.3.

Table 4.7.1 Inertia Coefficient

Shape of the object	Basic volume	Inertia coefficient
Cylinder 	$\frac{\pi}{4}D^2\ell$	2.0 ($\ell>D$)
Regular prism 	$D^2\ell$	2.19 ($\ell>D$)
Cube 	D^3	1.67
Sphere 	$\frac{\pi D^3}{6}$	1.5
Flat plate 	$\frac{\pi}{4}D^2\ell$	When D/ℓ =1, 0.61 When D/ℓ =2, 0.85 When $D/\ell=\infty$, 1.00

(e) Experimental values for drag coefficient and inertia coefficient of cylinder
There are many experimental values for the drag coefficient and inertia coefficient of a vertical cylinder; for example, those of Keulegan and Carpenter, [169] Sarpkaya, [170), 171), 172] Goda, [173] Yamaguchi, [174] Nakamura, [175] Chakrabarti, [176), 177] and Koderayama and Tashiro.[178] There are many variations between these values. However, there is not sufficient data in the region of high Reynolds number, which is subject to the actual performance verification. Oda [179] has produced a summary of these researches which may be referred to.

(f) Effects of neighboring members
When structural members neighbor one another, the values of the drag coefficient and inertia coefficient vary due to the effects of the other structural members. According to experiments on cylindrical columns, the drag coefficient increases in the event that two columns are arranged in a row perpendicular to in the direction of the flow, but it has been known that if the net space between the columns (s) is at least 2.5 times its diameter (D), its effects are small. In addition, in the event that they are arranged in a row in the direction of the flow, the drag coefficient for the column in back exhibits a tendency to decrease over a considerable range (s/D = about 9). However, it cannot be recognized that its effects have been adequately solved yet, and in general it is better not to consider this as a decrease in the drag coefficient due to the neighboring effect.

In addition, the value of the inertia coefficient has been calculated by diffraction theory, and an increase or decrease has been known compared with the case of a single column depending on the values of s/D and D/L, [182] but when D/L is small its effects are small. Nakamura and Abe [183] have investigated experimentally the increase in the inertia coefficient in a range of $D/L < 0.1$, and have pointed out that although the results are scattered the upper limit of the coefficient value is extremely large in the vicinity of $D/L = 2$ to 3, and it is better to avoid a situation where the interval between the two columns matches such conditions.

(g) Facilities composed of many structural members
The wave force that acts on an entire facility composed of upright columns, slanted members and/or horizontal members is calculated by equation **(4.7.22)** considering the phase difference of the wave force acting on each structural member, and by compound the vector sums of these. In the case of facilities composed of many of structural members, there is a risk that the whole might collapse due to the failure of one point in the structural member, so the distribution of the wave force that is most severe for the individual structural members and the entire facility should be considered in particular.

(h) Resonance with waves and random wave force
In the event that the rigidity of the facilities is low, and the natural frequency period is long, it is preferable to consider the effects of the dynamic response on the wave force that acts periodically. The wave force in this case may be calculated for the temporal changes of $\bar{u}_n, \bar{a}_n$ based on equation **(4.7.22)**. However, since only the specific dynamic effects are reflected in the examination for waves with a constant period, it is reasonable

to view this as the continuous action of random waves. When calculating the wave force for random waves, suitable measures may be devised for the way to provide the height of the wave crown height and the drag coefficient, and the water particle movement component may be calculated based on small amplitude wave theory.

② Wave force when breaking waves act

When breaking waves act on facilities on a steep sea bottom surface, there are cases when an impulsive wave force similar to the impulsive breaking wave pressure that acts on upright walls acts in addition to the drag and inertia forces given by equation **(4.7.22)**. Since the response characteristics of the facilities become the dominant effect factor for such an impulsive action, not only a calculation of the wave force but also an examination that includes the behavior of the entire facility as well as the structural members should be carried out.[184), 185)]

③ Uplift

In addition to the drag and inertia forces of equation (**4.7.22**), wave force acting on submerged members is the uplift acting in the direction perpendicular to the plane containing the member axis and the direction of the water particle motion. In general, it is acceptable to ignore this uplift, but it is necessary to pay attention to the fact that the uplift may become a problem for horizontal members that are placed near to the seabed.[186), 187), 188), 189), 190), 191)] Moreover, for long and thin members, it is necessary to pay attention to the fact that the uplift may induce vibrations.

④ Wave force due to random waves

Of the wave force components acting on structural members in the sea, the inertia force is linear, so the spectrum of the wave force can be calculated easily from the spectrum of the waves, but when the drag force is included this becomes difficult owing to its nonlinearity. Borgman [192)] has introduced a theoretical equation for the wave force spectrum that includes drag force based on probability theory. The first approximation of this drag force corresponds to something where the nonlinear drag force is made linear in a form in which the *rms* value of the water particle speed is incorporated in the coefficient, and this is employed occasionally in spectrum analysis of on-site observational data and other cases. In addition, Hino [193)] has introduced a theory of a case where waves and a uniform flow co-exist by using the characteristic function method. A simulation method where the random wave forms and water particle movement are simulated based on a prescribed wave spectrum, and its time series is inputted and the wave force is calculated, is also being employed commonly as a method for the statistical nature of random waves including the nonlinear drag and the dynamic response of the facilities. Borgman [194)] has explained this method, and there is the calculation example of Ito et al.[195)]. These are simulations based on linear theory, but recently nonlinear simulation calculations that consider everything up to the second order interference terms between component waves have also been carried out, [196)] and in addition nonlinear simulation calculations of multi directional random waves has also been tried.[197)] As for the probability distribution of the wave force, the wave height exhibits a Rayleigh distribution, whereas its local maximum value may become considerably larger than the Rayleigh type owing to the nonlinearity of the drag. Tickell-Elwany [198)] has calculated the theoretical value of the wave force distribution based on three-dimensional random waves. In addition, Kimura et al.[199)] has calculated the probability distribution of the wave force acting on a single cylindrical column based on the joint distribution of the wave height and period of random waves, and shown a method for calculating the anticipated values for the maximum wave force.

⑤ Equation for calculating the breaking wave force acting on slanted columns

Tanimoto, Takahashi, et al.[185)] have developed the research of Goda et al.[184)], and have proposed a method for calculating the breaking wave force acting on cylindrical columns based on experimental results. The calculation of the impulsive breaking wave force acting on upright cylindrical columns or slanted cylindrical columns installed on a sea bottom with a uniform slope may be carried out based on this method. In the experiment both regular and random waves were employed, and were carried out with a cylindrical column with a D/h = 1/5, for gradients i = 1/100 and 1/30, and θ = -30°, -15°, 0°, +15° and +30°. The position of the impulsive wave force that acts and the changes over time can be calculated by the equation, and the response of the cylindrical column member to the impulsive wave force can also be calculated.

⑥ Breaking wave force acting on small diameter cylindrical columns on a reef

Goda et al.[200)] have proposed a method for calculating the breaking wave force that acts on upright cylindrical columns on reefs, where the water depth changes suddenly, and it is possible to carry out calculations of the wave force based on this method for waves like those that break on the slope of reefs.

⑦ Effects of multi-directionality of waves

As the multi-directionality of waves becomes stronger, the components of the wave force other than the principal direction of the wave becomes larger. Therefore, the multi directional dispersion of the waves should be considered in facilities constructed in deep waters where the multi-directionality is strong.

(2) Wave Force Acting on Large Isolated Structures

① General

The wave force acting on a large isolated structure whose dimensions are comparable to the wavelength can be calculated using the velocity potential, because it is generally possible to ignore the drag force. In particular, for structures of a simple shape, analytical solutions obtained by diffraction theory are available. However, it is necessary to calculate the breaking wave force by hydraulic model tests if there is a possibility of breaking wave force exerted on structure.

② Diffraction theory

MacCamy -Fuchs [202)] have determined the velocity potential of waves around an upright cylindrical column of large diameter using diffraction theory, and calculated the wave force from the water pressure distribution at the surface of cylinder. Goda and Yoshimura [203)] have applied diffraction theory to an upright elliptic cylinder, and presented their results in terms of the inertia coefficient C_M. Yamaguchi [204)] has examined the effect of the wave nonlinearity on the wave force acting on an upright cylindrical column of large diameter by nonlinear diffraction theory, and pointed out that it is necessary to consider these effects when the water is shallow.

③ Isolated structure of arbitrary shape

For a structure that is complex in shape, it is difficult to obtain the wave force analytically, and so it is necessary to carry out a numerical calculation. Various methods are available, such as integral equation methods (see **4.9 Action on Floating Body and its Motions**).

4.7.4 Wave Force Acting on Structures near the Water Surface

(1) Uplift Acting on Horizontal Plates near the Water Surface

① General

In the case of facilities near still water surface, such as the superstructure of piled piers on pile-type dolphins, and in particular those facilities that are roughly parallel to the water surface, there is a risk that a rising wave surface will strike on the bottom surface of the facilities and an impulsive wave force (hereinafter, “uplift”) will act on. In particular, it becomes a large impulsive force when the wave height is large and the clearance with the still water surface is small. In addition, in a case where there is a reflecting wall at the rear as in the case of open type wharf, and the waves become standing waves and act on this, the impulsive uplift should be calculated by a suitable method such as a hydraulic model test. Due attention should also be paid to the fact that ordinary uplift that is not an impulsive force also acts on the bottom surface of such structures, in addition to the impulsive uplift.

② Characteristics of impulsive uplift

If the bottom surface of the plate is flat, the impulsive uplift acting on a horizontal plate near the still water surface level varies with the impact velocity of the wave surface and the angle between the wave surface and the plate. As shown in **Fig. 4.7.9 (a)**, when there is an angle between the wave surface and the plate, the wave surface runs along the bottom surface of the plate and the wave pressure distribution becomes as shown there. The distinct feature of the wave pressure in this case is its rapid rise in time. On the other hand, when the angle between the wave front and the plate is close to 0, as shown in **Fig. 4.7.9 (b)**, a layer of air is trapped between the wave surface and the plate, and compression of this layer of air results in the almost uniform wave pressure distribution. The distinct feature of the wave pressure in this case is its oscillation in time with having a short period damping vibration.

In case of a piled pier with a deck plate supported by horizontal beams, the wave surface is disturbed by the beams, and the uplift becomes of complex nature. With beams, a layer of trapped air is often formed and this layer of air is compressed by the uprising wave surface. It is thus necessary to give consideration to the change in the uplift with respect to the shape of the bottom face of the horizontal plate. The shape of the impacting wave surface varies greatly according to the condition whether the wave is progressive or standing in nature. With standing waves, the shape of the impacting wave front varies with the distance between the position of wave reflection and the horizontal plate. It is thus necessary to consider such differences.

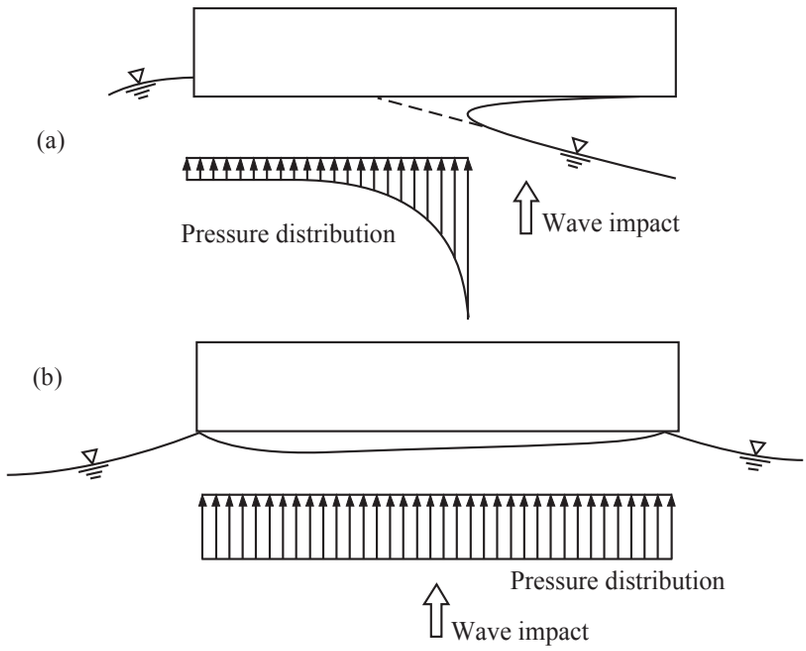


Fig. 4.7.9 Impact between Wave Front and Horizontal Plate

③ Calculation of uplift from standing waves

(a) Uplift acting on horizontal plate with flat bottom surface

Goda [112] considered the uplift acting on a horizontal plate as being the force results in the sudden change in the momentum of wave by its impact on the plate. Using von Karman's theory, he obtained the following formulas for calculating the uplift of standing waves acting on a horizontal plate.

$$P = \zeta \frac{\rho_0 g}{4} HLB \tanh \frac{2\pi h}{L}\left(\frac{H}{s'} - \frac{s'}{H}\right) \tag{4.7.23}$$

$$s' = s - \pi \frac{H^2}{L} \coth \frac{2\pi h}{L} \tag{4.7.24}$$

where

P : total uplift (kN)
ζ : correction factor
$\rho_0 g$: unit weight of seawater (kN/m^3)
H : wave height of progressive waves, generally the highest wave height H_{max} (m)
L : wavelength of progressive waves (m)
B : width of plate (m)
H : water depth (m)
S : clearance of the plate above the still water surface (m)
s' : clearance of the plate above the level corresponding to the middle of the wave crest and trough (m)

It is necessary to pay attention to the fact that the uplift in the above equations does not depend on the length of the horizontal plate.

The impact force has the magnitude given by the above equations and takes the form of a pulse that lasts for a time τ from the moment of the impact, that is given as follows:

$$\tau = \frac{\pi T \ell^2}{L^2} \frac{s'}{\sqrt{H^2 - s'^2}} \tag{4.7.25}$$

Where T is the wave period and ℓ is the length of the horizontal plate. Provided the length of the horizontal plate is sufficiently short compared with the wavelength L and the bottom surface of the horizontal plate is flat, equation **(4.7.23)** well represents the features of the uplift well with simple equation. Comparing calculated

values with $\zeta = 1.0$ to experimental values, agreement is relatively good provided H/s' is no more than 2.

Tanimoto et al.[205] have proposed another method for calculating the uplift acting on horizontal plate based on Wagner's theory. With this calculation method, the angle of contact β between the wave surface and the horizontal plate as well as the impact velocity V_n are given by Stokes' third order wave theory, making it possible to obtain the spatial distribution of the impact pressure and its change over time. Note however that the use of Stokes' third order wave theory makes the calculation rather complex. This calculation method is intended for use when the bottom face of the horizontal plate is flat. It cannot be applied directly to structures of complicated shape such as an ordinary piled pier that have beams under the floor slab; the impact between the wave surface and the floor slab is disturbed by the beams. In general, the presence of beams causes air to become trapped in and the wave surface to be disturbed, the result being that the impact force is less than for a horizontal plate with a flat surface. Accordingly, the value obtained from this calculation method may be considered as being the upper limit of the uplift for an ordinary piled pier.

(b) Uplift Acting on Piled Pier

Ito and Takeda [206] have conducted scale model tests of piled pier to obtain the uplift acting on an access bridge, and it's a vibration threshold weight and a falling threshold weight. The experimental conditions were the wave height up to 40 cm, a period of 1.0 s and 2.4 s, and a water depth of 56 cm and 60 cm. According to the measurement records of wave pressure gauges attached to the access bridge, the peak value of the uplift varied considerably from wave to wave even under the same conditions. Nevertheless, the mean of these peak values is given approximately by the following equation **(4.7.26)**:

$$p_k = \rho_0 g(8H - 4.5S) \tag{4.7.26}$$

where

p_k : characteristic value of the mean peak value of the intensity of uplift (kN/m^2)
$\rho_0 g$: unit weight of seawater (kN/m^3)
H : incident wave height (m), (H_{max})
S : distance from the water level to the underside of access bridge (m)

Note however that the peak value of the intensity of the uplift given by equation **(4.7.26)** acts only for an extremely short time, and that the phase of this uplift varies from place to place. This means that even if the intensity of the uplift p exceeds the self weight q i.e., the weight per unit area (kN/m^2) of the access bridge, the bridge will not necessarily move or fall down immediately. Based on this perspective, Ito and Takeda have obtained the threshold weight at which the access bridge vibrate and that at which the deck slab falls down. For waves of period 2.4 s, the relationship between the vibration threshold weight and the wave height is given below:

$$q = \rho_0 g(1.6H - 0.9S) \tag{4.7.27}$$

The vibration threshold weight given by equation **(4.7.27)** is one fifth of the intensity of the uplift as given by equation **(4.7.26)**. The falling threshold weight was found to be 1/2 to 1/3 of the vibration threshold weight.

In these access bridge experiments, Ito and Takeda also tested the access bridge with holes or slits of various sizes, and investigated how the threshold weights changed when the void ratio was changed. In general, the change in the vibration threshold weight by the void ratio is only slight compared to access bridges without holes, when the void ratio is small i.e., around 1%, air escapes easily and the water surface strikes the access bridge impulsively. The falling threshold weight, on the other hand, drops noticeably when the void ratio exceeds 20%. Note that the bridge weight referred to here is the weight per unit area of the substantial part i.e., the weight per unit area excluding the voids. In this way, since there is little change to the vibrating threshold weight, namely the stable weight per unit area of the substantial part of the access bridge, the weight of an entire surface area can be reduced by boring holes. What is more, the falling threshold weight decreases with the increase in the void rate. From these two reasons, it can be concluded that it is best to raise the void rate.

Furthermore, Ito and Takeda [206] have attached a strain gauge to the deck slab of the model of piled pier and measured the stress. Based on their results, they proposed the following equation for the equivalent static load (kN/m^2) assumed to act with uniform distribution on the deck slab.

$$p_k = 4\rho_0 gH \tag{4.7.28}$$

Note however that the value given by this equation corresponds to the upper limit of the experimental values and should thus be considered corresponding to the case that the distance s from the water level to the underside of the superstructure is almost 0. The equivalent static load given by equation **(4.7.28)** is generally lower than the uplift acting on a horizontal plate with a flat bottom face. It is considered that this is partly because the beams disturb the impacting wave front and cause air to become trapped in. Experimental research into the uplift acting on a piled pier has also been carried out by Murota and Furudoi, [207] Nagai and Kubo et al.[208], Horikawa and Nakao et al.[209], and Sawaragi and Nochino.[210]

④ Calculation of uplift by progressive waves

(a) Uplift acting on horizontal plate with flat bottom surface
An impulsive uplift also acts when progressive waves act on a horizontal plate that is fixed near to the still water level. Tanimoto and Takahashi et al.[211] have proposed a method for calculating this impulsive uplift, based on the same theory that was used for impulsive uplift by standing waves.

(b) Uplift acting on superstructure of detached pier

1) Ito and Takeda [206] have also carried out studies on the uplift of progressive waves acting on a detached pier. Specifically, they measured the stress occurring in the deck slabs of a detached pier model. Based on the upper limits of their experimental results, they proposed the following equation for the uniformly distributed equivalent static load.

$$p_k = 2\rho_0 gH \tag{4.7.29}$$

2) Allsop and Cuomo, et al.[212], [213], [214], [215] have undertaken a systematic examination of the uplift due to progressive waves that act on detached pier by model tests based on random waves and theoretical analysis. They have proposed the following calculation equations concerning the ordinary uplift that is not an impulsive force.

$$\frac{F_{qs}}{F^*} = a\left(\frac{\eta_{\max} - c_\ell}{H_s}\right)^{-b} \tag{4.7.30}$$

$$F^* = b_w b_l p_2 \tag{4.7.31}$$

$$p_2 = (\eta_{\max} - c_\ell)\rho g \tag{4.7.32}$$

Here,

F_{qs} : uplift that is not an impulsive force (equivalent to the maximum value in the wave group) (kN)
F^* : standard wave force (kN)
a, b : coefficient dependent on the structural member
$\eta_{\max}$: maximum rise in water level when the maximum wave height $H_{\max}$ is acting (m)
c_l : clearance from the still water surface (m)
H_s : significant wave height (m)
b_w : width of horizontal plate or beam (m)
b_l : length of horizontal plate or beam (m)
p_2 : equivalent still water pressure by the action on the lower surface of horizontal plate due to a rise in the water level (kN/m^2)
$\rho_0 g$: unit weight of seawater (kN/m^3)

According to the experimental results, the value of the coefficient a, b are 0.82 and 0.61 in the case of horizontal plates and beams outside a harbor, 0.71 and 0.71 in the case of horizontal plates inside a harbor, and 0.82 and 0.66 in the case of beams inside a harbor, respectively. It is necessary to calculate the maximum rise in water level $\eta_{\max}$ by appropriate theoretical analysis. In addition, the effects of nonlinearity become greater as the water depth becomes shallower, and the proportion of the maximum rise in the water level relative to the wave height becomes higher. Owing to this, even if the clearance and the design wave height are the same, it is necessary to pay attention to the fact that not only does the uplift becomes relatively great when the water depth is shallow, but also that the frequency with which the uplift acts increases.[215]

On the other hand, a comparison with the uplift has been carried out for the impulsive force,[214] and according to the experimental results it is 2.0 to 2.4 times the ordinary uplift in the case of horizontal

plates outside the harbor, 2.0 to 2.9 times this in the case of beams outside the harbor, 1.7 to 2.3 times this for horizontal plates inside the harbor, and 1.9 to 2.6 times this for horizontal beams inside the harbor. However, it is necessary to pay attention to the fact that although the action time is short as described above, the variance due to conditions is great.

4.8 Design Wave Conditions

4.8.1 Setting of the Design Wave Conditions for Verification of Stability of Facilities and the Ultimate Limit State of Structural Members

(1) General

During the performance verification of the facilities of ports, it is necessary to set appropriately the wave conditions such as the wave height, period, wave direction, as the design conditions. These wave conditions are preferably set by statistical analysis based on long-term observational data, but in the cases where the observation data are inadequate it is common to supplement the data by wave hindcasting.[219), 220), 221)]

The waves for the verification of stability of the facilities and the ultimate limit state of structural members are generally the probabilistic waves whose return period is 50 years, for facilities whose design working lifetime is 50 years. This is the return period of waves that is generally taken into consideration in conventional design, and conventional design is followed in order to provide for continuity with the philosophy of conventional design methods and to avoid confusion in practical design work. Owing to this, the return period may be established appropriately by taking into consideration the design working lifetime and degree of importance of objective facilities, as well as the natural conditions of objective location.

(2) Design Waves at External Values

As far as the abnormal wave characteristics that are employed for the examination of the stability of facilities are concerned, it is preferable to carry out statistical treatment for the peak waves, and to express this as the return wave height.

(3) Statistical Treatment of External Waves

The wave height relative to abnormal waves that are the object of design is generally expressed as the return wave height in respect of the return period for peak waves, from the long-term data i.e. a period of at least 30 years as a standard. Since the number of locations for which it is possible to utilize observational data over the long term is extremely small, wave hindcasting results are generally used instead.

Peak waves, which are the hindcasting data for return wave height, refer to waves, in general, significant waves, when the wave height reaches the maximum in the process where the waves develop and attenuate under one certain meteorological condition, and it is assumed that the peak waves that are sampled are statistically independent from each other. During hindcasting of the return wave height, there are cases where the time series of data where the peak wave height is at least a certain designated value are used in the subject duration, and cases where the maximum value of the peak wave height is calculated each year, and the data for this annual maximum wave is used. As the mother distribution function of the return wave height is unknown in general in either case, the Gumbel distribution, the Weibull distribution, or some other distribution function is applied. The function form most suited to the data is found, and the return wave height in respect of the required return period, for example 50 years or 100 years, etc., is estimated with that estimation equations.

Since the accuracy of such estimated values is dependant more on the accuracy of the data used than the method of statistical treatment, in the event that the data for the peak waves is prepared by wave hindcasting, care should be paid so that appropriate selection of the hindcasting method and verification based on the observed values of the hindcasting results should be applied. Moreover, the relationship between the wave height and the period are plotted for the data for the peak waves, which is the hindcasting data for the return wave height, and the period corresponding to the return wave height is determined as appropriate based on the correlation of these data.

(4) Process in the Statistical Treatment of External Waves

During the statistical treatment, the wave height is arranged in order of size, and the non-exceedance probability for each wave height value is calculated.

Now, assuming that the number of data is N, and the number mth wave height from the larger side is $x_{m,N}$, the probability F_m that the wave height does not exceed $x_{m,N}$ is calculated by the following equation.

$$F_m = 1 - \frac{m - \alpha}{N + \beta} \tag{4.8.1}$$

The values for each probability distribution function shown in **Table 4.8.1** are employed for α and β in the above equation. Since Gringorten has calculated the values for the Gumbel distribution, it has been set so that the effects of the statistical variance of the data are minimized with the non-exceedance probability F corresponding to the anticipated value of the order statistics x_m. Petruaskas and Aagaard [223)] have calculated the values for the

Weibull distribution based on the same viewpoint.

In hydrological statistics, the Gumbel distribution (double exponential distribution), the logarithmic peak value distribution or the square root exponential type maximum value distribution are employed as the distribution functions. Since collection of data over the long term has not been obtained for the peak values of wave height, it is not known clearly what sort of distribution function this accords with. Reference 224) can be referred as materials relating to the peak values for wave height.

Table 4.8.1 Parameters for Non-exceedance Probability Calculation of Abnormal Waves

Distribution function	α	β
Gumbel distribution	0.44	0.12
Weibull distribution (k = 0.75)	0.54	0.64
Same as above (k = 0.85)	0.51	0.59
Same as above (k = 1.0)	0.48	0.53
Same as above (k = 1.1)	0.46	0.50
Same as above (k = 1.25)	0.44	0.47
Same as above (k = 1.5)	0.42	0.42
Same as above (k = 2.0)	0.39	0.37

(5) Proposed Example of Fitted Distribution Functions

① Following Petruaskas and Aagaard, in Reference 225), Goda has proposed a method in which eight kinds of distribution function wherein k = 0.75, 0.85, 1.0. 1.1, 1.25, 1.5 and 2.0 are applied in the Gumbel distribution in equation **(4.8.2)** and the Weibull distribution in equation **(4.8.3)**, and the function that accords the most with the data among these is selected with the correlation coefficient.

$$F(x) = \exp\left[-\exp\left\{-\left(\frac{x-B}{A}\right)\right\}\right] \tag{4.8.2}$$

$$F(x) = 1-\exp\left\{-\left(\frac{x-B}{A}\right)^k\right\} \tag{4.8.3}$$

Here, the non-exceedance probability F_m is calculated with equation **(4.8.1)**. The values shown in **Table 4.8.1** are adopted for the values of α and β.

Next, from the non-exceedance probability F_m, the standard amount of change is calculated by using equation **(4.8.4)** in the case of the Gumbel distribution, and equation **(4.8.5)** in the case of the Weibull distribution, respectively.

$$y_m = -\ln\left\{-\ln\left(F_m\right)\right\} \tag{4.8.4}$$

$$y_m = \left\{-\ln\left(1-F_m\right)\right\}^{1/k} \tag{4.8.5}$$

If the data completely accord with equation **(4.8.2)** or equation **(4.8.3)**, a linear relationship exists between x and y_m. Therefore, the estimation equation for the return wave height is calculated by assuming a linear relationship for equation **(4.8.6)**, and establishing its coefficient by the least square method.

$$x_m = \hat{A}y_m + \hat{B} \tag{4.8.6}$$

Here, $\hat{A}$, $\hat{B}$ are the estimated value for the coefficients A and B of equation **(4.8.2)** and equation **(4.8.3)**.

② Moreover, in Reference 226), Goda has proposed the following method, which revises the above-mentioned procedure.

(a) Modification of the fitted function (introduction of peak value type II)

The peak value type II function is given by the following equation.

$$F_m = \exp\left[-\left\{1+\left(x-B\right)/kA\right\}^{-k}\right] \tag{4.8.7}$$

Here, the examination has been done in a total of nine ways, one way with the Gumbel distribution in equation **(4.8.2)**, four ways with the Weibull distribution in equation **(4.8.3)** (k = 0.75, 1.0, 1.4 and 2.0), and four ways with the peak value type II distribution in equation **(4.8.7)** (k = 2.5, 3.33, 5.0 and 10.0), as the fitted functions.

In addition, formulation of the following equation is carried out instead of **Table 4.8.1**, for α and β in equation **(4.8.1)**. In other words, it is set as follows.

In the Gumbel distribution:

$$\alpha = 0.44, \quad \beta = 0.12 \tag{4.8.8}$$

In the Weibull distribution:

$$\begin{aligned} \alpha &= 0.20 + 0.27/\sqrt{k} \\ \beta &= 0.20 + 0.23/\sqrt{k} \end{aligned} \tag{4.8.9}$$

In peak value type II:

$$\begin{aligned} \alpha &= 0.44 + 0.52/\sqrt{k} \\ \beta &= 0.12 - 0.11/\sqrt{k} \end{aligned} \tag{4.8.10}$$

(b) Modification of the procedure for selecting the optimal coefficient by introduction of rejection criteria

There are two kinds of criteria for rejecting unsuitable functions, the REC criterion and the DOL criterion. In practical work a procedure is adopted where when a function has been rejected based on either of these criteria, the optimal function is selected according not to the value of the simple correlation, but rather the MIR criterion.

REC criterion, a criterion where the 95% non-exceedance probability of the residual of the correlation coefficient is calculated beforehand for each distribution function, and when the residual of the correlation coefficient when the peak value data are fit to this correlation coefficient exceeds this limit value, that function is rejected as unsuited.

DOL criterion, a criterion wherein the maximum value in the data is made dimensionless with the overall mean value and standard deviation, and if this is below the 5% value or above the 95% value in the distribution function to which this value is fitted, that function is rejected as unsuited.

MIR criterion, a criterion consideration is given to the fact that the mean value of the residual relative to the correlation coefficient of 1 differs depending on the distribution function, and something where the ratio between the residual of the correlation coefficient of the sample and the residual mean value in the applied distribution is judged to be most suitable.

(6) Design tide level and design waves of temporary structures

In the performance verification of temporary structures as well, the design waves are basically set based on the above-described principles. However, since the installation period is limited in the case of temporary structures, it is possible to set the objective return period of the action shorter. If it is a temporary structure whose period is about 2 to3 years, it is common for verification to be carried out for action with a return period of about 10 years.

4.8.2 Setting of Wave Conditions for Verification of Harbor Calmness

The ordinary wave properties that are employed for verification of harbor calmness are generally expressed as a joint frequency distribution table of the wave height and period by wave direction for data broken down by month, by season and annually from the wave data. In a detailed examination of events in which the effects of the period appear strongly, for example, the operating rate it is preferable to arrange an occurrence distribution for the equivalent wave height and wave direction for each period band. Conducting an examination of the wave conditions with observation data serves as the criterion. When the wave observation data are not available, the wave hindcasting results can be utilized. However, in the utilization of the wave hindcasting results, it is preferable to undertake the verification by observation data. It is possible to refer to the manual in Reference 228) as concerns the setting of the ordinary wave conditions for the verification of harbor calmness.

4.8.3 Setting of Wave Conditions for Verification of Durability, Serviceability Limit State, of the Structural Members

(1) The waves for verification of the durability (serviceability limit state) of the structural members are set appropriately as waves acting during the design working lifetime. For the setting of such waves, a joint frequency distribution table of the wave height and period by wave direction is preferably calculated from wave observation data broken down by month, by season and annually. However, since wave observation points where, continuous wave observations have been implemented over a long period of at least three years, and statistical analysis of the occurrence frequency of individual waves has already been conducted, are extremely rare, usually estimation must be carried out based on an occurrence frequency of for significant wave height by class. In the event that the design working lifetime is 50 years, this may be calculated by the following method.

① Wave data
It is possible to employ the NOWPHAS (Nationwide Ocean Wave Information Network for Ports and Harbors) wave observation data, where wave observations are continuously conducted in ports throughout Japan. The occurrence frequency statistics by wave height class of the significant waves every two hours are summarized in the annual Wave Observation Annual Report [229)] or Long-Term Statistical Report [230)] issued by NOWPHAS. Estimation of the circumstances of occurrence of individual waves within an observation time of two hours is carried out based on the significant wave height values provided once in these two hours.

② Estimation of the circumstances of occurrence of individual waves
Since the above-mentioned wave observation materials concern the occurrence frequency of significant waves, the circumstances of the occurrence of significant waves during the observation period is estimated based on the following hypothesis.

(a) The occurrence distribution of individual wave height follows a Rayleigh distribution. Assuming that the significant wave height during two hours is constant, it is possible to assume that the distribution of several individual wave heights occurring during the two hours follows a Rayleigh distribution where the significant wave height is equal.

(b) The number of individual waves during the time differs depending on each observation or period, however since it is extremely difficult to set the number of respective individual waves for each observation for two hours, it can be hypothesized that the value obtained by dividing two hours or 7,200 seconds by the long-term mean period of objective wave observation point is the number of waves during two hours.

③ Frequency distribution of individual waves in the design working lifetime
The number of waves occurrence in the design working lifetime is calculated with the mean period of individual waves during the observation period. The waves for the verification of the serviceability limit state for facilities whose design working lifetime is 50 years can be set as waves for which the number of waves with a wave height that or high or above strike is the order of 10^4 , based on the number of the waves set by the above-mentioned method appear. In the **Design Manual for Pre-stressed Concrete Structure for Ports and Harbors Facilities**, these waves are the waves for the verification of the serviceability limit state, [231)] based on the provisions of the International PC Association, and this is applied here as well.

4.8.4 Conditions of Design Waves in Shallow Waters

(1) Utilization of Numerical Calculation
In cases where design waves are determined in shallow waters, estimation with an appropriate numerical calculation method based on **4.3 Wave Transformations** is generally employed.[232)]

(2) Examination of Stability Against Waves During Construction
It is preferable to use design waves for the facilities in the state of completion, and design waves for verifying the stability of facilities during construction.

(3) Probabilistic of Offshore Waves
During the verification of stability when the facilities in ports are in service and when they are under construction, offshore waves that have an appropriate return period must be employed in accordance with the degree of importance of the facilities. In the case of general facilities of ports, this may be set as probabilistic waves of 50 years in the event that the design working lifetime is 50 years. However, it is necessary to set appropriately the waves during construction i.e. in cases like when the facilities are left as is for a certain period of time, at an uncompleted section by considering the construction period of the facilities and the natural conditions of objective spot, but it is possible to employ something with about probabilistic waves of 10 years, as convenient.

(4) Philosophy about the waves for the verification of stability of facilities
In the performance evaluation of facilities, it is necessary to determine the waves that are acting by considering the following points:

① Random waves are employed.

② Offshore waves are determined by suitable observations, or wave hindcasting.

③ Offshore waves are set as probabilistic waves considering the return period.

④ Wave deformation calculations are implemented considering the topology of the subject location.

⑤ An appropriate numerical calculation model is employed for the calculation of design waves.

(a) Relatively deep marine waters…… Linear calculation model

(b) Shallow waters with complicated topology…… It is desirable to consider the nonlinearity.

(c) Breaking waves and reflected waves occur markedly …. A hydraulic model test is desirable.

⑥ The design tide level is set appropriately. In cases of damage examples in recent years, it has been common for facilities for which high tide level has been used as the design tide level in wave force calculations, however, damages occur in many cases during storm surge.[237] Therefore, as in the case of the performance verification for wave overtopping, it is preferable to consider the simultaneous occurrence with waves and to set this as the tide level that is severest on the facilities, by for example setting this as a tide level where an appropriate storm surge height is added to the high tide level.

⑦ Adequate examination is done concerning stability of facilities during construction as well.

⑧ The return year of design waves during construction is set appropriately.

⑨ The correlation of the waves and flow are considered when the effects of river flow are strong.

4.9 Actions on Floating Body and its Motions

4.9.1 General

(1) The motions of the floating body produced by external forces such as those due to winds, currents and waves, along with the mooring force, should be given due consideration in the performance verification of the floating body.

(2) In general, a floating body refers to a structure that is buoyant in water and its motions within a certain range is permitted during use [239]. When verifying the performance of a floating body, it is necessary to examine both its required functions and its stability. It is necessary to pay attention that the setting of the design conditions on each case is different in general.

(3) Mooring equipment includes a variety of types and is generally composed of a combination of mooring lines, mooring anchors, sinkers, intermediate weights, intermediate buoys, mooring rods, connection joints, and fenders. The mooring equipment has a large influence on the motions of a floating body, and so it is important to verify the stability of the floating body appropriately.

(4) The floating bodies used as port facilities can be divided into floating piers [240], offshore petroleum stockpiling bases [241], floating breakwaters [242], floating bridges [243], [244] and floating disaster-prevention bases.[245], [246] Moreover, researches for development of very large floating structures [247], [248], [249], [250] are being carried out.

(5) Floating bodies can also be classified by the type of mooring methods. As described below, mooring methods include catenary mooring (slack mooring), taut mooring, and dolphin mooring.

① Catenary mooring (**Fig. 4.9.1(a)**)
This is the most common mooring method. With this method, the chains or whatever used in the mooring are given sufficient lengths to make them slack. This means that the force restraining the motions of the floating body is small, but the mooring system fulfills the function of keeping the floating body in more-or- less the same position. There are various types of catenary mooring, depending on factors like the material of the mooring lines, the number of mooring lines, and the presence or absence of intermediate buoys and sinkers.

② Dolphin mooring (**Fig. 4.9.1(b)**)
With this method, mooring is maintained using either a pile-type dolphin or a gravity-type dolphin. In general, this method is suitable for restraining the motions of a floating body in the horizontal direction, but a large mooring force acts on the dolphin. This method has been used for mooring floating units of offshore petroleum stockpiling bases.

③ Taut mooring (**Fig. 4.9.1(c)**)
This is a mooring method that reduces the motions of the floating body greatly; a tension leg platform (TLP) is an example. With this method, the mooring lines are given a large initial tension so that they do not become slack even when the floating body moves. The advantages of this mooring method are that the floating body does not move much, and only a small area is needed for installing the mooring lines. However, it is necessary to take note of the fact that because a large tensile force is generated in the mooring lines, the design of the lines becomes the critical factor on the safety of the floating body.

④ Mooring method using a universal joint (**Fig. 4.9.1(d)**)
The mooring system shown in the figure is an example of a mooring method that can be used to moor a large offshore floating body. Examples of mooring systems that use a universal joint on the sea bottom include a SALM (Single Anchor Leg Mooring) type single point mooring buoy [251] and a MAFCO (MAritime Facility of Cylindrical cOnstruction) tower [252].

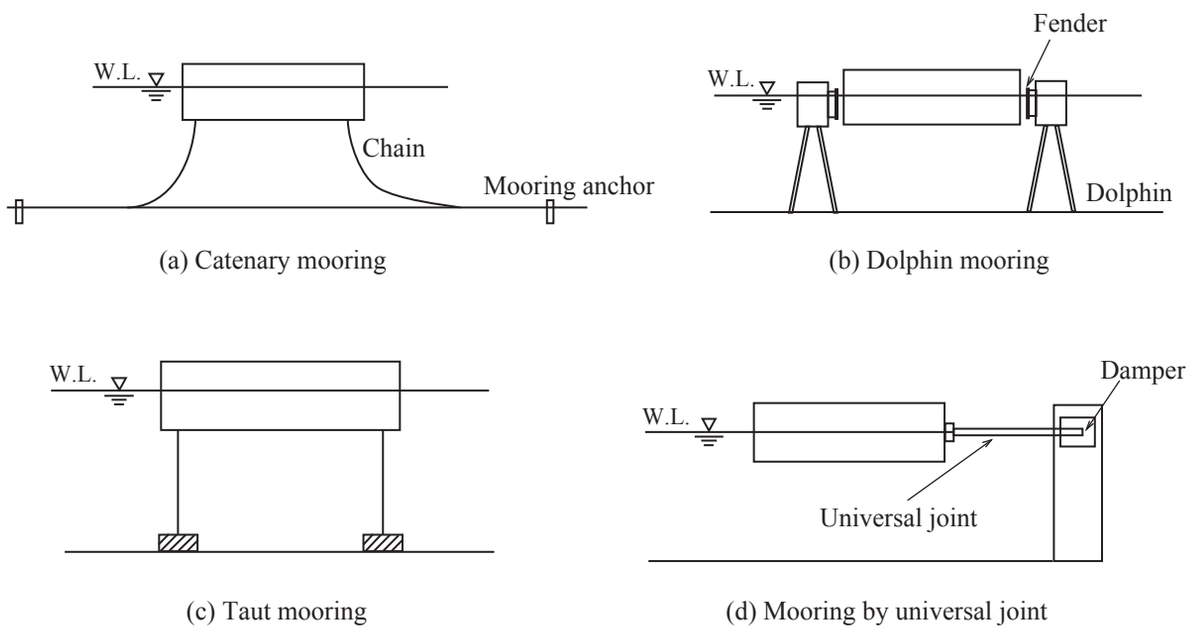


Fig. 4.9.1 Examples of Mooring Methods for Floating Body

4.9.2 Actions on Floating Body

(1) Types of Actions and Calculation Methods
When a port facility is made of floating structures, it shall be standard to consider the following forces : wind drag force, drag force by currents, wave-exciting force, wave-drift force, wave-making resistance, restoring force, and mooring force. These actions shall be calculated by an appropriate analytical method or hydraulic model tests, in accordance with the mooring method for the floating body and the size of facility.

(2) Wind Drag Force
With a structure for which a part of the floating body is above the sea surface, winds exert an action on the structure. This action is called the wind drag force or wind pressure, and is composed of a pressure drag and a friction drag. If the floating body is relatively small in size, the pressure drag is dominant. The pressure drag is proportional to the square of the wind velocity and is expressed as in the following equation. The subscript k in the equation refers to the characteristics value:

$$F_{Wk} = \frac{1}{2}\rho_a C_{DW} A_W U_{Wk}{}^2 \tag{4.9.1}$$

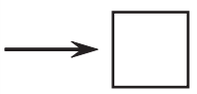
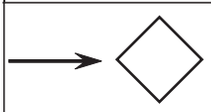
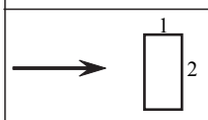
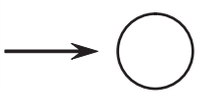
where

F_W : wind drag force (N)
ρ_a : density of air (kg/m^3)
A_W : projected area of the part of the floating body above the sea surface as viewed from the direction in which the wind is blowing (m^2)
U_W : wind velocity (m/s)
C_{DW} : wind drag coefficient

The wind drag coefficient is a proportionality constant and is also known as the wind pressure coefficient. It may be determined by wind tunnel tests. However, it is also acceptable to use a value that has been obtained in the past experiments for a structure with a shape similar to the structure under current study.

Values such as those listed in **Table 4.9.1** have been proposed as the wind drag coefficients of objects in the uniform flow. As can be seen from this table, the wind drag coefficient varies with the shape of the floating body, but it is also affected by the wind direction and the Reynolds number. Note that it is considered that the wind pressure acts in the direction of the wind flow, with the point of application being the centroid of the projection of the part of the floating body that is above the water surface. However, it is necessary to pay attention to the fact that this may not necessarily be the case if the floating body is large. Moreover, the velocity of the actual wind is not uniform in the vertical direction, and so the value of the wind velocity U_W used in the wind pressure calculation is set as that at the elevation of 10 m above the sea surface.

Table 4.9.1 Wind Pressure Coefficient

	Square cross-section	2.0 [1.2] (0.6)
	Square cross-section	1.6 [1.4] (0.7)
	Rectangular cross-section (ratio of side lengths = 1:2)	2.3 [1.6] (0.6)
	Rectangular cross-section (ratio of side lengths = 1:2)	1.5 (0.6)
	Rectangular cross-section (when one face is in contact with the ground)	1.2
	Circular cross-section (smooth surface)	1.2 (0.7)

(3) Drag Force by Currents

When there is a current such as tidal currents, these currents will exert a force on the submerged part of the floating body. This force is referred to as the flow pressure or the drag force by currents. Like the wind drag force, it is proportional to the square of the flow velocity. Note however that since the velocity of the current is generally small, the current drag force is actually expressed as being proportional to the square of the velocity of the current relative to the velocity of motion of the floating body as in the following equation. The subscript k in the equation refers to the characteristics value:

$$F_{Ck} = \frac{1}{2}\rho_0 C_{DC} A_C \left|U_{Ck} - U_k\right| \left(U_{Ck} - U_k\right) \tag{4.9.2}$$

where

F_C : drag force by currents (N)
ρ_0 : density of fluid (kg/m^3)
A_C : projected area of the submerged part of the floating body as viewed from the direction of the currents (m^2)
U_C : velocity of the currents (m/s)
U : velocity of motion of the floating body (m/s)
C_{DC} : drag coefficient with respect to the currents

The drag coefficient C_{DC} is a function of the Reynolds number. When the Reynolds number is large, however, the values for steady flow in **Table 6.5.1** in **6.5 Fluid Force due to Current** may be used.

The drag coefficient for the currents varies with the shape of the floating body, the direction of the currents and the Reynolds number. As with the wind pressure, the direction of the force exerted by the currents and the

direction of the currents itself are not necessarily the same.[253] In general, the deeper the draft of the floating body relative to the water depth, the larger the drag coefficient for the currents becomes. This is referred to as the shoaling effect, and the drag coefficient increases because the smaller the gap between the sea bottom and the base of the floating body, the harder it is for water to flow through this gap.

(4) Wave-exciting Force

The wave-exciting force is the force exerted by incident waves on the floating body when the floating body is considered to be fixed in the water. It is composed of a linear force that is proportional to the amplitude of the incident waves and a nonlinear force that is proportional to the square of the amplitude of the incident waves. The linear force is the force that the floating body receives from the incident waves as reaction when the floating body deforms the incident waves. The velocity potential for the deformed wave motion is obtained using wave diffraction theory. The nonlinear force, on the other hand, is composed of a force that accompanies the finite amplitude nature of waves and a force that is proportional to the square of the flow velocity. The former force due to finite amplitude effect can be analyzed theoretically, but in practice it is often ignored. The latter force that is proportional to the square of the flow velocity becomes large, in particular when the diameter of the floating body is small relative to the wavelength; it is necessary to determine this force experimentally.

(5) Wave Drift Force

When waves act on a floating body, the center of the floating body's motion gradually shifts in the direction of wave propagation. The force that causes this shift is called the wave drift force. If it is assumed that the floating body is two-dimensional and the wave energy is not dissipated, then the wave drift force is given by the following equations.[254] The subscript k in the equation refers to the characteristics value:

$$F_{d_k} = \frac{1}{8}\rho_0 g H_{i_k}^2 R \tag{4.9.3}$$

$$R = K_R{}^2\left\{1 + \frac{4\pi h / L}{\sinh(4\pi h / L)}\right\} \tag{4.9.4}$$

where

$\rho_0 g$: unit weight of seawater (kN/m^3)
h : water depth (m)
L : wavelength (m)
F_d : wave-drift force per unit width (N)
H_i : incident wave height (m)
K_R : reflection coefficient
R : wave drift force coefficient

If the dimensions of the floating body are extremely small relative to the wavelength, the wave drift force may be ignored as being much smaller than the wave-exciting force. However, as the floating body becomes larger, the wave drift force becomes dominant.

When random waves act on a floating body moored at a system having only a small restraining force, such as a single point mooring buoy designed for use of supertankers, the wave drift force becomes a dominant factor as it may give rise to slow drift oscillations. In this case, the long-period fluctuating drift force in the form of the wave drift force has a large effect to the slow drift oscillations of the floating body. For example, if a random wave is comprised of waves with two different frequencies, the fluctuating wave drift force is given by the following equation.[255] The subscript k in the equation refers to the characteristics value:

$$F_{d_k} = \frac{1}{4}\rho_0 g H_{i_k}^2 R\left(\frac{\omega_1 + \omega_2}{2}\right)\left(1 + \cos\left(\frac{\omega_1 - \omega_2}{2}\right)t\right) \tag{4.9.5}$$

where

$\rho_0 g$: unit weight of seawater (kN/m^3)
F_d : wave (fluctuating) drift force per unit width (N)
H_i : incident wave height (m)
$R((\omega_1+\omega_2)/2)$: wave drift force coefficient by regular waves of $\cos((\omega_1+\omega_2)/2)$
ω_1 and ω_2 : wave frequency (rad/s)
t : time (s)

(6) Wave-making Resistance Force

When a floating body moves in still water, the floating body exerts a force on the surrounding water, and the floating body receives a corresponding reaction force from the water; this reaction force is called the wave-making

resistance force. This force may be determined by forcing the floating body to move through the still water and measuring the force acting on the floating body. In general, however, an analytical method is used whereby each mode of the floating body motions is assumed to be realized separately, and the velocity potential, which represents the motion of the fluid around the floating body, is obtained. Only the forces that are proportional to the motion of the floating body may be determined analytically; the nonlinear forces that are proportional to the square of the motion cannot be determined analytically. Out of the linear forces i.e., that are proportional to the motion of the floating body, the term that is proportional to the acceleration of the floating body is called the added mass term, while the term that is proportional to the velocity is called the wave damping term.

(7) Restoring Force
The static restoring force is the force that makes a floating body return to its original position when the floating body moves in still water. It is generated by buoyancy and gravity when the floating body heaves, rolls or pitches. This force is generally treated as being proportional to the amplitude of the motion of the floating body, although this proportionality is lost if the amplitude becomes too large.

(8) Mooring Force
The mooring force is the force that is generated in order to restrain the motion of the floating body. The magnitude of this force depends greatly on the displacement-restoration characteristics of the mooring system.

(9) Solution Method for Wave-exciting Force and Wave-making Resistance Force Using Velocity Potential
The method adopted for calculating the wave-exciting force and the wave-making resistance force involves deriviation of the velocity potential, which represents the motion of the fluid, and then calculating the wave exciting force and the wave-making resistance force from the potential. The analytical method with the velocity potential is the same for both the wave-exciting force and the wave-making resistance force, the only difference being the boundary conditions. The velocity potential may be obtained using any of a number of methods, such as a region segmentation method, an integral equation method, a strip method, or a finite element method. Outline of the above mentioned numerical calculation methods are introduced in Reference 256) and 257).

(10) Wave Force Acting on Fixed Floating Body with Rectangular Cross Section
When a floating body is fixed in position, the velocity potential that satisfies the boundary conditions at the sea bottom and around the floating body can yield the wave force. The wave force acting on a floating body with a long rectangular cross section such as a floating breakwater can be determined using the approximation theory of Ito et al.[258].

(11) Materials for Mooring and Mooring Force
Reference 259) may be referred for the materials used in mooring and their characteristic features.

(12) Forces Acting on a Very Large Floating Structure
For a very large floating structure, the external forces described in (2) through (11) above are different from those for a smaller floating body because of its large size and elastic response characteristics of the floating body structure. It is thus necessary to carry out sufficient examinations on their characteristics.[260]

4.9.3 Motions of Floating Body and Mooring Force

(1) Calculation Methods of Motions of Floating Body and Mooring Force
The motions of a floating body and the mooring force need to be calculated by an appropriate analytical method or hydraulic model tests, in accordance with the shape of the floating body and the characteristics of the external forces and the mooring system.

(2) Motions of Floating Body
The motions of a floating body can be determined by solving the dynamic equilibrium equation, with the external forces taken to be the forces due to winds and waves, the restoring force of the floating body itself, and the reaction forces of the mooring lines and fenders. If the floating body is assumed to be a rigid body, then its motions are comprised of the six components shown in **Fig. 4.9.2**, namely surging, swaying, heaving, pitching, rolling and yawing. Out of these, the modes that represent motions within the horizontal plane, namely surging, swaying and yawing, may show long-period motions with the period of a few minutes or more. Such long-period motions have a large influence on the verification of the anchoring area of ships and the mooring equipment. One may thus give separate consideration to the long-period motions, taking only the wave-drift force and the long-period fluctuating components of the winds and waves as the external forces when doing analysis.

If the floating body is very long, elastic deformation may accompany the motions of the floating body and this should be examined, as necessary.

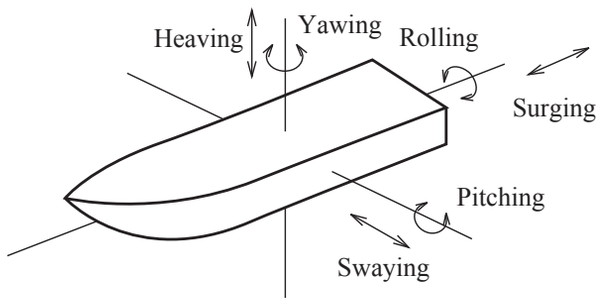


Fig. 4.9.2 Components of Motions of Floating Body

(3) Methods of Solving the Equations of Motion

① Steady state solution method for nonlinear equations of motion
The equations of motion for a floating body are nonlinear, meaning that it is not easy to obtain solutions. Nevertheless, if it is assumed that the motion amplitudes are small and the equations of motion are linearized by using linear approximations for the nonlinear terms, the solutions can be obtained relatively easily. For example, for a three-dimensional floating body, one ends up with a system of six simultaneous linear equations involving the amplitudes and phases of the six modes of motions. Note that if the floating body is assumed to be a rigid body and its motions are linear, then the motions are proportional to the external forces. In particular, if there are no currents or wind, then the motions are proportional to the wave height.

② Numerical simulation of nonlinear motions
The wind drag force and the drag force by currents are in general nonlinear, and moreover the restraining forces of mooring equipment are also often nonlinear. In this case, an effective solution method is to use a numerical simulation where the equations of motion are progressively solved for a series of time steps. The equations of motion for the floating body can be divided into the constant coefficient method [261] in which the coefficient values of the added mass term and wave damping term within the equations of motion is fixed at a specific frequency, and the phase lag function method [262), 263] in which these terms are changed over time in a simulation using a phase lag function. The phase lag function method is also called the memory effect function method. In the numerical simulation, first, the time series data are obtained for the wave-exciting force and the flow velocity due to the waves from the input of incident wave spectrum as well as the fluctuating wind speed from the wind spectrum. The external forces obtained from these time series data are then put into the equations of motions for the floating body, and the time series data for the motion of the floating body and the mooring force are calculated.

Numerical simulations are used for analyzing the motions of all kinds of floating bodies. For example, Ueda and Shiraishi [261] have carried out numerical simulations on the motions of a moored ship, and Suzuki and Moroishi [264] have analyzed the swinging motion of a ship moored at a buoy.

As preconditions to the numerical simulation, it is usually assumed that the fluid is an ideal fluid, that the amplitudes of motions of the floating body are small and that the incident waves are linear and their superposition is allowed. If these assumptions cannot be held, it is necessary to carry out hydraulic model tests.

(4) Hydraulic Model Tests
Hydraulic model tests provide a powerful technique for determining the motions of a floating body and the mooring force. Up to the present time, hydraulic model tests have been carried out for all kinds of floating bodies. For examples, see references 265) and 266).

When conducting hydraulic model tests of a floating body, attention should be paid to any similarities in the inertia moments of the floating body and the characteristics of mooring equipment.

(5) Statistical Treatment of Motions of Floating Body and Mooring Force
The motions and mooring forces for a floating body obtained by numerical simulation due to random waves and hydraulic model tests varies irregularly with time. Therefore, the peak values of the motion amplitudes and mooring forces for the floating body also vary. Even if the wave spectrum is identical, the maximum values for these vary when the duration time or the series of the waves are different. In other words, since the motion amplitudes and mooring forces for the floating body are probability variables, statistical treatment should be performed to estimate the expected values. In the usual statistical treatment, a normal distribution or a Rayleigh distribution are applied to the probability density distributions of the peak values, and the expected values are estimated.

(6) Motions and Mooring Force for Rectangular Section Floating Body
Ito's approximation theory, [258] which is relatively easy to handle, can be applied for calculating the motions and

mooring forces for a rectangular section floating body.

(7) Procedure for Estimating Expected Values of Motions

The expected values for the motions of a floating body, can be estimated by taking into consideration its motion characteristics and assuming either a normal distribution or a Rayleigh distribution.[266] The procedure is described below.

① Motion simulation

A motion simulation for adequate calculation time is performed, and the values of double amplitudes of the motions for each wave are calculated. The number of waves needed to estimate accurately the expected value of the maximum values is approximately 100 or more.

② Assumption of the distribution shape of the motion amplitude

A suitable distribution shape for the double amplitudes of the motions obtained by motion simulation is assumed. A Rayleigh distribution or a normal distribution shown below may be adopted as the distribution shape.

Rayleigh distribution:

$$P(x)dx = 2a^2 \exp\left(-a^2x^2\right)dx \tag{4.9.6}$$

Normal distribution:

$$P(A) = \frac{1}{\sqrt{2\pi\sigma}} \exp\left\{-\frac{1}{2}\left(\frac{A-\overline{A}}{\sigma}\right)^2\right\} \tag{4.9.7}$$

where,

x : A/A^*
A : double amplitude
A^* : arbitrary base double amplitude
a : $A^*/(8m_0)^{1/2}$
$8m_0$: A_{rms} (square root of the square mean of the double amplitudes)
σ : standard deviation

However, the value of a is 1.416 when the arbitrary base double amplitude A^* is the significant value $A_{1/3}$, and $\sqrt{\pi}/2$ when it is the mean value .

③ Estimation of the expected maximum value

Assuming that the number of waves is N, and that the value at which the exceedance probability becomes $1/N$ is the expected value of the maximum values at that number of waves, the expected value of the ratios of double amplitudes of motions in a Rayleigh distribution is approximated by the following equation when N is sufficiently large.

$$x_N = \frac{1}{a}(\ln N)^{1/2} \tag{4.9.8}$$

On the other hand, in a normal distribution, the expected value of the double amplitudes is expressed by the following equation.

$$A_N = \overline{A} + \mu_N \sigma \tag{4.9.9}$$

The expected maximum value varies depending on the number of waves N. **Table 4.9.2** shows the values of x_N relative to representative values of N and the values of μ_N, which is the parameter of the deviation of a standard normal distribution.

④ Calculation of the expected value of the maximum values

For example, assuming a Rayleigh distribution as the distribution shape, consider a case where the expected value of the maximum values for the number of waves of 1,000 is calculated. First of all, the significant value A^* of the double amplitudes of motions is calculated from the simulation results. Next, $a = 1.416$ and $N = 1{,}000$ are substituted into equation **(4.9.8)**, and the value of x_N is calculated. Finally, the expected value, A is calculated from $x_N = A/A^*$.

(8) Similarity Laws for Mooring Systems

The characteristics of the motions of a floating body vary greatly with the mooring method. When carrying out hydraulic model tests on a floating body, it is thus particularly important to give appropriate consideration to the

similarity laws for the displacement and reaction force characteristics of the mooring equipment. For example, with a mooring rope, if the material used in the hydraulic model tests is kept the same as that used in the field and the size is simply scaled down while maintaining the same shape, then the similarity laws will not hold; rather it is necessary to scale down the elastic modulus of the material used in the models relative to that used in the prototype. In practice, however, it will probably be unable to find such a material, in which case various other measurements must be used.

Table 4.9.2 Values for Estimation of Expected Values

Number of samples N relative to the expected maximum value	100	200	500	1000	10000
Rayleigh distribution x_N	1.52	1.63	1.76	1.86	2.14
Normal distribution μ_N	2.33	2.58	2.88	3.09	3.96

References

1) JSCE: Design Handbook for Shore Protection Facilities. JSCE, pp 9-25, 2000
2) Goda, Y: Random Seas and Design of Maritime Structures: (Enlarged Edition), Kajima Publishing, pp. 23, 1990
3) Goda, Y: Random Seas and Design of Maritime Structures: (Enlarged Edition), Kajima Publishing, pp. 24, 1990
4) Goda, Y. and Y. Suzuki: Computation of refraction and diffraction of sea waves with Mitsuyasu's directional spectrum, Technical Note of PHRI, No.230, pp.45, 1975
5) Goda, Y: Random Seas and Design of Maritime Structures: (Enlarged Edition), Kajima Publishing, pp. 11-22, 1990
6) Takayama, T. and T. Hiraishi: Amplification Mechanism of Harbor Oscillation Derived From Field Observation And Numerical Simulation, Technical Note of PHRI, No. 636, pp.70, 1988
7) Hashimoto, N.: Study on Estimation Methods of Directional Ocean Wave Spectrum, Technical Note of PHRI, No. 722, pp.118, 1988
8) GODA, Y.: Numerical Examination of the Measuring Technique of Wave Direction with the 'Covariance Method', Rept. Of PHRI Vol.20 No.3 1981.9
9) Sverdrup, H.U. and Munk, W.H.: Wind, sea, and swell: theory of relations for forecasting, U.S. Navy Hydrographic Office, H.O. Pub. 601, pp.1-44, 1947
10) Bretschneider, C.L.: Revised wave forecasting relationships, Proc. 2nd Conf. Coastal Engg., ASCE, 1952
11) Bretschneider, C.L.: Revisions in wave forecasting, Deep and shallow water, Proc. 6th Conf. Coastal Engg., ASCE, pp.30-67, 1958
12) Wilson, B.W.: Numerical prediction of ocean waves in the North Atlantic for December, 1959, Deut, Hydrogr. Zeit, Jahrgang 18, Heft 3, pp.114-130, 1965
13) Goda, Y.: Revisiting Wilson's Formulas for Simplified Wind-Wave Prediction, Journal of Port, Coast, and Ocean Engineering, ASCE, Vol.129, Issue 2, pp.93-95, 2003
14) JSCE: The Collected Formula of Hydraulics, JSCE, 1971, p.508
15) Wilson, B.W.: Graphical approach to the forecasting of waves in moving fetches, Tech. Memo. No. 73, Beach Erosion Board, U.S. Army Corps of Engrs., 1955
16) Ijima, T.: Numerical forecast of waves, JSCE Proceedings of Summer Training on Hydraulic Engineering A, pp.2-1-2-30, 1968
17) Horikawa, K, H. Nishimura, Y. Ozawa and Y. Miyamoto: Wave hindcasting under typhoon in Beppu Bay, Proceedings of the 18th Conference on Coastal Eng. JSCE ,pp.7-12,1971
18) Bretschneider, C.L.: Decay of ocean waves, Fundamentals of ocean engineering-Part 8b, Ocean Industry, pp.45-50, 1968
19) Horikawa, K.: (New Edition)Coastal Engineering, Tokyo University Press, pp, 84-88, 1991
20) lsozaki, I. and Uji, T.: Numerical prediction of ocean wind waves, Report of \Japan Meteorological Institute Vol. 24, No. 2, pp. 207-231, 1973
21) Uji, T. : A coupled discrete wave model MRI-II, J. Oceanogr. Soc. Japan, 40, pp.303-313, 1984
22) Uji, T and Ueno K.: Wave Model MRI-II New, Japan Meteorological Institute Oceanography Division,, p.118, 1987
23) Ueno, K. and H. Takano: The third generation eave hindcasting model MRI-3 for on-site operation. Proceeding of Fall Term conference of Oceanographic Society of Japan, P. 153, 1998
24) lnoue, T.: On the growth of the spectrum of a wind generated sea according to a modified Miles-Philips mechanism and its application to wave forecasting, Dept. Meteorol. and Oceanogr., New York Univ., TR-67-5, p.74, 1967
25) Yamaguchi, M. and Y. Tsuchiya: Numerical forecast method for waves in finite fetch, Proceedings of the 26th Coastal Engineering, JSCE, pp.96-100, 1979
26) Joseph, P. S. Kawai, S. and Toba, Y.: Ocean wave prediction by a hybrid model-combination of single-parameterized wind waves with spectrally treated swells, Sci. Rept. Tohoku Univ., Ser. 5, Geophs., Vol. 28, No.1, pp.27-45, 1981

27) Yamaguchi, M, Y. Hatada and Y. Utsunomiya: A shallow water prediction model at a single location and its applicability, Journal of the ASCE Vol.. 381/H7, pp. 151-160, 1987
28) The WAMDI Group: The WAM model- A third generation ocean wave prediction model, J. Phys. Oceanogr., 18, pp.1775-1810, 1988
29) Hasselmann, S and K. Hasselamann: Computations and parameterizations of the nonlinear energy transfer in a gravity wave spectrum, Part 1, A new method for efficient computations of the exact nonlinear transfer integral, J. Phys. Oceanogr., 15, pp.1369- 1377, 1985
30) Janssen, P.A.E.M.: Wave-induced stress and the drag of air flow over sea wave, Journal of Physical Oceanography, vol.19, pp.745-754, 1989
31) Janssen, P.A.E.M.: Quasi-linear theory of wind wave generation applied to wave forecasting, Journal of Physical Oceanography, vol.21, pp.1631 1642, 1991
32) Hasselmann, K.: On the non-linear energy transfer in a gravity-wave spectrum, Part 1, General theory, J. Fluid Mech., 12, pp.481- 500, 1962
33) Booij, N., Holthuijsen, L. H. and R. C. Ris: The SWAN wave model for shallow water, Proceeding of 25th International Conference on Coastal Engineering, Vol.1, pp. 668-676, 1996.
34) Goda, Y. and Y. Suzuki: Computation of refraction and diffraction of sea waves with Mitsuyasu's directional spectrum, Technical Note of PHRI, No.230, pp.45,1975
35) Goda, Y: Standard wave spectrum and stochastic characteristics based on numerical simulation, Proceedings of Coastal Eng. JSCE, Vol.34, pp.131-135, 1987
36) JSCE: Design Handbook for Shore Protection Facilities. JSCE, pp 33-36, 2000
37) Takayama, T., N. Ikeda and T. Hiraishi: Practical Computation Method of Directional Random Wave Transformation, Report of PHRI Vol.30 No.1, pp.21-67, 1991
38) Coastal Engineering Committee, JSCE: Coastal waves, JSCE, pp.15-118, 1994
39) Goda, Y: Random Seas and Design of Maritime Structures: (Enlarged Edition), Kajima Publishing, pp. 41-66, 1990
40) The WAMDI Group : The WAM model- A third generation ocean wave prediction model, J. Phys. Oceanogr., 18, pp.1775-1810, 1988
41) Sato, S. and Y. Goda: Coast and Harbour, Easy-to-understand Civil Engineering Lecture, Shokokusha Publishing, pp.72-75,1971
42) Takayama, T.: Wave Diffraction and Wave Height Distribution inside a Harbor, Technical Note of PARI No367, p.140, 1981
43) Kubo, Y., Y. Kotake, M. Isobe and A. Watanabe: Time-dependent mild slope equation for random waves, Proceedings of Coastal Engineering JSCE No. 38, pp.46-50, 1991
44) Hiraishi, T. I. Uehara and Y. Suzuki: Applicability of Wave Transformation Model in Boussinesq Equation, Technical Note of PHRI No.814, p.22, 1995
45) Nadaoka, K and Y Nakagawa: Derivation of fully-dispersive wave equations for irregular wave simulation and their fundamental characteristics, Journal of the JSCE, No,467/H-23,pp.83-92,1993
46) Hirayama, K, et. al. : Utilization of Numerical Simulation on Nonlinear Irregular Wave for Port and Harbor Design, Technical Note of Port and Airport Research Institute, No.1036, 162p, 2002
47) Hirayama, K., Y. Minami, M. Okuno, K. Minemura, H. Kawai and T. Hiraishi: Case Study on Wave Disaster due to Typhoons in 2004, Technical Note of PHRI, No.1101, p.42, 2005
48) Tanimoto, K., K. Kimura, A. P. dos Santos Pinto: Random Wave Forces and Design Wave Periods of Composite Breakwaters under the Action of Double Peaked Spectral Waves, Rept. Of PHRI, Vol.25 No.2, pp.3-25, 1986
49) Nishimura, K., H. Kato, J. Inoue, I. Nakamura and S. Muratra: Numerical simulation and hydraulic model test on wave height distribution about an island, Proceedings of 27th Coastal Eng. JSCE, pp.65-69, 1980
50) Hirayama, K: Numerical Simulation of Nonlinear Partial Standing Waves using the Boussinesq Model with New Reflection Boundary, Rept. of PHRI Vol. 40, No. 4, pp.3-48, 2001
51) Sawaragi, T Edition: New Coastal Engineering for environmental sphere, Fuji Techno System, pp.251-252,1999
52) Tanaka, N. et al.: New dictionary of applied coastal engineering- waves around structures, Fuji Techno System, 1999
53) Goda, Y., Y. Suzuki, Y. Kishira and O. Kikuchi, Estimation of incident and Reflected Waves in Random Wave Experiments, Technical Note of PHRI No.248, p.24, 1976
54) Kobune, K and M. Osato: A Study of Wave Height Distribution along a Breakwater with a Corner, Report of PHRI, Vol. 15 No.2, 1976
55) Ito, Y. and K. Tanimoto: Meandering Damages of Composite Type Breakwaters, Technical Note of PHRI, No.112, 1971
56) Goda, Y. and T. Yoshimura: Wave Force Computation for Structures of Large Diameter, Isolated in the Offshore, Report of PHRI
57) Shuto, N.: Transformation of long wave- for the case of changing channel width and depth-, Proceedings of 21st Conference on Coastal Engineering JSCE, pp.57-64, 1974
58) Goda, Y.: Deformation of Irregular Waves due to Depth-Controlled Wave Breaking, Report of PHRI, Vol. 14 No.3, 1975
59) JSCE: Design Handbook for Shore Protection Facilities. JSCE, pp 90, 2000
60) Goda, Y: Rearrangement of breaker indices, Journal of Civil Engineering No. 180, JSCE, ,pp.39-49, 1970
61) Takayama, T., Y. Kamiyama and O. Kikuchi: Wave Transformation On a Reef, Technical Note of PHRI No.278, pp.32, 1977
62) Egashira, K, I. Fukuda, Y. Kishira and T. Nishimura: Field observation of transformation of waves on reefs., Proceedings of

32nd conference on Coastal Engineering, JSCE, pp.90-94, 1985
63) Hiraishi, T.: Generation and Application of Directional Random Waves in a Laboratory, Technical Note PHRI No, 723, p. 176, 1992
64) Mase H.: Random wave runup height on gentle slope, J. Waterway, Port, Coastal and Ocean engineering, ASCE, Vol.118, No.5, pp.649-664, 1989
65) Van der Meer, J.W. and C.M.Stam: wave runup on smooth and rock slopes of coastal structures, J. Waterway, Port, Coastal and Ocean Engineering, ASCE, Vol.118, No.5, pp.534-550, 1992
66) Mase, H., T.S, Hedges M. Share and S. Nagase: Study on wave overtopping rate of sloping coastal dikes considering wave up rush,
67) Saville, T. Jr.: Waverun-up on composite slopes, Proc. 6th Conf. on Coastal Eng., pp.691-699, 1958
68) Nakamura, A, Y. Sasaski and J. Yamada: Study on wave run-up on the composite slopes, Proceedings of 13th Conference on Coastal Eng., JSCE, PP.309-312, 1972
69) Hosoi, M. and H. Mitsui: Wave up rush on the sea walls placed inner shore inside breaker line, Proceedings of 9th Coastal Eng., JSCE, 1962
70) Hosoi, M. and N. Shuto: Wave up rush for the case of diagonal incident wave, Proceedings of 9th Coastal Eng,. JSCE, pp.149-152, 1962
71) Study Group on application of numerical wave tank to structural design against waves: Study and development of numerical wave channel, Coastal Development Technology Library, CDIT, pp,156-163, 2001.
72) Sakuraba, M,, S. Hirosaki and K. Kashiyama: Numerical calculation of wave overtopping and wave run-up by CIVA/Level set method, Proceedings of Coastal Eng. No. 51, JSCE, pp,36-40,2004,
73) Hiraishi, T. K. Hirayama and H. Kawai: A Study on Wave-0vertopping by Typhoon No. 9918, Technical Note of PHRI No.972, p.19, 2000
74) Goda, Y, Y. Kishira and Y. Kamiyama: Laboratory investigation on the overtopping rate of seawalls by irregular waves, Rept. Of PHRI Vol. 14, No. 4, pp.3-44,1975
75) GODA Y. and Y. KISHIRA: Experiments on irregular Wave Overtopping Characteristics of Seawalls of Low Crest Types, Technical Note of PHRI No. 242, p.28, 1976
76) TAKAYAMA, T. T. NAGAI and K. NISHIDA: Decrease of Wave Overtopping Amount due to Seawalls of Low Crest Types, Rept. Of PHRI Vol.21 No.2,pp.151-205,1982
77) GODA, Y: Estimation of the Rate of Irregular Wave Overtopping of Seawalls, Rept. Of PHRI Vol.9No.4,pp.3-41,1970
78) Fukuda, N. T. Uno and I. Irie: Field observation of wave overtopping at sea walls (Second Report)Proceeding s of 20th Coastal Eng. JSCE, PP.113-118, 1973
79) Nagai, S. and A. Takada: Effect of wave absorbing blocks on wave overtopping at coastal dikes, Proceeding of 11th Coastal Eng. JSCE, pp.279-286,1964
80) Coastal Development Institute of Technology (CDIT): Study and Development of Numerical wave channel (CADMAS-SURF), Coastal Development Technology Library No. 12, 78p, 2001
81) YASUDA, T and T. HIRAISHI: Highly Accurate Inundation Simulation by MARS Method on Compound Coastal Urban Area: Rept of PARI Vol.43, No.2, pp.3-34,2004
82) Takayama, T. T. Nagai, K. Nishida and T. Sekiguchi: Hydraulic Model Test for wave overtopping characteristics of sea walls against diagonal random incident waves, Proceeding of Coastal Eng. JSCE, Vol. 31, pp.542-546, 1984
83) HIRAISHI, T. N. MOCHIZUKI, K. SATO, H. MARUYAMA, T. Kanazawa: Effect of Wave Directionality on Overtopping at Seawall, Rept of PHRI Vol. 35 No. 1, pp. 39-64, 1996
84) Miyajima, S. S. Ogura, Y. Ogusu, T. Morikawa and S. Okuda: Experimental Study on wave overtopping rate of sloping sea wall having parapet, Proceeding of Coastal Eng. JSCE, Vol. 51, pp. 636-640, 2004
85) Goda, Y. Y. Suzuki and Y. Kishira: Random wave model test and the characteristics, Proceedings of 21st Coastal Eng. JSCE, pp.237-242, 1974
86) Goda, Y: Random Seas and Design of Maritime Structures, Kajima Publishing, pp. 100-102, 1990
87) Kondo, T. and I. Sato: A study on the crest height of breakwaters, Journal of Public Works Research Institute (PWRI), Hokkaido Regional Development Bureau (HRDB)No. 117, pp.1-5, 1964
88) Sakamoto, Y. Y. Miyaji T. Uenishi and H., Takeda: Experimental study on hydraulic characteristics of sloping sea walls, Journal of Public Works Research Institute, Hokkaido Regional Development Bureau Vol. 82, 1984
89) Kondo, T. and H. Takeda: wave absorbing facilities, Motikita Publishing, pp. 70-129, 1983
90) MORIHIRA, M., S. KAKIZAKI and Y. GODA: EXPERIMENTAL INVESTIGATION OF A CURTAIN-WALL BREAKWATER, Rept. Of PHRI Vol. 3 No.1, pp. 1-27, 1964
91) Longuet-Higgins, M.S. and R.W. Stewart: Radiation stress and mass transport in gravity waves with application to surf beats, Jour. Fluid Mechanics Vo1.13, pp.481-504, 1962
92) Goda, Y. Design diagram of wave set-up and longshore currents under directional random waves, Proceedings of Ocean Development Vol. 21, pp.301-306, 2005
93) Coastal Development Institute of Technology (CDIT): Impact Evaluation Manual for long-period waves in ports, Coastal Technology Library No. 21, CDIT, 2004, 86p.
94) Kondo, T. and H. Takeda: Wave Absorbing Structures, Morikita Publishing Co., 1983, ,pp. 70-129,
95) KATO, K., S. NAKAMURA and N. IKEDA: Estimation of Infragravity Waves in Consideration of Wave Groups (An

Examination on Basis of Field Observation at HORF), Rept of PHRI Vol. 30 No.1, pp.137-163, 1991
96) HIRAISHI, T, A. TADOKORO and H. FUJISAKU: Characteristics of Long Period Wave Observed in Port, Rept. Of PHRI Vol.35 No.3, pp.3-36,1996
97) Coastal Engineering Committee, JSCE: Coastal waves, JSCE, ,pp.15-118, 1994
98) TAKAYAMA, T. and T. HIRAISHI: Amplification Mechanism of Harbor Oscillation Derived From Field Observation And Numerical Simulation, Technical Note of PHRI No.636, P.70, 1988
99) Hiraishi, T., S. Kawano S. Tamaki and J. Hasegawa: Standard wave spectrum of long period wave employed in the design of port and habour facilities, Proceeding of Coastal Eng. JSCE No.. 44, pp.246-250, 1997
100) Coastal Development Institute of Technology (CDIT): Impact Evaluation Manual for long-period waves in ports, Coastal Technology Library No. 21, CDIT, 2004, 86p.
101) UEDA, S., S. SHIRAISHI, H. OSHIMA and K. ASANO: Allowable Wave Height and Wharf Operation Efficiency Based on the Oscillations of Ships Moored to Quay Walls, Technical Note of PHRI No.779, P.44, 1994
102) Coastal Development Institute of Technology (CDIT): Impact Evaluation Manual for long-period waves in ports, Coastal Technology Library No. 21, CDIT, 2004, 86p.
103) Japan Association of Maritime Safety: Survey and study on fundamental elements of sea transport safety (Report 3)- Effect of ship waves on small crafts, p.83, 1976
104) Japan Shipping Exchange, Inc.: The Annual “Register of Ships” (SENPAKU MEISAISHO)1975,- 1975
105) Takeuchi, T. and J. Shichizawa: So-called “Gunkan-ha” (Ship waves caused by battleships)on Imabari Coast, Journal. of the Oceanographic Society of Japan Vol. 17 No.2, pp.80-90, 1961
106) Ertekin, R.C., W.C. Webster and J.V. Wehausen: Ship generated solution, Proc. 15th Symp. Nav. Hydlodyn., pp.347-364, 1985
107) JSCE: The Collected Formula of Hydraulics (1985 Edition), JSCE, Nov. 1985
108) Goda, Y: Random Seas and Design of Maritime Structures, Kajima Publishing, pp. 323, 1990
109) Study Group on application of numerical wave tank to structural design against waves: Study and development of numerical wave channel, Coastal Development Technology Library, CDIT, pp,156-163, 2001.
110) Mitauyasu, H. : Theory of wave pressure, Summer Training on Hydraulic Engineering B Coast and Harbour 1965, Hydraulic Committee, JSCE pp.B-16-1-B-16-25,1965
111) ITO, Y. M. FUJISHIMA and T. KITATANI: ON THE STABILITY OF BREAKWATERS, Rept. Of PHRI Vol.5 No.4,p.134, 1966
112) Goda, Y. : Wave forces on port facilities, Lecture Note of Summer Training on Hydraulic Engineering B Coast and Harbour 1967, Hydraulic Committee, JSCE pp.B-10-1-B-10-34,1967
113) Tanimoto, K: Wave forces on composite breakwaters, Proceeding of Lecture Meeting of PHRI 1976, pp.1-26, 1976
114) Goda, Y. :Motion of Composite Breakwater on Elastic Foundation under the Action of Impulsive Breaking Wave Pressure, Rept. Of PHRI Vol. 12 No,3, pp. 31-69, 1973
115) Tanimoto, K, K. Moto, S. Ishizuka and Y. Goda: Discussions on formula of design wave forces on breakwaters, Proceedings of 23rd Coastal Eng. JSCE pp.11-16,1976
116) Takahashi, S., K. Tanimoto, K. Shimosako and T. Hosoyamada: Proposal of Pressure coefficient of impact-breaking waves on composite type breakwater considering foundation mound profile, Proceedings of Coastal Eng. JSCE Vol. 39, pp. 676-680, 1992
117) G. Sainflou: Essai sur les diques maritimas verticals, Annaled des Ponts et Chaussees Vol.98, No. 1, PP.5-48,1928
118) I. Hiroi: On a method of estimating the force of waves, Journal of School of Engineering, Tokyo Imperial University, Vol. X, No.1, p.19, 1919
119) Kuroda, S. and T. Ishiwata: Disaster Prevention Engineering, Sankaido Publishing Co., 1960, 276p>
120) Nagai, K, T. Ozawa, Y. Hishira and T. Nishimura: Characteristics of field waves having a bi-modal wave spectrum generated ed by typhoon, Proceeding s of 33rd Coastal Eng. JSCE, ,pp.144-148,1986
121) Tanimoto, K, K. KIMURA, A. P. dos Santos Pinto: Random Wave Forces and Design Wave Periods of Composite Breakwaters under the Action of Double Peaked Spectral Waves, Rept of PHRI Vol. 25 No.2, pp.3-25, 1986
122) Nakata, K. K. Terauchi, H. Nishida and I. Umeki: Characteristics of low crest breakwaters, Proceedings of 31st Coastal Eng. JSCE, ,pp.532-536,1984
123) Mizuno, Y. Y. Sugimoto, K. Kimura, K. Yamanaka and S. Kikuchi: Study of Characteristics of wave force on high crest breakwaters, Proceedings of Coastal Eng. JSCE Vol.
124) TANIMOTO, K., K. KIMURA: A Hydraulic Experimental Study on Trapezoidal Caisson Breakwaters, Technical Note of PHRI No.528,p.28, 1985
125) Nagai, S, T. Kubo and K. Tokikawa: Study on Steel pile breakwater (First Report)Proceeding of Coastal Eng, JSCE ,pp.209-218,1965
126) Hayashi, T, T. Kano, M. Shirai and S. Hattori: Hydraulic characteristics of cylindrical permeable breakwaters, Proceeding of 12th Coastal Eng. JSCE, pp.193-197, 1965
127) Goda, Y. and S. Kakizaki: STUDY ON FINITE AMPLITUDE STANDING WAVES AND THEIR PRESSURES UPON A VERTICAL WALL, Repot of PHRI Vo1.5 No.10,pp.1-57,1966
128) TANIMOTO, K., S. TAKAHASHI and T. KITATANI: Experimental Study of Impact Breaking Wave Forces on a Vertical-wall Caisson of Composite Breakwater, Rept of PHRI Vol. 20 No.2, pp3-39, 1981

129) H. Mitsuyasu: Experimental study on wave force against a wall, Report of Trans. Tech. Res. Inst. No.47, pp.1-39, 1962
130) Honma, M., K. Horikawa and N. Hase: Wave forces on sea dikes, Proceeding of 9th Coastal Eng. JSCE, pp.133- 137,1962
131) MORIHIRA, M. S. KAKIZAKI and T. KIKUYA: EXPERIMENTAL STUDY ON WAVE FORCE DAMPING EFFECTS DUE TO DEFORMED ARTIFICIAL BLOCKS, Rept. PHRI Vol.6 No.4, pp.3-31, 1967
132) GODA, Y. and S. HARANAKA: Title AN EXPERIMENT ON THE SHOCK PRESSURE OF BREAKING WAVES, Technical Note of PHRI No.32, pp.1-18, 1967
133) Horikawa, K. and Y. Noguchi: Relationship between breaker pressure and wave profile acting on a vertical wall, Proceeding of 17th Coastal Eng. JSCE, pp,177-184,1970
134) Fujisaki, H, A. Sasada and Y. Takahashi: Discussion on the construction techniques of breakwaters on steep slope sea bed, Proceeding of Chubu Branch of JSCE, ,pp.181-183,1996
135) TAKAHASHI, S., K. TANIMOTO and S. SUZUMURA: Generation Mechanism of Impulsive Pressure by Breaking Wave on a Vertical Wall, Rept of PHRI, Vol. 22 No.4, pp.3-31, 1983
136) Shimosako, K and N. Osaki: Study on the Application of the Calculation Methods of Wave Forces acting on the Various Type of Composite Breakwaters, Technical Note of PHRI, No.1107,pp.1-14,2005
137) Goda, Y: Motion of Composite Breakwater on Elastic Foundation under the Action of Impulsive Breaking Wave Pressure, Rept. Of PHRI, Vol.12 No.3, pp.3-29, 1973
138) Dynamic response of composite type breakwaters to impact breaking waves, Proceedings of Coastal Engineering JSCE, Vol. 40, pp. 766-770, 1993
139) Tanimoto, K, M. Osato, S. Takaoka, T. Uchida and T. Ikada: An experiment on stability of breakwaters with wave-absorbing blocks against random waves, Proceedings of the 26th Coastal Eng., JSCE, pp,343-347, 1979
140) TANIMOTO, K. S. TAKAHASHI and, K. MIYOSE: Title Experimental Study of Random Wave Forces on Upright Sections of Breakwaters, Repot of PHRI Vol.23 No.3,pp.47-100, 1984.9
141) TAKAHASHI, S. K. TANIMOTO and K. SHIMOSAKO: Title Wave and Block Forces on a Caisson Covered with Wave Dissipating Blocks, Rept. Of PHRI Vol.29 No.1,pp54-75, 1990.3
142) Sekino, T., T. Kadono and T. Imoto: Study on design methods for wave-absorbing block armored breakwaters having un armored parapet, Journal of Public Works Research Institute, Hokkaido Regional Development Bureau No.397, pp,14-20, 1986
143) Tanaka, H., N. Abe and M. Kato: Evaluation of horizontal wave force on wave-absorbing block armored caisson breakwater. Proceedings of the 32nd Conference on Coastal Eng. JSCE, pp.490-494, 1985
144) Kougamu, Y. and K. Tokikawa: Experimental study on the wave-pressure decreasing effect of wave absorbing blocks during the construction, Report of the Public Works Research Institute (PWRI), Hokkaido Regional Development Bureau (HRDB) No. 53, pp.81-95, 1970
145) Shiomi, M., H. Yamamoto, A. Tsugawa, T. Kurosawa and K. Matsumoto: Damages and countermeasures of breakwaters due to the wave force increase at discontinuous points of wave-absorbing blocks, Proceedings of the 41st conference on Coastal Eng. JSCE, pp.791-795,1994
146) TANIMOTO, K. and R. OJIMA: Title Experimental Study of Wave Forces Acting on a Superstructure of Sloping Breakwaters and on Block Type Composite Breakwaters, Technical Note of PHRI No.450 ,p.32, 1983.6
147) Hiromoto, F., H. Nishijima, S. Konno and I. Kimura: Load of blocks of wave-observing block armored breakwaters under wave actions, Proceedings of the 40th Conference on Coastal Eng. JSCE, pp. 347-351, 1983
148) ARIKAWA, T., M. IKEBE, N. OSAKI, T. KURODA, T. ORITA and K. SHIMOSAKO: Title Study on punching shear failures of caisson wall caused by the impulsive force of dissipating blocks Rept of PARI Vol.44 No.1, pp. 3-34, 2005.3
149) Yamaguchi, T., M. Beppu and T. Ono: Experimental study of bumping force of wave-absorbing blocks on vertical walls under the action of impact breaking waves, Proceedings of Coastal Eng. JSCE Vol. 50, pp. 711-715, 2003
150) T. Hosoyamada, S. Takahashi and K. Tanimoto: Application of sloping top caisson breakwater at isolated islands, proceedings of Conference on Coastal Eng. JSCE, Vol. 41, pp.325-329, 1994
151) Morihaira, M. and O. Kunita: An model experiments on hydraulic characteristics of sloping breakwater, Proceedings of the 26th Conference on Coastal Eng. JSCE, ,pp.295-298,1979
152) Sato, T. N. Yamagata, M. Furukawa, S. Takahashi and T. Hosoyamada: Hydraulic characteristics of sloping-top breakwaters armored with wave-absorbing blocks- Development of a new structural type of breakwaters in deep water area in Naha Port-, Proceedings of Coastal Eng. JSCE Vol. 39, pp.556-560,1992
153) Katayama, H., T. Sekimoto, Y. Kawamata and K. Ueki: On wave force characteristics acting on submerged sloping-top caisson breakwaters, Proceedings of Coastal Eng. JSCE Vol. 45, pp. 776-780, 1998
154) TAKAHASHI, S., K. SHIMOSAKO and H. SASAKI: Title Experimental Study on Wave Forces Acting on Perforated Wall Caisson Breakwaters Rept of PHRI Vol. 30 No.4, pp.3-34, 1991.12
155) TANIMOTO, K., S. HARANAKA, E. TOMIDA, Y. IZUMIDA and S. SUZUMURA : Title A Hydraulic Experimental Study on Curved Slit Caisson Rept of PHRI Vol. 19 No.4, pp.3-53, 1980.12
156) TANIMOTO, K., S. TAKAHASHI and T. MURANAGA : Title Uplift Forces on a Ceiling Slab of Wave Dissipating Caisson with a Permeable Front Wall- Analytical Model for Compression of an Enclosed Air Layer-, Rept of PHRI Vol.19 No.1,pp.3-31, 1980.3
157) TAKAHASHI, S. and K. TANIMOTO: Title Uplift Forces on a Ceiling Slab of Wave Dissipating Caisson with a Permeable Front Wall(2nd Report)-Field Data Analysis- Rept of PHRI Vol. 23 No. 2, pp.3-25, 1984.6

158) ITO, FY. And K. TANIMOTO: Title Meandering Damages of Composite Type Breakwaters. Technical Note of PHRI No.112, pp.13, 1971.3
159) Kougami, Y., N. Fujiki and Y. Uehara: An experiment on wave height and pressure at a corner of breakwater, Journal of Public Works Research Institute (PWRI), Hokkaido Regional Development Bureau (HRDB)No. 218, pp.1-11, 1971
160) Y. Kougami, N. Fujiki and T. Tsutsumi: An experiment on wave height and pressure at a corner of breakwater (Second Rep.), Journal of Public Works Research Institute (PWRI), Hokkaido Regional Development Bureau (HRDB)No. 230, pp.1-13,1972
161) Goda, Y, T. YOSHIMURA and M. ITO: Reflection and Diffraction of Water Waves by an Insular Breakwater, Rept of PHRI Vol.10 No.2,pp.3-52, 1971.6
162) ITO, Y., K. TANIMOTO, K. KOBUNE, T. KITATANI and M. TODOROKI: Title An Experimental Investigation of Upright Breakwater s on Reefs, Technical Note of OHRI No.189, 1974.6
163) Tominaga, M. and I. Kuzumi: braking wave pressure on coastal dikes, Proceedings of the 18th Conference on Coastal Eng. JSCE, pp. 215-221, 1971
164) Coastal Engineering Research Center: Shore Protection Manual Vil.II, Department of Army Corps of Engineers, 1984
165) JSCE Edition: Guideline and commentary for design of Offshore Structures (Draft), JSCE, pp.28-53, 1973
166) J.R. Morison, M.R. O'Brien, J.W. Johonson,S.A. Schaaf: The force exerted by surface waves on piles, Petroleum Trans., 189, TP 2846, pp.149-154, 1950
167) T.E. Stelson and R.T. Mavis: Virtual mass and acceleration in fluid, Proc. ASCE., Vol. 81, Separate No.670, pp.670-1-670-9, 1955
168) Hamada, H. and N. Hasegawa: Wave forces on structures, Proceedings of the 3rd Conference on Coastal Eng. JSCE ,pp.67-83, 1956,
169) Keulegan, G.H. and L.H. Carpenter: Forces on cylinders and plates in an oscillating fluid, Journal of the National Bureau of Standards, Vol. 60, No. 5, pp.423-440, 1958
170) Sarpkaya, T.: Forces on cylinders and sheres in a sinusoidally oscillating fluid, Journal of Applied Mechanics, Trans. of ASME, Vol. 42, No. 1, pp. 32-37, 1975
171) Sarpkaya, T.: In-line and transverse forves on cylinders in oscillatory flow at high Reynolds number, Proc. of the Offshore Technology Conference, Vol.11, pp.95-108, 1976
172) Sarpkaya, T., N,J. Collins and S.R. Evans: Wave forces on rough-walled cylinders at high Reynolds numbers, Proc. Of OTC, Vo. 111, #2901, pp.167-184, 1977
173) Y. Goda: Wave-forces on a vertical circular cylinder; Experiments and proposed method of waveforce computation, Report of P.H.T.R.I., No.8, p.74, 1964
174) Yamaguchi, M.: Fundamental study on wave forces on sphere and standing waves, Kyoto Univ. Dissertation ,p.147,1974
175) Nakamura, M.: Study on wave forces on coastal structures and their dynamic response, Osaka Univ. Dissertation ,p.297, 1977
176) Chakrabarti, S.K., A.L. Wollbert, and A.T. William: Wave forces on vertical circular cylinder, Journal of Waterways, Harbors and Coastal Engineering Division, ASCE, Vol.102, No.WW2, pp.203-221, 1976
177) Chakrabarti, S.K.: Inline forces on fixed vertical cylinder in waves, Journal of the Waterway, Port, Cow and Ocean Division, ASCE, Vol.106, WW2, pp.145' 155, 1980
178) Koterayama, W. and M. Tashiro: Wave forces on semi-submerged horizontal cylinder in diagonal incident wave and vertical cylinder in deep water waves, Report of Research Institute for Applied Mechanics Kyushu Univ. Vol. 5o, pp. 35-61, 1979
179) Oda, K.: Wave forces on structures- Cylindrical structures-, Lecture note of Summer training for Hydraulic Engineering 1982, (18th)Course B, Hierodulic Committee of JSCE, pp.B-4-1-B-5-27,1982
180) Mizuno, Y. T. Kadono, Y. Nagai, O. Momose and T. Sekino: Field experiment on offshore structure in a rough sea zone, Rept. of the Public Works Research Institute (PWRI), Hokkaido Regional Development Bureau (HRDB)No.87,p.77,1988
181) Kim, Y.Y. and H.C. Hibbard: Analysis of simultaneous wave force and water particle velocity Measurements, Proc. of OTC, Vol.1, No.2192, pp.461-469, 1975
182) Ogusu, A. : Wave forces on multiple row cylinders, Proceedings of the Society of Naval Architects of Japan No. 131, 1972
183) Nakamura, H. and N. Abe: Hydraulics of intake tower of power plant, Proceedings of the 19th Conference on Coastal Eng. JSCE, pp. 391-394, 1972
184) GODA, Y. S. HARANAKA and M. KITAHATA: Title STUDY OF IMPULSIVE BREAKING WAVE FORCES ON PILES, Rept of PHRI Vol. 5 No.6, pp. 1-30, 1966.4
185) TANIMOTO, K. S. TAKAHASHI, T. KANEKO, K. SHIOTA and K. OGURA: Title Experimental Study on Impulsive Forces by Breaking Waves on Circular Cylinder Rept of PHRI Vol. 25 No.2, pp.33-87, 1986.6
186) Goda, Y. : Design of marine structures- wave forces-, Jour. of JSCE Vo55 No.9, pp.2-7, 1970
187) Nagasaki, S. and K. Ogata: Wave forces on underwater pipelines (First Rept.), Proceedings of the 18th Conference on Coastal Eng. JSCE, pp. 223-227, 1971
188) T. Sarpkaya, M. Isaacson: Mechanics of Wave Forces on Offshore Structure, Van Nostrand Reinhold Company, p.651, 1981
189) T. Yamamoto, J.H. Nath: Forces on many cylinders near a plane boundary, ASCE, National Water Resources and Ocean Engineering Convention, Preprint No.2633, 1976
190) T. Sarpkaya: In-line and transverse forces on cylinders near a wall in oscillatory flow at high Reynolds numbers, OTC paper No.2898, pp.161 166, 1977
191) T. Sarpkaya, F. Rajabi.: Hydrodynamic drag on bottom-mounted smooth and rough cylinders in periodic flow, OTC Paper

No.3761, pp.219-226, 1980
192) Borgman, L.E.: Spectral analysis of ocean wave forces on pilling, Proc. ASCE, Vo1.93, No. W W2, pp.129-156, 1967
193) Hino, M.: Spectral theory of wave forges on piles, Proceedings of the 15th Conference on Coastal Eng. JSCE, pp. 103-108, 1968
194) Borgman, L.E.: Ocean wave simulation for engineering design, Proc. ASCE, VoL95,No,WW4,pp.557-583, 1969
195) ITO, Y. K. TANIMOTO and K. KOBUNE: Title Dynamic Response of an Offshore Platform to Random Waves Rept of PHRI Vol. 11 No.3, pp.59-86, 1972.9
196) Hudspeth, R.T.: Wave force prediction from non-linear random sea simulation, 7th OTC, # 2193, pp.471-486, 1975
197) Sharma, J. and Dean, R.G.: Second-order directional seas and associated wave forces, 11th OTC, #3645, pp.2505-2514, 1979
198) Tickell, R.G. and Elwany, M.H.S.: A probabilistic description of forces on a member in a short-crested random sea, Mechanics of Wave-Induced Forces on Cylinders, Pitman Pub. Ltd., London, pp.561-576, 1979
199) Kimura, A. A., Seyama and A. Ueno: Stochastic characteristics of random wave forces acting on cylinder, Proceedings of 27th Conference on Coastal Eng. JSCE ,pp. 373-377, 1980
200) GODA, Y. T. IKEDA, T. SASADA and Y. KISHIDA: Title Study on Design Wave Forces on Circular Cylinders Erected upon Reefs Rept of PHRI Vol. 11 No.4, PP.45-81, 1972.12
201) Hiraishi, T. Y. Tomita and Y. Suzuki: Effect of wave directionality on wave forces on Cylinder, Proceedings of 27th Conference on Coastal Eng. JSCE Vol. 41,pp. 836-840, 1994
202) MacCamy, R.C. and R.A. Fuchs: Wave Forces on Piles, a Diffraction Theory, U.S. Army, Corps of Engineers, Beach Erosion Board, Tech. Memo. No.69, p.17, 1954
203) GODA, Y. and T. YOSHIMURA: Title Wave Force Computation for Structures of Large Diameter, Isolated in the Offshore, Rept of PHRI Vol. 10 No.4, pp.3-52, 1971.12
204) Yamaguchi, M: Wave forces on pile structures, Lecture note of Summer training for Hydraulic Engineering 1975, (11th) Course B, Hierodulic Committee of JSCE, pp.B-6-1-B-6-26,1975
205) TANIMOTO, K., S. TAKAHASHI and Y. IZUMIDA: A Calculation Method of Uplift Forces on a Horizontal Platform, Rept. of PHRI Vol.17 No.2,pp.3-47,1978
206) ITO, Y. and H. TAKEDA: Uplift on Pier Deck due to Wave Motion, Rept. of PHRI Vol. 6 No. 4, pp. 37-68, 1967
207) Murota, A. and M. Furudoi: Experimental study on uplift force on deck of pier, Proceedings of the 13th Conference on Coastal Eng. JSCE, pp.i20-125,1966
208) Nagai, S. T. Kobo and K. Tokikawa: Study on uplift on piers (First Rep.), Proceedings of the13th Conference on Coastal Eng. JSCE, pp. 112-119, 1966
209) Horikawa, K., T. Nakao and A. Yajima: Experimental study on uplift on piers, Proceedings of the14th Conference on Coastal Eng. JSCE, pp.148-153,1967
210) Sawaragi, T. and M. Nochino: Discussion on Similarity law for hydraulic model tests on uplift acting on decks of piers, Proceedings of Coastal Eng. JSCE Vol. 35, pp. 677-681, 1988
211) TANIMOTO, K., Shigeo TAKAHASHI, Masahiko TODOROKI and Yoshikazu IZUMIDA: Horizontal Wave Forces on a Rigid Platform, Rept. of PHRI Vol. 16 No.3,pp.39-68,1977
212) Tirindelli, M., Cuomo, G., Allsop, W., and McConnell, K.: Exposed jetties: Inconsistencies and gaps in design methods for wave-induced forces, Proc. Of the 28th International Conference on Coastal Engineering, Cardiff, pp.1684-1696, 2002
213) Tirindelli, M., Cuomo, G., Allsop, W., and McConnell, K.: Physical model studies of wave-induced loading on exposed jetties: Towards new prediction formulae, Proc. Of the International Conference Coastal Structures 2003, Portland,pp,382-393,2003
214) Cuomo, G., Allsop, W., and M.cConnell, K.: Dynamic Wave Loads on Coastal Structures: Analysis of impulsive and pulsating wave loads, Proc. Of the International Conference Coastal Structures 2003, Portland, pp.356-368,2003
215) Bentiba, R., Cuomo, G., Allsop, W., and Bunn, N.: Probability of occurrence of wave loading on fatty deck elements, Proc. Of the 29th International Conference on Coastal Engineering, Lisbon, pp.4113-4125, 2004
216) Kubou, M. M. Takezawa and T. Ueki: Experimental study on wave pressure at the vicinity of wave crest, Proceedings of the 20th Conference on Coastal Eng. JSCE, pp. 279-284, 1973
217) Michio Morihira,Shusaku Kakizaki and Yoshimi Goda :EXPERIMENTAL INVETIGATION OF A CURTAIN-WALL BREAKWATER ,REPORT OF PORT AND HARBOUR TECHNICAL RESEARCH INSTITUTE MINISTRY OF TRANSPOTATION No.1,p.16,1964
218) Sekimpto, T., H. Kondo, T. Oshita, S. Imai and M. Nakamura: Design wave force of curtain-wall type breakwater considering incident wave angle, Proceedings of Coastal Eng. JSCE Vol. 35, pp. 657-661, 1988
219) JSCE: Design Handbook for Shore Protection Facilities (2000). JSCE, pp 301-305, 2000
220) Coastal Development Institute of Technology (CDIT): Commentary on analysis and applications of Coastal Waves, Coastal Observation data, ,p.181,2000
221) Goda Y. Edition: Working Group for Analysis and application of coastal observation data: Measurement of waves (guidebook of coastal wave observation)CDIT, ,p.212,2001
222) Gringorten, I.I : A plotting rule for extreme probability paper, J. Geophysical Res., Vol. 68, No.3, PP.813-814,1963
223) Petruaskas, C. and R.M. Aggaard: Extrapolation of historical storm data for estimating design wave heights, Preprints 2nd OTC, No.1190., 1970
224) Takada, E., K. Morohoshi, T. Hiraishi, T. Nagai and S. Takemura: Distribution of wave, storm and tsunami design conditions on Japanese nationwide coastal structures, Technical Note of PARI No.88,2003

225) Goda, Y.: Numerical Investigations on Plotting Formulas and Confidence Intervals of Return Values in Extreme statistics, Rept. of PHRI Vol. 27 No. 1, pp. 31-92, 1988
226) Goda, Y: Random Seas and Design of Maritime Structures: (Enlarged Edition), Kajima Publishing, Chapter 11、pp.265-322,1990 (in Japanese)
227) Goda, Y. Demonstrative study on Distribution generating function of extreme wave statistics, Proceedings of Coastal Eng. JSCE Vol. 45, JSCE, p.211-215,1998
228) Coastal Development Institute of Technology (CDIT): Impact Evaluation Manual for long-period waves in ports, Coastal Technology Library No. 21, CDIT, 2004, 86p.
229) NAGAI, T and H. OGAWA: Annual Report on Nationwide Ocean Wave information network for Ports and Harbors (NOWPHAS 2002), Technical Note of PARI, No.1069, p.336, 2004
230) Nagai, T: Long Term Statistics Report on Nationwide Ocean Wave information network for Ports and Harbors (NOWPHAS 1970- 1999), Technical Note of PARI No.1035,p.336,2002
231) Bureau of Port and harbours: Design Manual for prestressed concrete port facilities, ,1987
232) JSCE: Design Manual for Coastal Protection Facilities, JSCE. 2000, pp. 31-68
233) TAKAYAMA, T. and H. FUJI: Probabilistic Estimation of Stability of Slide for Caisson Type Breakwater Author Tomotsuka, Rept. of PHRI Vol. 30 No. 4, pp. 35-64, 1991
234) HIRAYAMA, K.: Utilization of Numerical Simulation on Nonlinear Irregular Wave for Port and Harbor Design, Technical Note of PARI No. 1036, pp. 162, 2002
235) HIRAISHI, T.: Generation and Application of Directional Random Waves in a Laboratory, Technical Note of PHRI, Technical Note of PHRI, No.723 1992.3
236) Coastal Development Institute of Technology: Manual for the evaluation of Long-period waves in harbors, Coastal Technology Library No. 21,pp. 86, 2004
237) HIRAYAMA, K., Yasuhiko MINAMI, Mitsuhiro OKUNO, Koji IMAMURA, Hiroyasu KAWAI and Tetsuya IRISH: Case Study on Wave Disaster due to Typhoons in 2004,, Technical Note of PAIR No .1101, pp. 42, 2005
238) Hirayam,a, K. and K. Minemura: simple estimation methods of the amplification rate of wave height at river mouths, Proceedings offshore Development VoL21, pp. 313-318, 2005
239) Coastal Development Institute of Technology: Technical Manual for Floating Structures, 1991 (in Japanese)
240) Kanda, K., Shiraishi, N. and Takino, Y.: On the Peculiarity of Shaking for Floating Wharfs, Proc. of Civil Engineering in the Ocean, JSCE, Vol.2, pp.163-168, 1986 (in Japanese)
241) Toyoda, S.: Floating Offshore Oil Storage System, Proc. of the 4th Ocean Engineering Symposium, SNAJ, pp.25-50, 1979 (in Japanese)
242) Kogure, K., Suzuki, Y., Katagiri, M. and Yakuwa, T.: Design and Construction of Floating Breakwater, Proc. of the 30th Japanese Conference on Coastal Engineering, JSCE, pp.435-439, 1983 (in Japanese)
243) Maruyama, T.: On the Construction of Swing and Floating Bridge, Proc. of the 14th Ocean Engineering Symposium, SNAJ, pp.327-334, 1998 (in Japanese)
244) Osaka City: Construction Report of Yumemai Bridge, Public Works Bureau, Osaka City, 2002 (in Japanese)
245) Miyamoto, T.: Floating Disaster Prevention Basis, Cargo Handling Japan, Japan Cargo Handling Mechanization Association, Vol.44 No.3, pp.316-321, 1999 (in Japanese)
246) Kozawa, T., Miyachi, Y., Koizumi, T., Wada, K. and Matsushita, Y.: Provision of Floating Disaster Prevention Basis, Proc. of Techno-Ocean 2000 International Symposium, Techno-Ocean Network, pp.623-628, 2000
247) Coastal Development Institute of Technology and Floating Structures Association of Japan: Report on Study of Large-scale Floating Structures, 1995 (in Japanese)
248) Technological Research Association of Mega-Float: Report on the Fiscal 1997 Research Results of Mega-Float Floating Offshore Structure, 1998 (in Japanese)
249) Floating Structures Association of Japan: Large-scale Floating Structures, Kajima Institute Publishing, 2000 (in Japanese)
250) Technological Research Association of Mega-Float: Empirical Research on Airport Applicability of Mega-Float (Report on Technical Research Results of Mega-Float) -Brief Overview of Results in Phase II and the Fiscal 2000 Research Results-, 2001 (in Japanese)
251) Takayama, T., Hiraishi, T., Furukawa, M., Sao, K. and Tachino, S.: Field Observation of Motions of a SALM Buoy and Tensions of Mooring Hawsers, Technical Note of Port and Harbour Research Institute, No.542, 1985 (in Japanese)
252) Iwatani, F., Miyai, S. and Mishina, K.: Observations of Sea Waves with the New Structure System in Sakata Port, Proc. of Civil Engineering in the Ocean, JSCE, Vol.2, pp.97-102, 1986 (in Japanese)
253) Tsuji, T., Mori, N. and Yamanouchi, Y.: On the Force Acting on a Ship in Oblique Flow (Restricted Water Effects), Report of Ship Research Institute, Vol.6 No.5, pp.15-28, 1969 (in Japanese)
254) Nojiri, N. and Murayama, K.: A Study on the Drifting Force on Two-Dimensional Floating Body in Regular Waves, Transactions of the West-Japan Society of Naval Architects, No.51, pp.131-152, 1976 (in Japanese)
255) Motora, S., Koyama, T., Fujino, M. and Maeda, H.: Dynamics of Ships and Offshore Structures -revised edition-, Seizando, pp.220-224, 1997 (in Japanese)
256) Coastal Engineering Committee, JSCE: Coastal Waves, Japan Society of Civil Engineers, pp.291-429, 1994 (in Japanese)

257) Akishima Laboratories (Mitsui Zosen): Fifteen Years of Akishima Laboratories -History of Research and Development-, pp.105-130, 1995 (in Japanese)
258) Ito, Y. and Chiba, S.: An Approximate Theory of Floating Breakwaters, Report of Port and Harbour Research Institute, Vol.11 No.2, pp.15-28, 1972 (in Japanese)
259) Port and Harbour Research Institute and District Port Construction Bureaus, Ministry of Transport: Report on Development of Floating Structures, 1985 (in Japanese)
260) Ocean Engineering Committee, SNAJ: Mega-Float Offshore Structure,Seizando, 1995 (in Japanese)
261) Ueda, S. and Shiraishi, S.: Method and Its Evaluation for Computation of Moored Ship's Motions, Report of Port and Harbour Research Institute, Vol.22 No.4, pp.181-218, 1983 (in Japanese)
262) Cummins, W.E.: The Impulse Response Function and Ship Motions, Schiffstechnik, Bd.9 Helf 47, pp.101-109, 1962
263) Kubo, M. and Okamoto, S.: A Study on Accuracy Improvement of Time Series Analysis Using Retarded Function for a Floating Body Moored to Quay Walls, Proc. of the 34th Japanese Conference on Coastal Engineering, JSCE, pp.611-615, 1987 (in Japanese)
264) Suzuki, Y. and Moroishi, K.: On the Motions of Ships Moored to Single-point Mooring Systems, Report of Port and Harbour Research Institute, Vol.21 No.2, pp.107-150, 1982 (in Japanese)
265) Suzuki, Y.: Study on the Design of Single Point Buoy Mooring, Technical Note of Port and Harbour Research Institute, No.829, 1996 (in Japanese)
266) Ueda, S.: Analytical Method of Ship Motions Moored to Quay Walls and the Applications, Technical Note of Port and Harbour Research Institute, No.504, 1984 (in Japanese)