5 Tsunamis

Public Notice

Tsunamis

Article 9
Tsunamis shall be appropriately defined in terms of the tsunami height and others based on historical tsunami records or numerical analyses.

[Commentary]

Specification of Tsunamis:
The tsunami parameters shall be specified appropriately based on historical tsunami disaster records and/or results of numerical analysis that include estimation of initial conditions for tsunamis generated by earthquakes and others.

[Technical Note]

(1) Definition of Terminology Related to Tsunamis

① Tsunami
A tsunami is a series of waves that mainly occurs when an earthquake causes sudden uplift and subsidence of the sea bottom, leading to a vertical fluctuation of the sea surface that gets transmitted to the coast. Other causes of tsunamis are large landslides near the coast and under sea, the eruptions of undersea volcanoes, and other impacts upon the sea surface including meteorites.

The displacement of the sea bottom due to an earthquake may extend for several tens of kilometers or more in the relatively shallower water several kilometers deep at most. The bottom movement in the considerably wide and thin layer is directly transmitted to the sea surface. This sea surface movement becomes the initial tsunami profile with extremely long wavelength compared to the water depth. The sea surface movement is then transmitted outward as a long wave.

Terminology of the tsunamis is as shown in **Fig. 5.1** Tsunami Terminology.

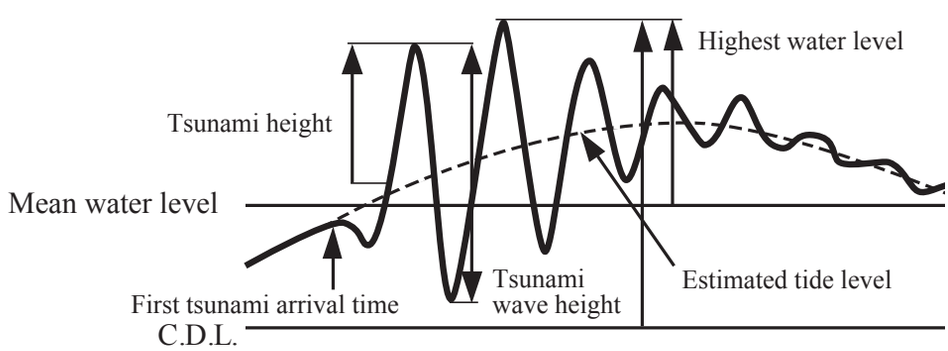

Fig. 5.1 Tsunami Terminology

② Estimated tide level (Ordinary tide level)
This is the estimated level of the sea surface when there is no tsunami. It is obtained by smoothing the tide level on a tide observation record by removing the components that are considered to be of the tsunami and any oscillation components of shorter period by seiche. The estimated tide level is basically the astronomical tide level, however it may be deviated from the astronomical tide level calculated from harmonic components of tide due to factors such as atmospheric pressure changes, winds, and changes in ocean currents near the shore.

③ Tsunami height
The absolute value of the difference between a peak or trough of the actual tide level and the estimated tide level is referred to as deviation. The maximum value of the deviation when the actual tide level is higher than the estimated one is referred to as the maximum deviation or the tsunami height. It is necessary to recognize that the tsunami height is different from the tsunami wave height as described later.

④ Highest water level
The maximum value of the tide level that is measured during a tsunami is called the highest water level.

⑤ Tsunami wave height and period

Time series of tsunami is usually irregular. In the same way as analysis of wind waves, the tsunami can be analyzed by the zero up-crossing method to define the tsunami wave height and period for an individual wave. An individual wave is defined to extend from a point where the observed sea surface water level crosses over the estimated tide level from the negative side to the positive side, to the next such point. The difference between the highest water level and the lowest water level within the individual wave is defined as the tsunami wave height, while the time duration of the individual wave is defined as the tsunami period. Finally, the highest value within a series of tsunami wave heights is called the highest tsunami wave height.

⑥ Initial movement

This refers to the instance at which a tsunami reaches the observation point and the observed tide level starts to shift from the estimated tide level. When the first observed sea surface water level change due to the tsunami is higher than the estimated tide level, such initial motion is referred to as the pushing initial motion, when it is lower than the estimated tide level, the initial motion is called the drawing initial motion.

⑦ Runup height and tsunami trace height

The runup height is the elevation to which a tsunami has runup the land or a structure. The height of tsunami is often determined by the trace that the tsunami leaves at that location, and the height of that trace is also called the tsunami trace height.

(2) Tsunami Period

The predominant period of a tsunami depends on factors such as the size of the source area of the tsunami, the distance from the epicenter, and the resonance characteristics of a bay. Since the tsunami that hits the shore is generally not regular waves with a simple period, but rather random waves, it may have components whose periods are the same as the natural frequency periods of the bay or harbor and which are amplified through resonance. During performance verification, it is necessary to investigate characteristics and effects of tsunamis that have not only predominant periods of the past and possible tsunamis but also the periods that are the same as the natural frequency periods of bays and harbors.

(3) Tsunami Wave Celerity

Because a tsunami is a long wave, the tsunami wave celerity C is a function only of the water depth, as in the following formula:

$$C = \sqrt{gh} \tag{5.1}$$

where

C : wave celerity (m/s)
g : gravitational acceleration (m/s^2)
h : water depth (m)

For example, the tsunami wave celerity would be 713 km/hour in the average depth of the Pacific Ocean which is 4000 meters. In 1960, the tsunami that formed off the coast of Chile reached Japan about one day later. At the shore, with a depth of 20 meters, the wave celerity decreases to 50 km/hour.

If the tsunami arrival times are known at many locations as well as bathymetry then it is possible to calculate inversely the source area of the tsunami by means of the characteristics that the tsunami wave electricity depends on the water depth only.

(4) Tsunami Transformation

① Wave shoaling, refraction, and diffraction

In the deep sea, the spatial scale of a tsunami is several tens of kilometers or more, while the vertical fluctuation is only about several meters. Tsunami is not prominent at the deep sea. However, the tsunami is transformed by wave shoaling and refraction in the same way as wind waves. This process provides the increase of tsunami wave height, resulting in being visible near the shore. In addition, it is affected by local topological features along the shore, on a scale of 200 to 300 meters, making it possible for the tsunami to runup the shore 20 to 30 meters. For example, the tsunami by the 1993 Hokkaido-oki earthquake (the 1993 Okushiri Tsunami) ran up 32 meters at the V-shaped cliff area of Okushiri Island.[1), 2)] Also, a tsunami can be concentrated in a cape area due to refraction induced by bathymetry change off the cape. Further, due to diffraction, a tsunami wave may reach the opposite side of an island, cape or as seen from the direction of approach of the tsunami. For example, the 1993 Okushiri tsunami approached Okushiri Island from the west side, but tsunami damage also occurred on the island's east side as well as the west side, and the 2004 Indian Ocean tsunami reached Sri Lankan Island from the east side but the tsunami of about 5 meters also hit the southwest shore.

② Transformation of tsunamis within bays

A tsunami increases its wave height and fluid velocity, if it propagates into a bay where the water depth becomes shallower and the wave ray becomes narrower toward the end of the bay. If the ratio of the wave height to the

water depth is small then the wave height can be calculated from Green's law, as shown in equation **(5.2)**:

$$\frac{H_2}{H_1} = \left(\frac{b_1}{b_2}\right)^{1/2} \left(\frac{h_1}{h_2}\right)^{1/4} \tag{5.2}$$

where

H_1 : the tsunami wave height for a cross-section of width b_1 and water depth h_1
H_2 : the tsunami wave height for a cross-section of width b_2 and water depth h_2

However, equation **(5.2)** only holds if one assumes that the width and water depth change gradually and there is no reflected wave, and it does not consider energy loss due to sea bottom friction. It cannot be applied in places such as shallow water area and the inner portion of a bay where there is a strong effect of the reflected wave.

(5) Bore Type Tsunamis [3)]

A remarkable characteristic of the tsunami by the 1983 Nihonkai-Chubu earthquake (the 1983 sea of Japan Earthquake) occurred along the northern shore of Akita prefecture where the shore has a mild bottom slope of about 1/200 that extends for 30 km. Propagating toward the shore, the tsunami was greatly deformed into a bore, accompanied by a short periodic waves of about 5 to 10 seconds. On the other hand, when this same tsunami hit a shore with a relatively steep slope of about 1/50, such as the western shore of Oga peninsula, it did not become a pronounced bore type tsunami, but rather similar to standing waves. For incoming tsunamis with the same tsunami height, a bore type tsunami tends to have a greater runup height than a standing-wave type tsunami.

(6) Edge Waves

If propagates onto a continental shelf obliquely from the deep sea, the wave refraction can make the tsunami reflected from a coast propagate along the coast and consequently part of energy of the tsunami can be trapped near the coast. Such a wave is referred to as an edge wave. For example, the tsunami by the 2003 Tokachi-oki earthquake in the sea off Tokachi in Hokkaido, a tsunami that could be considered an edge wave was detected along the coast from Cape Erimo to Kushiro in the southeastern coast of Hokkaido, Japan. The fact that a tsunami can continue for a long time due to formation of edge waves means the increase of possibility that the tsunami can meet a high tide resulting in inundation in coastal areas.[4)]

(7) Tsunami Wave Force

Tsunami wave force on an upright wall may be determined as in **Fig. 5.2**, in which the wave pressure distribution can be assumed as a linear distribution with a value of $p = 0$ at a height of $\eta^* = 3.0\, a_1$ above the still water level and a value of $p_1 = 2.2\, \rho_0 g a_1$ at the still water level, and a constant value for the wave pressure below the still water level.[3)]

$$\eta^* = 3.0 a_I \tag{5.3}$$

$$p_I = 2.2\, \rho_0 g a_I \tag{5.4}$$

$$p_u = p_1 \tag{5.5}$$

where

η^* : wave pressure acting height above the still water surface (m)
a_I : incident tsunami height (m)
$\rho_0 g$: unit weight of the seawater (kN/m^3)
p_1 : wave pressure intensity at the still water surface (kN/m^2)
p_u : uplift pressure at the lower edge of the front surface (kN/m^2) $p_I = 2.2\, \rho_0 g a_I$

For a bore type tsunami, the still water surface is the level of the water just before the incident of the tsunami.

For a non-breaking type of tsunami, the tsunami wave height H_I may be related to the incident tsunami height as follows:

$$H_I = 2 a_I \tag{5.6}$$

Conducting a numerical simulation with breakwaters, the tsunami wave height in front of the breakwaters is about twice the value when there are no breakwaters because of adding the reflected tsunami. In this case the highest water level in front of a breakwater, measured from the still water surface, may be taken as equal to the incident wave height.

The wave force of the tsunami with soliton fission may be calculated by means of empirical formulae based on experimental results.[5)]

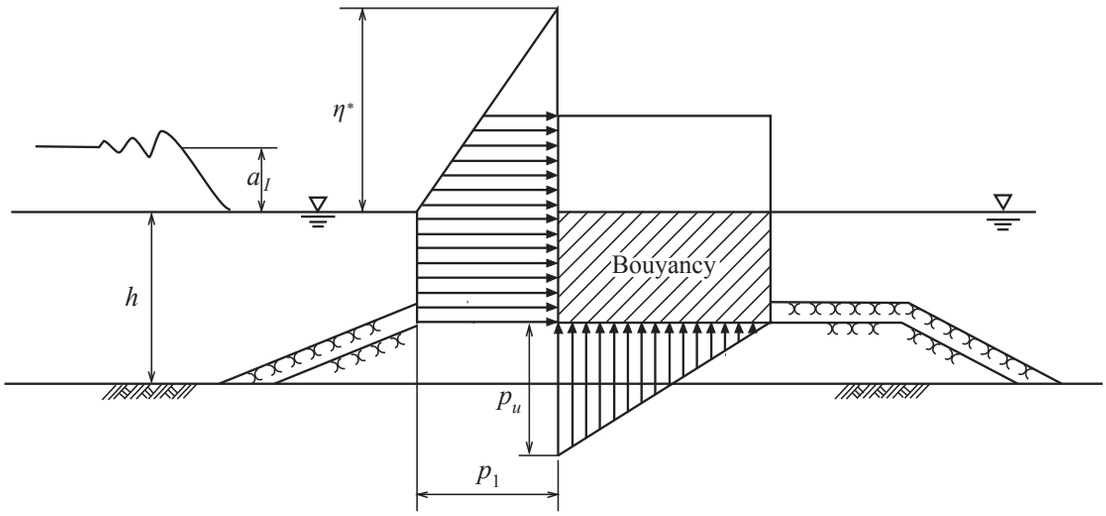

Fig. 5.2 Description of the Tsunami Wave Force

(8) Fluid Velocity of Tsunami

For a tsunami, different from the case of a wind wave, water particles can move even near the sea bottom as well as around the sea surface. Usually, the movement of the sea water due to a tsunami is uniform from the sea surface to the sea bottom, and the fluid velocity u may be given by equation **(5.7)**. As shown by this equation, the fluid velocity of tsunami is faster in the shallower water.

$$u = \frac{C\eta}{h} = \eta\sqrt{\frac{g}{h}} \tag{5.7}$$

where

η : sea surface deviation due to the tsunami (m)
C : wave celerity (m/s)
h : water depth (h)
g : gravitational acceleration (m/s^2)

(9) Tsunamis in Tide Records

① Tide records are extremely useful as records of tsunamis. However, when using such data it is necessary to keep in mind the following items.[1)]

② Tsunami records measured at a tide station within a harbor may indicate different characteristics of tsunami, from those in the area outside the harbor, because they are affected by facilities such as breakwaters.

③ A tsunami with a relatively short wave period will have an energy loss as the water flows through the tide station's inlet pipe until it enters the tide well, so that it is measured to be a smaller tsunami wave than the one that exists around the tide station.

(10) Model Experiments of Tsunamis

In model experiments, by reproducing tsunami profiles determined by numerical simulations at the boundary in a wave basin or flume, it is possible to investigate the stability and protective effect of tsunami breakwaters [6)] and the effect of topological alterations such as reclamations on tsunamis. The scouring of a breakwater entrance mound by the 1993 Okushiri tsunami has been investigated in the model experiments.[7)]

(11) Numerical Simulations of Tsunamis

① Numerical simulations of tsunamis must use appropriate numerical models which are based on fundamental equations that can reproduce the subject tsunamis. The following two types of theories are mainly used for a regional tsunamis that occurs near the coast:

(a) Non-dispersive long wave theories [8)] : Among these, there are the linear long wave theory that applies to waves whose wavelengths are long compared to the water depth, and also the ratio of wave height to water depth is small, and nonlinear long wave theories that apply to long waves when the ratio of wave height to water depth is not small. According to Shuto, [9)] the linear long wave theory may be applied in the water 200 meters or deeper.

(b) Dispersive long wave theories: For a dispersive tsunami, such as the wave observed near the coast for the tsunami by the 1983 Nihonkai-Chubu earthquake, the phenomenon can be explained better with a non-linear dispersed-wave theory.[10), 11)] A nonlinear dispersive wave theory includes factors that take wave dispersion into account (dispersion terms) to a nonlinear long wave theory.

For a distant tsunami (teletsunami) that originates from a far away source, such as the 1960 Chilean Tsunami that traveled across the Pacific Ocean from the Chilean coast to Japanese coast, it is possible to use linear dispersive wave theories, which add dispersion terms to the linear long wave theory. Since the tsunami is a series of waves whose components have various periods, and a wave component with longer period has slightly faster wave celerity. The difference of the celerity is usually negligible small for the tsunami traveling short distance, but is not negligible for those traveling long distance. Further, accurate calculation of a distant tsunami generally need to consider the Coriolis force and to use the spherical coordinates.

② In numerical simulations of tsunami for a time series of the tsunami is provided as the boundary conditions for the calculation region, and an initial tsunami profile in the source area as the initial condition. The initial tsunami profile may be calculated as being the same as displacement of the sea bottom by the earthquake. The displacement may be calculated from an earthquake fault model [12)] by means of the elastic theory of Mansinha and Smylie [13)] and others In another way to setup the initial tsunami profile, the asperity of the fault is recently considered.

③ In order to calculate the tsunami runup on the land, the method of Iwasaki and Mano, [14)] or improvements on it [15)] can be used. If the tsunami overflows structures such as breakwaters or seawalls, it is possible to use the Honma formula [16), 17)] to calculate the amount of overflow for a unit width. In order to evaluate tsunami reduction effect of breakwaters, and other structures, momentum loss due to such facilities should be considered. The momentum loss, which is proportional to the mean flow velocity, includes the sea bottom friction that can be evaluated by Manning's roughness coefficients and others, and the momentum loss due to abrupt narrowing and widening of the cross-section as seen in the opening section of breakwater. Comparing model experiments [18)] with numerical simulations for the breakwaters at the entrance to Kamaishi Bay provided a value of 0.5 for the coefficient of the momentum loss due to the breakwaters.[8)]

Recently it has also become possible to evaluate the flows near submerged breakwaters in the opening section of tsunami breakwater as well as the tsunami wave force that acts on the submerged breakwaters [19), 20)] by using non-hydrostatic and three-dimensional numerical models, and by using such models it should be possible to calculate characteristics and effects of tsunami in detail including direct simulation of tsunami wave force.

(12) Determination of Tsunami for the Performance Verification of Facilities

For establishment of measures to prevent and reduce tsunami damage, it is necessary to presume tsunamis in the subject area and to estimate tsunami parameters such as tsunami height, wave runup height, and arrival times by simulations such as appropriate numerical calculations or model experiments.

The tsunamis that are used for performance verification of facilities shall be determined based on the conditions of the area, such as the coastal landscape, environment, coast use, and economics taking the largest tsunamis that are estimated by historical tsunami records in the subject region and possible tsunami into consideration. The expected tsunamis are as follows:

① The largest tsunami that has previously hit the subject region.

② Among recent tsunamis with a comparatively large amount of data, the tsunami whose scale is considered appropriate for disaster prevention.

③ Expected tsunamis in a seismic gap region which is experienced little or no earthquake activity for a long period.

With regard to protective facilities such as to safeguard the lives and property of the people of an area, it is important to evaluate safety and protective ability of the facilities against the largest tsunamis that are expected to occur in that area. It is necessary to verify the safety and protective ability against the largest tsunamis calculated by means of fault models of previous earthquakes and possible tsunamis.

Since facilities may often suffer damage by earthquake motion before the arrival of a tsunami, it is important to consider the earthquake resistance of facilities.

Recently, a GPS buoy system has been developed, where the horizontal and vertical position of the buoy is determined by analysis of signals from the Global Positioning System. A sampling interval of a second or less, makes it possible to measure sea surface fluctuation with various periods, such as tide motion, tsunamis, storm surge, and waves. It is expected that it will be possible to use measurement records of tsunamis in the deep sea for the performance verification of facilities.

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6 Water Currents

6.1 The Flow of Sea Water in Coastal Zone

Public Notice

Flow of Sea Water

Article 10
Flow of water in the sea or others shall be appropriately defined in terms of the current velocity and direction based on field measurements or numerical computation.

[Commentary]

Setting Methods for the Flow of Sea Water
In the performance verification of facilities subject to the technical standards, when combining the flow of sea water, with other actions, out of all the possible flows of sea water that have a high probability of occurring simultaneously with other actions, specify the current velocities and current directions that would be the severest conditions from the viewpoint of the stability of the target facilities.

[Technical Note]

(1) General
Movements of sea water are superpositions of currents that have various periods, caused by different natural actions, and their current velocities and current directions are greatly affected by topography and structures, and change in complicated ways both in space and in time. Movements of sea water cause sediment on the sea bottom to move, causing problems such as siltation in navigation channels and basins and scouring of the area around facilities. Also, the flow of sea water due to coastal development can cause wide-scale changes in the natural environment, such as water quality, sedimentation changes, and biological changes.
With regard to their origins and their scales over time and space, flows of sea water are classified as ocean currents, tide currents, wind-driven currents, density currents, and nearshore currents. These currents are greatly affected by maritime meteorological and geological conditions, showing flow patterns that are unique to specific regions of the sea.

6.2 Estuarine Hydraulics

Public Notice

Estuarine Hydraulics

Article 11
Influence of estuarine hydraulics shall be assessed with appropriate methods by taking into account the river flow based on field measurements or numerical computation.

[Commentary]

Effect of Estuarine Hydraulics
Effects of estuarine hydraulics include such factors as tides in rivers, river runoff, density currents at river mouth, waves entering into river mouth, and siltation. Their evaluation shall be performed appropriately taking into account of the action from the seaside on estuaries, the river flow and sand discharge from rivers.

[Technical Note]

(1) General
The range over which to define estuarine hydraulics is not necessarily clear, and if it is broadly taken as the area over which fresh water and sea water interact then that is a large area extending from the limit of tidal influence in the upstream river to the mouth of the bay. However, from the viewpoint of actions and effects that are related to port facilities, the estuarine area is generally defined as extending from the upstream point where salt water reaches by average tidal motion to the front portion of the estuarine terrace that is composed mainly of sand expelling during floods (hereafter this will simply be called the estuarine areas). In the estuarine area, in addition to actions such as tidal currents, tide motion, waves, and nearshore currents, there are also river current fluctuations from the seaside, such as the outflow of river flood or drought. As the state of water motion and water density change, there are complex hydraulic phenomena such as density currents, and sediment movement phenomena such as chemical flocculation and settlement. In an estuarine area, organisms live in a fine balance

of the physical environment and chemical environment, and the natural environment and biological environment in the estuarine area can easily be influenced by human activities, so that the development of facilities requires sufficient study and continuous monitoring of such influences.

(2) Tidal Motion, Waves, and Water Currents in an Estuary

In an estuary there are complex hydraulic phenomena as a result of the mixture of the actions such as tidal level fluctuation and tide currents caused by tide motion, water level rises due to waves, and fluctuation of nearshore currents. There are still many issues to be resolved to take all of these factors into account in calculating the movement of water, but with regard to the intrusion of tidal currents into river channels, owing to factors such as the river bed slope and the river current, the duration of flood tide is shorter, and that of ebb tide is longer, so that the maximum and minimum values of the current velocity and discharge occur later than the times of high and low water. These various phenomena vary in time and space in accordance with the location of the river mouth, its shape, and the hydraulic capacity of the river and outer sea. In general, the current at the river mouth can be characterized as follows.

① The current is strong when the river flood, so that the gradient current of the river is predominant, which is a uniform flow.

② When the river is ordinary water, the characteristics of the water flow are complicated because that tidal currents and density currents add into the gradient current.

③ During times of drought the tidal current characteristics take precedence. However, in estuaries where the tidal range is small, the tide current is not so strong, and the density current characteristic is intensified.

④ In estuaries where the tidal range is large, the tide current characteristics tend to predominate.

(3) Density Currents at River Mouth

In an estuary, where river water meets sea water, the sea water penetrates the lower layer of the river water due to the difference in their densities, and there is a mixing of their flows in order to achieve dynamic equilibrium. These flows are called 'density currents at river mouth'. They are divided into three main types, called weakly mixing, moderate mixing, and fully mixing, depending on how the density layers form in the river water and sea water. However, in fact, this changes depending on the time of the tides and the season.

(4) Waves Entering into River Mouth

When waves enter an estuary, the waves are transformed by the effects of the topography and the river currents. The wave height increases due to refraction and concentration caused by the topography of the estuarine terrace and due to wave shoaling. The wave propagation is reduced by the river currents that oppose the direction of wave propagation, increasing the wave height. As the incident waves whose height has been increased run up the river channel, they are attenuated by the wave breakings, the bottom friction, and turbulent flow. Also, when the river current is extremely fast, the waves are unable to run up against the current.

(5) Currents in an Estuary and Sediment Movement

The sediment within the estuary of a bay is mostly sand, containing small particles of substances such as clay and silt. The sediment moves under the action of wave motion, forming characteristic tidal flats, sand spits, river mouth terraces, and bars in the estuary. Such movement of sand is called littoral drift. Also, the fine particles are widely dispersed as they are suspended in the currents, accumulating in calm areas such as waterways and basins, or in places of slower currents in the harbor, causing problems for port facility maintenance and environmental management.

In places where there is a large accumulation of fine particles, the movement of high density mud and accumulation of mud due to sediment cohesion is specifically called 'siltation'. The main difference between 'siltation' and 'littoral drift' is that mud flocculates through mixing with sea water in the estuary, which greatly changes its settling characteristics. Also, the mud that deposit to the sea bottom are capable of changing into a very hard sediment through consolidation over a long period of time. Their ability to be stirred up by the action of waves and water movement is affected by factors such as mud characteristics, the salt content of the sea water, the texture, the water content, and the organic material content, all of which change with time after deposition. These characteristics of mud make difficult to solve problems caused by siltation.

6.3 Littoral Drift [1), 2), 3), 4), 5), 6), 7)]

Public Notice

Littoral Drift

Article 12
Influence of littoral drift shall be assessed by appropriate methods based on field measurements or numerical prediction.

[Commentary]

Effect of Littoral Drift
The evaluation of the effect of littoral drift appropriately takes into account factors such as the sediment grain size, the threshold depth of sediment movement, the longshore sediment transport rate, and the predominant direction of longshore sediment transport.

[Technical Note]

6.3.1 General

(1) When port facilities are to be affected by the littoral drift phenomena, characteristic values of littoral drift shall be established appropriately for sediment grain size, threshold depth of sediment movement, longshore sediment transport rate, and predominant direction of longshore sediment transport.

(2) Littoral drift refers to either the phenomenon that the sediment composing a sea coast or lakeshore is moved by the actions of some forces such as waves and currents, or material itself that is moved by the above processes.

(3) Although the movement of sand by winds and the sand itself that is thus moved are referred to as the wind-blown sand, in the broad definition the littoral drift is also considered to include wind-blown sand at beaches.

(4) Sediment that forms a beach is supplied from nearby rivers, coastal cliffs, and the adjacent coastline. The sediment is exposed to the actions of waves and currents during the supply process or after it has accreted on the beach. This is why the sediment shows characteristics that reflect the characteristics of external forces such as waves and currents. This is referred to as the sediment sorting action by external forces.

(5) As a natural beach is repeatedly subjected to process of erosion when storm waves attack and that of accretion during periods when waves are moderate, it achieves a relatively balanced topography over a long period. This balance may be lost by a reduction in the supply of sand owing to river improvements, by changes in sand supply conditions following construction of coastal structures, and by changes in external forces such as waves and currents. Then beach deformation will occur as the beach moves toward new equilibrium conditions. When building structures such as breakwaters, groins, detached breakwaters, and training jetties, careful attention should be paid to the changes that the construction works will bring about in the balance of the beach. Topographical changes that might be induced by a construction project should be sufficiently investigated in advance. In addition, careful attention should be paid to the deformation conditions of the beach both during construction and following completion of any structure, and appropriate coastal protection countermeasures are recommended to be taken any time when there are concerns about the possibility for disaster triggered by coastal erosion.

(6) When waves approach a coast from offshore, the movement of water particles near the sea floor does not have the force to move the sediment in places where the water depth is sufficiently deep. At a certain water depth, however, the sediment begins to move. The water depth at this boundary where sediment begins to move is called the threshold depth of sediment movement. Sato [1)] studied the movement of sediment by placing radioactive glass sand on the sea floor and investigating the distribution of their movement. From this study, he defined two conditions that are called the surface layer sediment movement and the complete sediment movement, respectively. He applied the former term to the situation in which the sand in the surface layer on the sea floor is moved collectively in the direction of wave movement. The latter term he applied to the situation that the sand shows striking movement with a distinctly visible change in water depth.

(7) Longshore sediment transport rate refers to the rate of littoral drift in the direction parallel to the coast that is caused by waves obliquely incident to a coast.

(8) Longshore sediment moves in either the right or left direction along a coast, corresponding to the direction of incoming waves. The direction with the larger volume of movement during a year is called the predominant direction.

(9) Littoral drift parallel to the coastal line is called longshore sediment transport. In the long term, the topographical changes due to longshore sediment transport are often irreversible. For example, considering topographical

changes near a groin, if waves come in from the right side, looking out toward the sea from the coast, there will be accumulation on the right side of the groin, and erosion on the left side. Or, if the waves come in from the left side then the opposite topographical change occurs. Taking the direction perpendicular to the coastline as a standard, it is the case for most coasts that the energy of the waves coming in from the right is not equal to the energy of the waves coming in from the left, but rather one of them usually predominates. For example, if the average energy of the waves coming in from the right is larger than that of the waves coming in from the left, then even though the right side of the groin sees repeated accumulation and erosion, eventually the amount of accumulation will grow, while the erosion will increase on the left side of the groin. Therefore, the topographical changes due to longshore sediment transport can be called irreversible, so that when building port or coastal facilities it is desirable to first understand the predominant direction of the longshore sediment transport for that coastal, as well as the longshore sediment transport rate, so as to be able to estimate the degree of coastal deformation in that area.

(10) Coastal Topography

① Terminology for various sections of a beach profile

Typical sections of a sandy beach are defined with the terminology shown in **Fig. 6.3.1**. The "offshore" is the area on the offing where normal waves do not break, and in many cases the bottom slope is comparatively gentle. The "inshore" refers to the area between the landward boundary of the offshore and the ebb tide shoreline, where waves break and longshore bars or steps are formed. The "foreshore" is the zone from the ebb tide shoreline to the location where waves will reach normally, and the "backshore" is the zone from the landward boundary of foreshore to the coastline, where waves will reach during stormy weather with the rise of water level.

The names shown along the top row of **Fig. 6.3.1** classify regions based on the type of sediment movement. In the surf zone the sediment is suspended due to the action of large eddies generated by breaking waves and carried in sand clouds of high density. As for the littoral drift in the swash zone, when the wave is uprush the sand is lifted up and carried by the agitation at the front edge of the running-up waves, but when the wave is downwash the agitation on the sea bottom predominates and the sediment is carried as bedload.

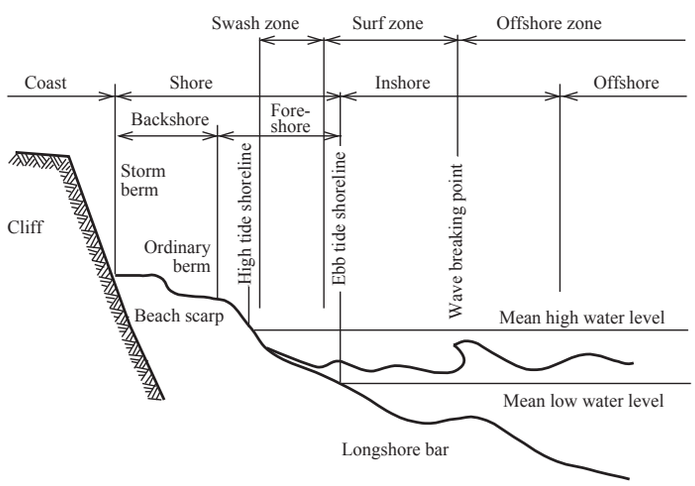


Fig. 6.3.1 Terminology of Beach Profile

② Topography of sandy beaches such as longshore bars

A longshore bar is the most distinctive topographical feature of a sandy beach, around which interrelated sandy beach cross-sections form. Viewing the shape of a coastal sandy beach horizontally, it is either ① long and linear, roughly parallel to the shoreline as in **Fig. 6.3.2 (a)**, or ② a repeating arch as in **Fig. 6.3.2 (b)**. In particular, the latter type of coastal sand bar is called a crescentic bar. Also, a coastal sand bar often forms multiple stages in a sequence leading out to sea, in which case it exists on a large scale as an offshore sand bar.

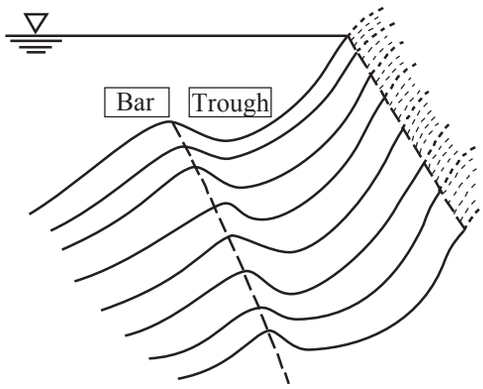


Fig. 6.3.2(a) Longshore Bar

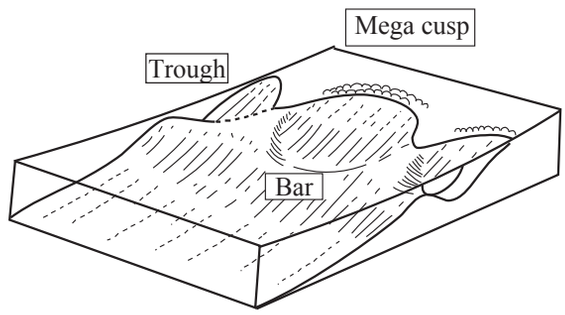


Fig. 6.3.2(b) Crescentic Bar

③ Foreshore topography

As shown in **Fig. 6.3.3**, when there is continued calmness a nearly horizontal area forms in the foreshore, somewhat higher than the high tide level, sometimes slanting on the side toward the land. This topography is called a berm. When conditions are rough the berm is eroded, forming the sand bar called an inner bar near the position of the last breakers. Inner bars dissipate the wave energy when waves break upon them, and therefore are thought to prevent further erosion of the foreshore. The sediments of the inner bars that form during rough conditions gradually return to the foreshore when it is calm, and the foreshore eventually returns to its condition prior to the rough period.

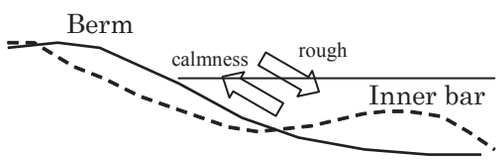


Fig. 6.3.3 Foreshore Topography

(11) Form of Littoral Drift Movement

Littoral drift is classified into three categories of bedload and suspended load, and sheet flow according to the modes of sediment movement.

① Bedload: littoral drift that moves by tumbling, sliding or bouncing along the surface of the sea floor through the direct action of waves and currents.

② Suspended load: littoral drift that is suspended in seawater by turbulence of breakers and others.

③ Sheet flow: littoral drift that moves as a layer of high density flow near the bed surface

Shallow water zones can be classified into three regions as shown in **Fig. 6.3.4**, depending upon the physical properties of waves that provide the external forces for the littoral drift phenomenon. The dominant mode of the littoral drift movement in each region is as follows.

[Offshore zone] In order for sand to be moved by the action of fluid motion, oscillatory movement, the current velocity of the fluid must exceed a certain value. This condition is generally called "the threshold of movement". For littoral drift the threshold of movement is defined with the water depth, threshold depth of sediment movement. When the water depth is shallower than the threshold depth of sediment movement, regular, small undulating topographic contours that are called the sand ripples will form on the sea bottom surface. When sand ripples form, vortices are generated by the fluid motion in the vicinity of the sand ripples and movement of suspended sediment trapped in the vortices occur. As the water depth becomes shallower, sand ripples are extinguished, and a sheet flow condition occur in which sediment moves in stratified layers extending several layers below the sea bed surface.

[Surf zone] Inside the surf zone, high-density suspension of sediment is formed by the severe agitation and action of large-scale vortices that are generated by breakers. The volume of sand that moves near the seabed surface in a bedload state also increases. For convenience the sand movement inside the surf zone is divided into a component called the longshore sediment transport that moves parallel to the shoreline and a component called the cross-shore sediment transport that is perpendicular to the shoreline. While the time frame for beach deformation caused by longshore sediment transport is long, the time frame for cross-shore sediment transport is relatively short, from a few days to about one week, like that for periods of storms passing.

[Swash zone] The sand movement in a swash zone differs for the times of wave runup and downflow. During the time of wave runup sand is put in suspension by the agitation at the front of a wave and transported by running-up water, whereas during the downflow sand is carried in a bedload mode.

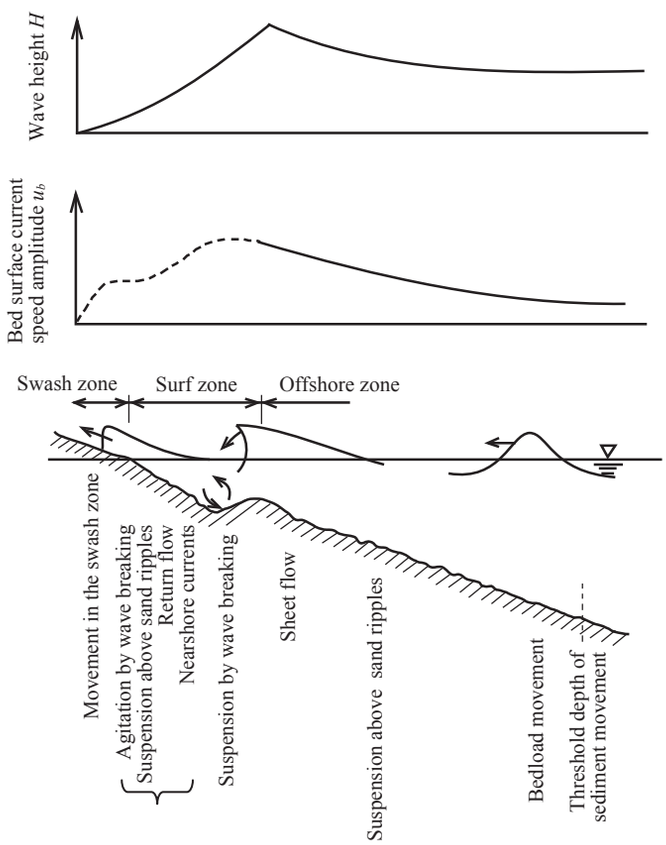


Fig. 6.3.4 Changes in Sediment Movement Modes in Cross-Shore Direction

(12) Physical Meaning of and Estimation Formulas for the Threshold Depth of Sediment Movement

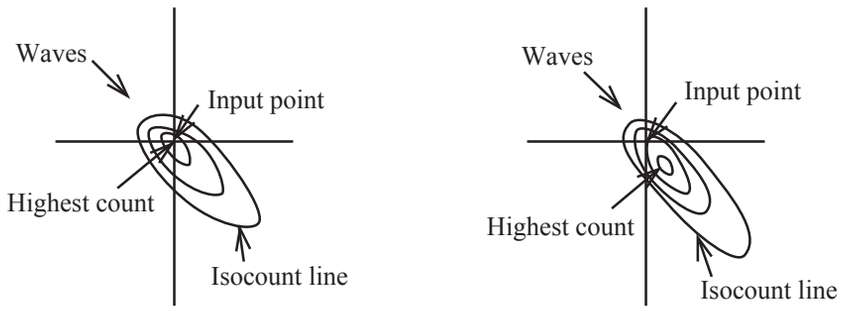
In respect of the threshold depth of sediment movement which is required to determine the extension of breakwater or the water depth at the head and the offshore boundary of beach deformation, Sato and Tanaka [8), 16)] conducted a number of field surveys using radioactive glass sand as a tracer. Based upon their observed results they defined the littoral drift movement conditions as follows.

(a) Surface layer sediment movement:

As shown in **Fig. 6.3.5 (a)**, the elongation of the isocount lines that show the distribution of radioactive glass sand after waves acted upon it on the sea floor demonstrates that all of the sand has moved in the direction of the waves. But the location of the highest count remained at the injection point of glass sand, indicating no movement. This corresponds to a situation in which the surface layer sand is moved collectively by traction, parallel to the wave direction.

(b) Complete sediment movement:

As shown in **Fig. 6.3.5 (b)**, this refers to a situation in which both the isocount lines and the portion of the highest count move in the wave direction. This corresponds to a situation of distinct sand movement with the result of apparent change in water depth. The threshold depth of complete sediment movement is often used as the threshold depth of sediment movement for engineering purposes.



(a) Surface layer sediment movement (b) Total sediment movement

Fig. 6.3.5 Spread of Radioactive Glass Sand in Surface Layer Sediment Movement and Total Sediment Movement

Based upon the field data, two equations are proposed by Sato and Tanaka for estimating the threshold depth of surface layer sediment movement and that of complete sediment movement.

① Threshold depth of surface layer sediment movement

$$\frac{H_0}{L_0} = 1.35\left(\frac{d}{L_0}\right)^{1/3} \sinh\frac{2\pi h_i}{L}\frac{H_0}{H} \tag{6.3.4}$$

② Threshold depth of complete sediment movement

$$\frac{H_0}{L_0} = 2.40\left(\frac{d}{L_0}\right)^{1/3} \sinh\frac{2\pi h_i}{L}\frac{H_0}{H} \tag{6.3.5}$$

where

L_0 : deepwater wavelength (m)
H_0 : equivalent deepwater wave height (m)
L : wavelength at water depth h_i (m)
H : wave height at water depth h_i (m)
d : sediment grain size, average grain size or median diameter (m)
h_i : threshold depth of sediment movement (m)

Repeated calculations are required to estimate the threshold water depths using equations (**6.3.4**) and (**6.3.5**). Calculation diagrams like those in **Fig. 6.3.6 (a)** and **(b)** have been prepared so that the depths can be easily estimated. By specifying d/L_0 and H_0/L_0, it is possible to determine h_i/L_0. Specific calculation examples are shown in Reference 1).

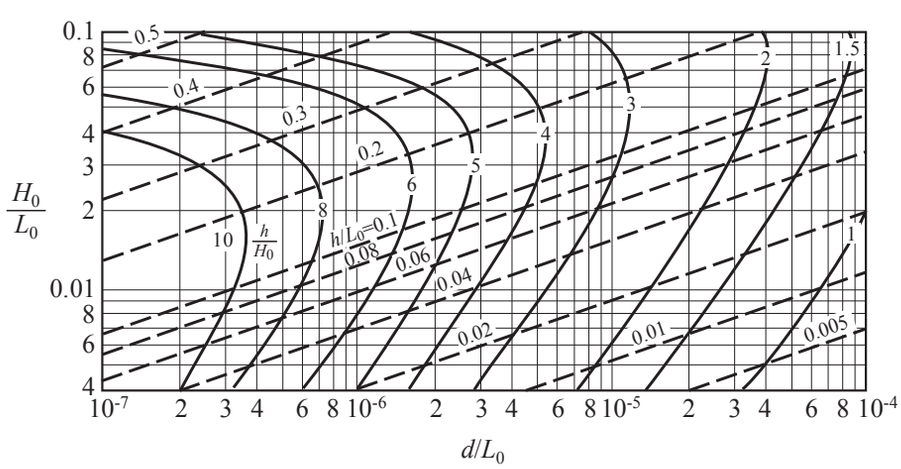


Fig. 6.3.6 (a) Calculation Diagram for Threshold Depth of Surface Layer Sediment Movement [1)]

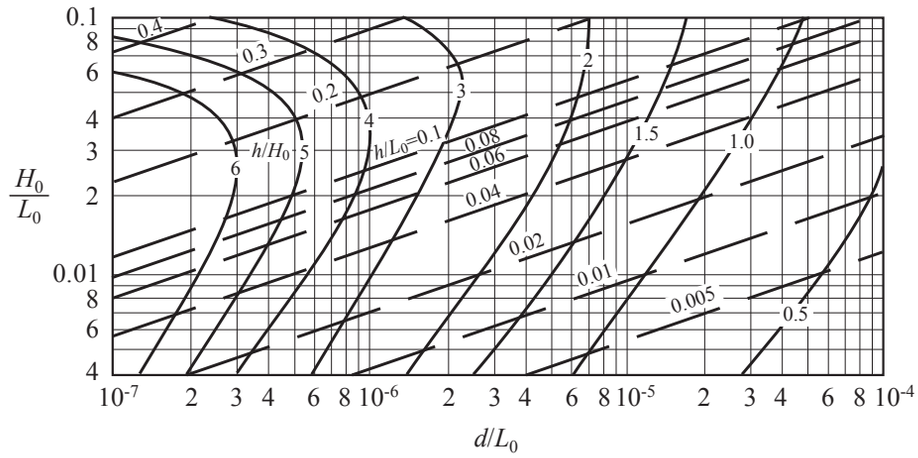


Fig. 6.3.6 (b) Calculation Diagram for Threshold Depth of Total Sediment Movement [1)]

(13) Longshore Sediment Transport

① The predominant direction of longshore sediment transport is determined using the following information:

(a) Topography of the natural coast and that around coastal structures (see **Fig. 6.3.7**)

(b) Alongshore distribution of the sediment characteristics such as median diameter, mineral composition, etc.

(c) Direction of movement of fluorescent sand tracers

(d) Direction of incident wave energy flux

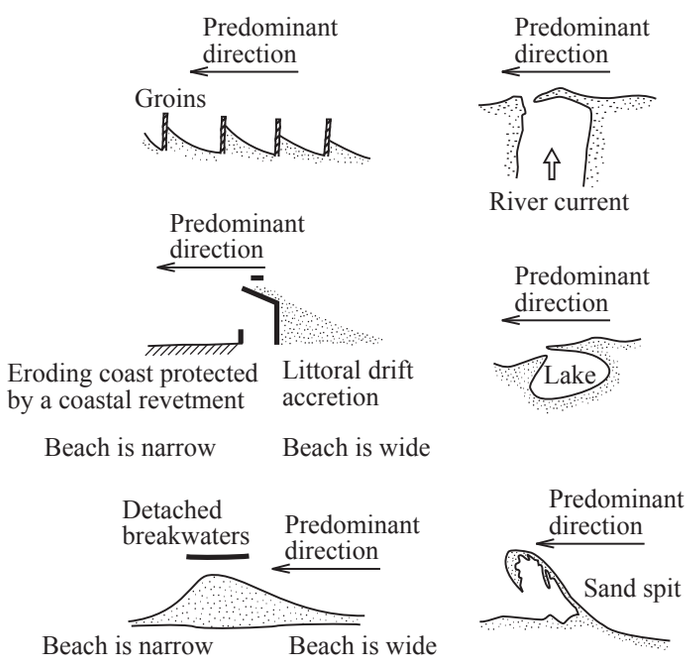


Fig. 6.3.7 Typical Coastal Topography Showing the Predominant Direction of Littoral Drift

② To estimate the longshore sediment transport rate, the following various data must be prepared and sufficiently investigated:

(a) Continuous observation data of the change in sediment volume around a coastal structure

(b) Data on the alongshore component of wave energy flux

(c) Data concerning the sediment transport rate at the surrounding coast

(d) Data on past dredging volume

(e) Continuous observation data on deposition volume at the experimental dredging site

(f) Data on the volume of movement of fluorescent sand tracers placed within the surf zone

③ Various formulas can be used to estimate an approximate value of longshore sediment transport rate.[1), 17), 18), 19)] The formulas are normally given in the expression shown in equation **(6.3.6)**, with the coefficient *a* for various formulas being as given in **Table 6.3.1**.

$$Q_x = aE_x$$
$$E_x = \sum K_r^2 \left(\frac{n_A w_0 H_A^2 L_A}{8T} \right) \sin\alpha_b \cos\alpha_b \tag{6.3.6}$$

where

Q_x : longshore sediment transport rate (m^3/s)
E_x : alongshore component of wave energy flux (kN · m/m/s)
K_r : refraction coefficient between the wave observation point and the wave breaking point
n_A : ratio of group velocity to wave celerity at the wave observation point
w_0 : unit weight of sea water (kN/m^3)
H_A : wave height at the wave observation point (m)
L_A : wavelength at the wave observation point (m)
T : wave period (s)
α_b : angle of wave incidence at the wave breaking point (°)

Table 6.3.1 Coefficient *a* for Longshore Sediment Transport Rate Formula

Savage [18)]	Sato and Tanaka [17)]	U.S. Army Corps of Engineers [19)]
0.022	0.03	0.04

(14) Littoral Drift Phenomena in the Surf Zone

Inside the surf zone, large quantities of sand move by turbulence caused by breakers, by the increase of the wave orbital velocity near the bottom due to shallower water depth, and by the existence of nearshore currents.

Komar[20)], based on longshore sediment transport rates obtained from studies of fluorescent sand, reported that bed load dominates in the surf zone. Sternberg et.al.[21)] reported that most of the longshore sediment transport rate can be explained by suspended load. As a counterpoint to these two conflicting results, Kato et.al.[22)] used fluorescent sand to measure local sediment transport rates within the surf zone and found that bed load dominates when the velocitiy of water particle due to waves is small, while suspended load dominates when the velocity is large.

The sediment movement when suspended sediment is predominant can be examined by dividing the movement into two processes.

① Sediment suspension process caused by organized vortices formed by wave breaking.

② Settling process during which sediment is buffeted by random external forces following breakup of organized vortices.

Fig. 6.3.8 gives the temporal variations of suspended sediment concentration and horizontal current velocity that were measured by Katoh et al.[23)] inside the surf zone in the field. The white arrows in the figure point out the waves that broke on the seaward side of the observation point and the black arrows point out the waves that passed the observation point and broke on the shoreward side. It is clear that the suspended sediment concentration increased rapidly when waves broke on the seaward side. This result indicates that sediment suspension is related to the organized vortices in particular obliquely descending vortices [24)] that occur after waves break.

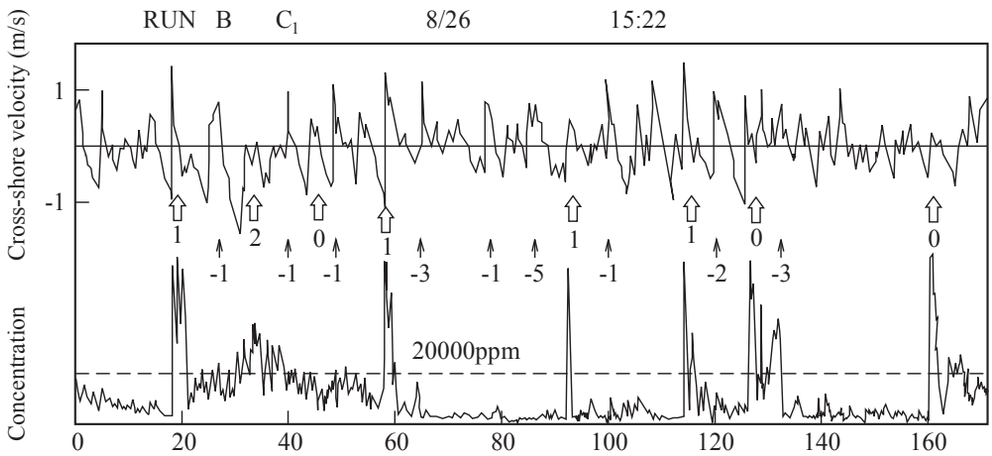


Fig. 6.3.8 Example of Field Observation of Concentration of Suspended Sediment [23)]

(15) Topographical or Shoreline Deformation in the Swash Zone

Horikawa et al [27)] investigated the criterion on for shoreline advance and retreat occurring as a result of sand movement in the swash zone based on laboratory experiments, and proposed equation **(6.3.7)** which is also applicable for the field condition.

$$\frac{H_0}{L_0} = C_s\left(\tan\beta\right)^{-0.27}\left(\frac{d}{L_0}\right)^{0.67} \tag{6.3.7}$$

where

H_0 : deepwater wave height (m)
L_0 : deepwater wavelength (m)
$\tan\beta$: average bottom slope from the shoreline to a water depth of 20 m
d : sediment grain size (m)
C_s : coefficient

Based on equation (**6.3.7**), a shoreline will retreat when $Cs \geq 18$ (see **Fig. 6.3.9**).

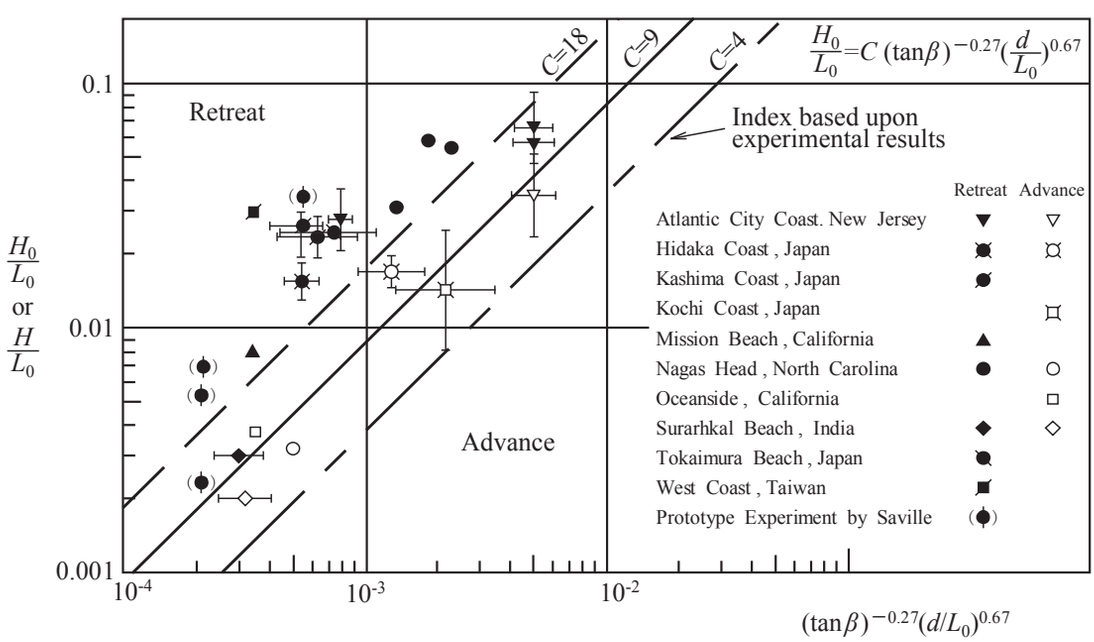


Fig. 6.3.9 Advance and Retreat of Shorelines in the Field [27)]

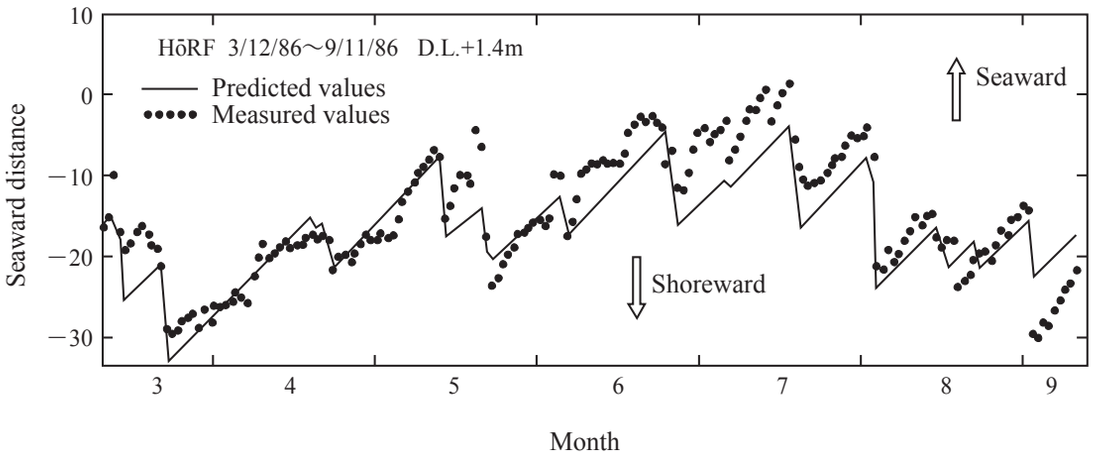


Fig. 6.3.10 Comparison of Prediction and Measurement of Shoreline Location [28)]

Katoh et al.[28)] revised equation (**6.3.7**) using deepwater wave energy flux and presented a model to predict the daily shoreline change. **Fig. 6.3.10** is a comparison of the predicted and measured results of shoreline location.

(16) Relationship between Foreshore Topographical Changes and Groundwater Level

The topographical changes that accompany the changes in the foreshore tide level can be explained as follows by using **Fig. 6.3.11**.[29)] When the tide level changes, the beach groundwater level also changes as a response. But because of the delay in response time, the groundwater level at the flood tide differs from that at the ebb tide even though the tide level is the same.

(a) During the flood tide the groundwater level is low, and it is easy for the seawater running up on the beach to permeate underground. Thus the sediment carried by the seawater when it runs up on the beach will accrete there.

(b) On the other hand, during the ebb tide the groundwater level is high and it is difficult for seawater to run up on the beach and to permeate underground. At certain conditions, the groundwater may flow out of the beach surface during the ebb tide. As shown in **Fig. 6.3.11**, the result is that the sediment that accreted during the flood tide will be eroded, and return to its original location.

When waves run up to a high level on a beach during storm periods, a high groundwater level condition continues throughout the stormy weather period because the run-up seawater permeates into the beach, and the condition becomes as shown in **Fig. 6.3.11 (b)**. Occurrence of rapid foreshore erosion during such the condition has been confirmed by field data.

Some shore protection methods make use of this relationship between the foreshore groundwater level and sand movement; i.e., lowering the groundwater level by forced means or gravity and thus preventing erosion. The method making use of gravity, a highly water-permeable layer is installed in the foreshore sand to cause the groundwater flow down offshore and to lower the groundwater level. With this method it is possible to preserve beach conditions very close to those of a natural beach because no structures are visible above the beach floor.

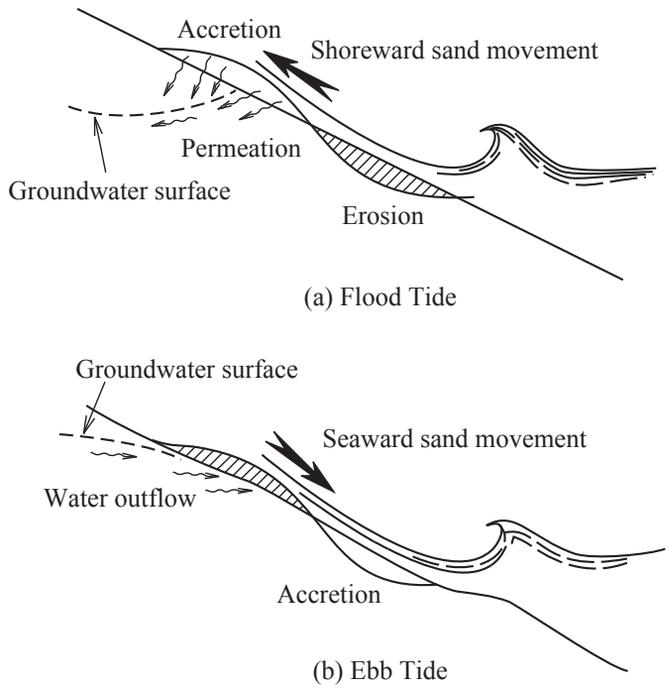


Fig. 6.3.11 Relationship between Foreshore Topographical Changes and Groundwater Level

(17) Movement of Longshore Bars
As already mentioned in **(10)** ③ **Foreshore Topography**, longshore bars sometimes form in the surf zone. Longshore bars form periodically and move offshore.[32] While longshore bars move offshore, cross-shore sediment transport may occur offshore or onshore in various places, so that offshore sediment transport occurs near the bar crests, while onshore sediment transport occurs in trough areas.[32] The period of cyclic offshore bar movement depends on the sea coast, and can range from one year to 20 years.

6.3.2 Scouring around Structures

(1) Scouring shall be taken into consideration when there are concerns that scouring around structures such as breakwaters, groins, and training jetties may affect the safety and integrity of structures.

(2) Wave characteristics that act on natural beaches can be considered as nearly constant over a long period of time. Topographies that form in response to these characteristics are nearly stable as well. Scouring will occur when structures are constructed and the equilibrium between external forces and topography will be disturbed locally or over a broad area. The mechanism and amount of scouring will change according to the location of a suructure because the wave action on the structure changes, and hence must be considered carefully when choosing scouring prevention works.

(3) Scouring in Front of Coastal Revetment
It is well known that scouring in front of coastal revetment has a close relationship with wave reflection coefficient. For example, **Fig. 6.3.12** has been proposed for determining scouring or accretion by means of the reflection coefficient K and the parameter $(H_0/L_0)(\ell/d_{50})\sin\alpha$ which is defined with the wave steepness H_0 / L_0, mean diameter of sediment d_{50}, slope gradient of coastal revetment α, for a vertical breakwater, α= 90º, and the distance l from the wave runup point on an equilibrium profile to the location of the coastal revetment. The diagram indicates that all other conditions being equal, it is advantageous against scouring in front of revetment to make the front surface of revetment inclined.

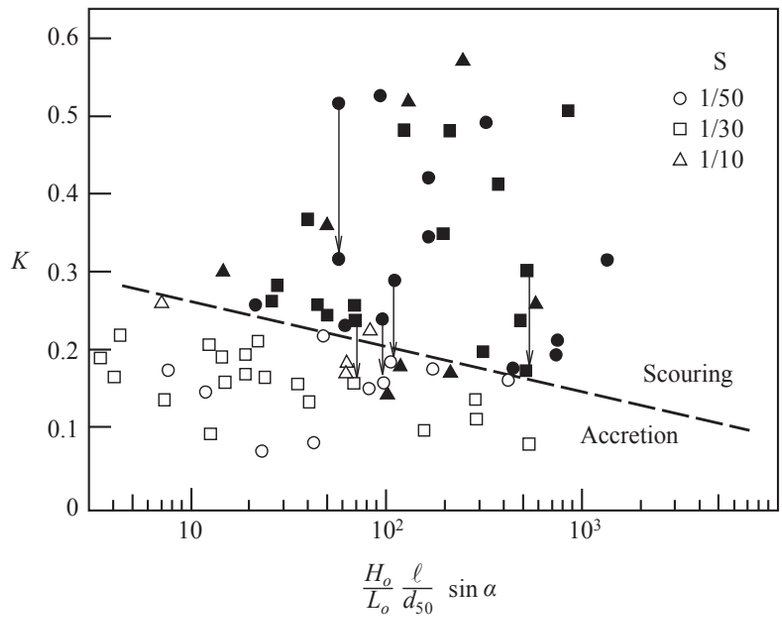


Fig. 6.3.12 Threshold Conditions between Scouring and Accretion in Front of Coastal Revetment [36)]

(4) Local Scouring around Breakwaters

① Scouring in the surf zone

(a) Local scouring at the breakwater head

Fig. 6.3.13 shows the local scouring conditions around a breakwater head, as analyzed by Tanaka.[37)] The maximum scouring depth is found to be nearly equal to the maximum significant wave height $(H_{1/3})_{max}$ during the period up to 15 days prior to the time of scouring measurements. In addition, **Fig. 6.3.14** shows the relationship between the water depth around a breakwater head and the scouring depth. The scouring depth becomes the largest when the water depth at breakwater head is about 3 m to 5 m (namely in the surf zone).

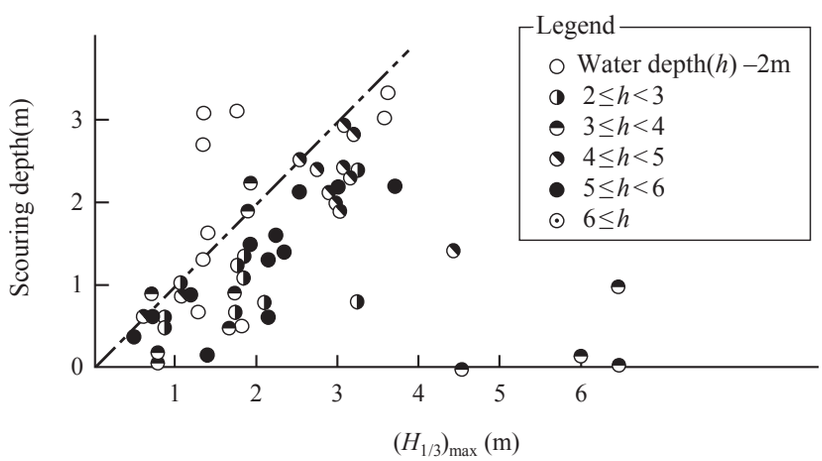


Fig. 6.3.13 Relationship between Scouring Depth at Breakwater Head and Maximum Significant Wave Height during the Prior 15 Days $(H_{1/3})max$

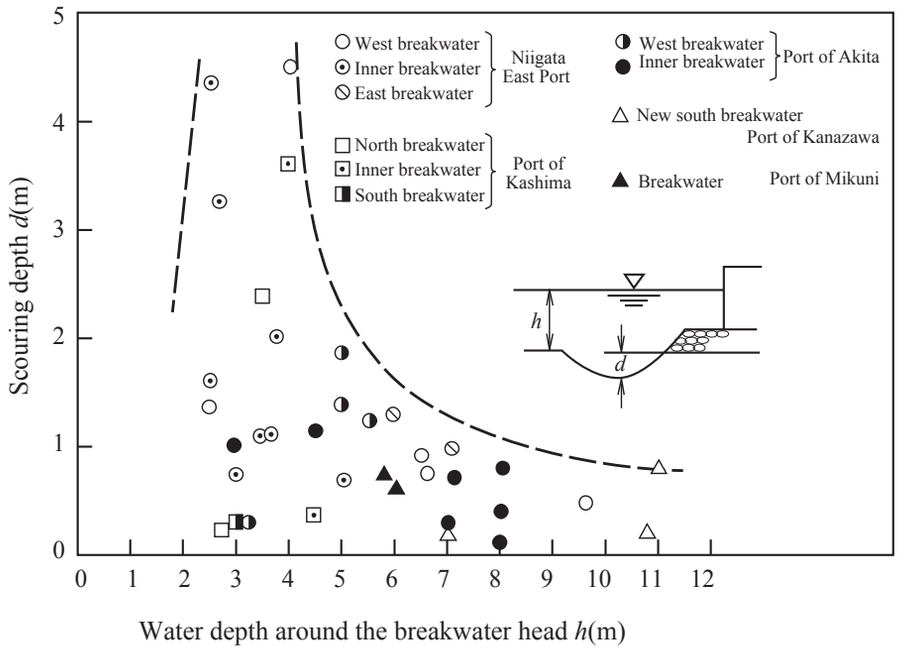


Fig. 6.3.14 Relationship between Scouring Depth and Water Depth around Breakwater Head

(b) Scouring at front of breakwaters

Fig. 6.2.15 shows the relationship between the scouring depth in front of a breakwater and water depth.[37] The black circles in the figure show the condition of scouring around the oblique part of the breakwater. The scouring depth shows its maximum at the bend of the breakwater, where the water depth is about 7m, and gradually decreases seaward. On the other hand, the scouring depth at the front of the straight part of the breakwater, which is shown by the white circles, has its maximum value at around a water depth of 2m and decreases in both shallower and deeper water. The location of the maximum scouring depth corresponds to the location of a longshore bar.

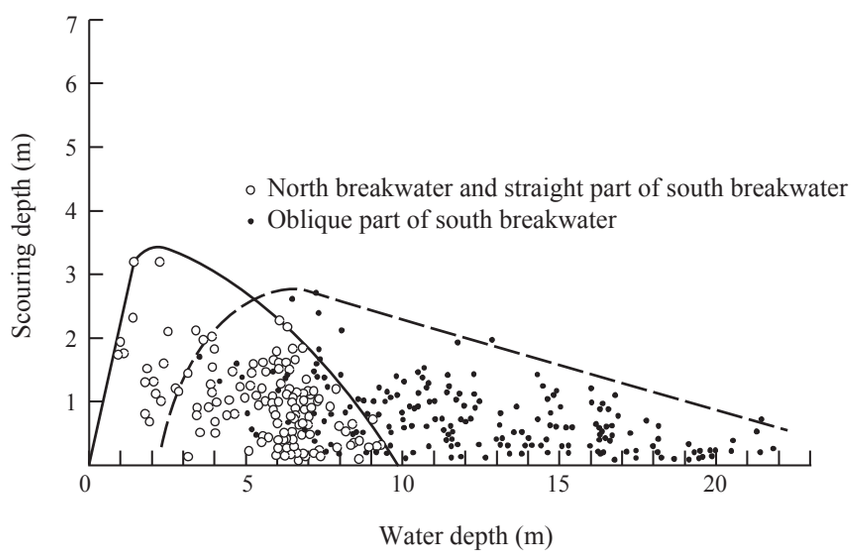


Fig. 6.3.15 Relationship between Scouring Depth and Water Depth [37]

(c) Local Scouring Outside Breakwaters

Fig. 6.3.16 shows examples of places where pronounced local scouring occurs as the result of breakwater extension:

(i) Breakwater head (especially pronounced when the breakwater head is in the surf zone).

(ii) Around the straight portion of the breakwater (especially pronounced near the point where the breakwater

crosses the longshore bar).

(iii) Around a front mound or a submerged breakwater (especially pronounced outside the harbor).

(iv) Places where the breakwater bends.

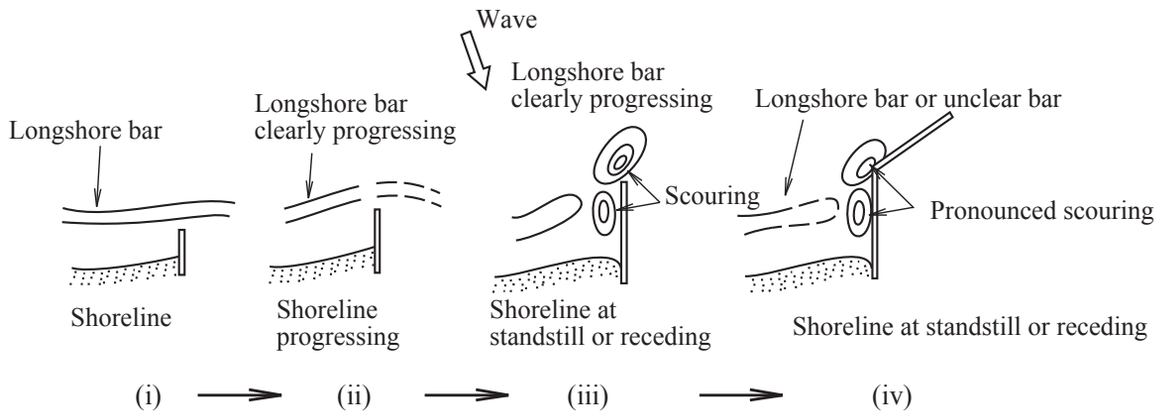


Fig. 6.3.16 Local Scouring Outside the Breakwater [38)]

② Scouring in standing wave domain

Scouring depth in front of a vertical wall tends to decrease as the initial water depth in front of the wall increases and the wave condition is shifted into the standing wave domain. In case of composite type breakwaters, where the toe of rubble mound is somewhat away from the wave reflection surface of the upright section, scouring at the toe of rubble mound by standing waves sometimes become a problem. Irie et al.[39)] carried out experiments concerning this type of scouring and highlighted the following issues:

(a) The basic parameter is U_b/ω, the ratio of the maximum horizontal velocity of water particles at the bottom by incident waves U_b to the settling velocity of sediment ω. When $U_b/\omega > 10$, sediment will move from the location of the node of standing waves to the location of the antinode, with scouring occurring at the node and accretion taking place at the antinode. It is called L-type scouring. When $U_b/\omega < 10$, the opposite phenomenon will occur. It is called N-type scouring (refer to **Fig. 6.3.17**). The L-type scouring refers to the phenomenon where accretion occurs at the antinode of standing waves and scouring occurs at the node, whereas the N-type scouring refers to the opposite phenomenon where scouring occurs at the antinode and accretion occurs at the node.

(b) The value of U_b/ω tends to be larger than 10 in the field, and generally scouring at the node of standing waves is predominant. Normally, because a toe of rubble mound is located at the distance of about 1/4 wavelength or so from the upright wall, scouring and subsidence of the rubble mound of breakwater will occur at its toe as the sediment there moves toward the location of antinode at one half wavelength from the upright wall.

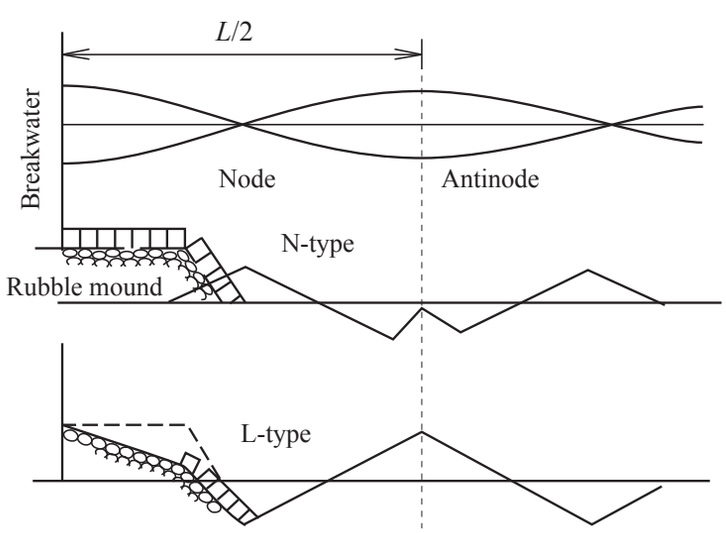


Fig. 6.3.17 Sketch of Scouring by Standing Waves [39)]

6.4 Prediction of Beach Deformation

(1) All the related factors shall be thoroughly investigated when predicting beach deformation, with consideration given to factors such as the predicted results by an appropriate prediction method and the data of past beach deformation at the site in question.

(2) Various methods exist as procedures for predicting beach deformation, including empirical prediction techniques, estimation based on hydraulic model experiments especially with movable bed model experiments, and numerical simulations. Because beach deformation is strongly governed by the characteristics of the region in question, however, it is inappropriate to rely on any single method. Efforts are desirably made to predict beach deformation by combining two or more procedures and by investigating the local data and information as comprehensively as possible.

(3) Empirical Prediction Techniques
The empirical method is a procedure that, on the basis of collection and analysis of the past examples of beach deformation, the layout and structural characteristics of structures to be built are compared with the past examples of similar nature. Based upon the similarities, the potential for beach deformation to be caused by the construction of structures is judged. Tanaka [37] has carried out research on modeling of the complicated topographical changes that occur after the construction of structures. He classified characteristics of typical topographical changes in numerous examples of beach deformation. As a result of this research, it is possible to understand the topographical changes in the vicinity of Japanese ports in several representative patterns (see **Fig. 6.4.1**). Exceptions to these patterns are relatively rare. By judging which pattern in **Fig. 6.4.1** is applicable to the coast under investigation, a qualitative prediction of beach deformation becomes possible.

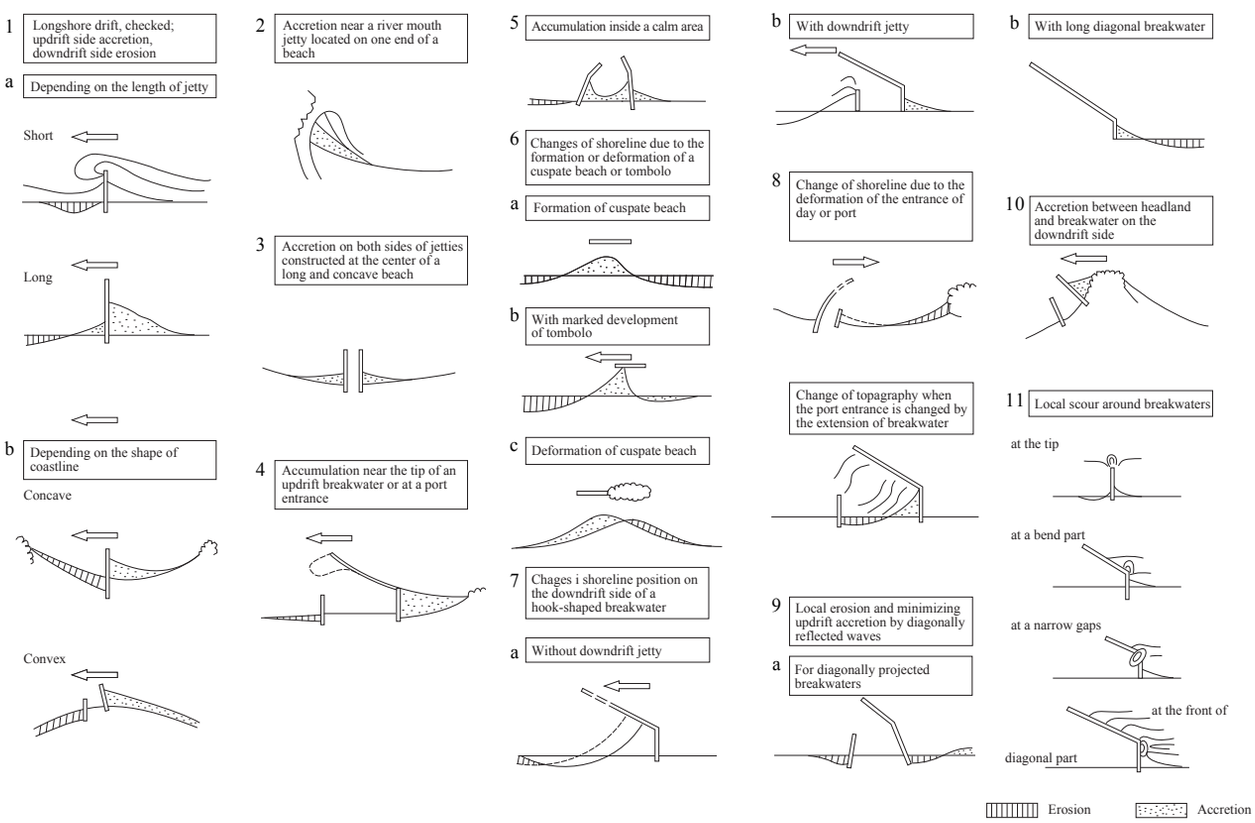


Fig. 6.4.1 Classification of Patterns of Topographical Changes after Construction of Structures

(4) Hydraulic Model Experiments particularly Movable Bed Model Experiments

The capability of predicting beach deformation based on hydraulic model experiments, particularly movable bed model experiments, is limited because the problem of similarity remains unresolved. But the advantage of model experiments is such that specific topographical changes can be reproduced in a laboratory basin and the phenomenon to be forecasted can be understood visually.

Because of the similarity problems being unresolved, experiments are carried out with partially distorting model scales, and by focusing attention on the reproducibility of the area of most concern, based on the comparison of several formulas on beach deformation similarity, and a topographic model that is judged most reliable is introduced in a laboratory basin. Before predicting future beach deformation, it will be necessary to verify the model for reproducibility of the topographical changes that occurred in the past in the study area and to confirm the model's kinematic similarity. The degree of kinematic similarity will be judged by the accuracy of reproduction. The reproductive accuracy of the experiment, therefore, cannot exceed the accuracy of the data collected on beach deformations in the past.

One can assume that sufficiently effective engineering predictions are possible if sufficient care is taken in preliminary experiments to study the reproducibility of actual beach deformations, and in particular the following problems can be addressed:

① The area of topographical changes caused by the construction of coastal facilities.

② Comparisons of alternative plans for measures to prevent coastal erosion, such as groins or detached breakwaters.

③ Qualitative evaluation of shoreline changes due to large-scale sea facilities.

However, predictions of beach deformation are difficult in the following types of cases:

(a) Stable cross-sectional shapes of large-scale artificial beaches that face rough seas.

(b) Deformation problems caused by large-scale sea facilities on beaches that face rough seas.

(c) Siltation rates in navigation channels and harbors.

(d) Countermeasures against siltaiton in small-scale ports such as marinas.

(e) Effects of permeable detached breakwaters and submerged breakwaters on beach stability.

For details concerning movable bed model experiments see Reference **40)**.

(5) Predictions by Numerical Simulations

At the present time, numerical simulations are divided into two models: those that predict changes in the shoreline location called shoreline change model, which is also called one-line theory from the fact that it predicts changes along a single shoreline, and those that predict three-dimensional changes in water depth; i.e., beach topographical changes called three-dimensional model or coast topographical change prediction model.

(6) Shoreline Change Model (1-line Theory)

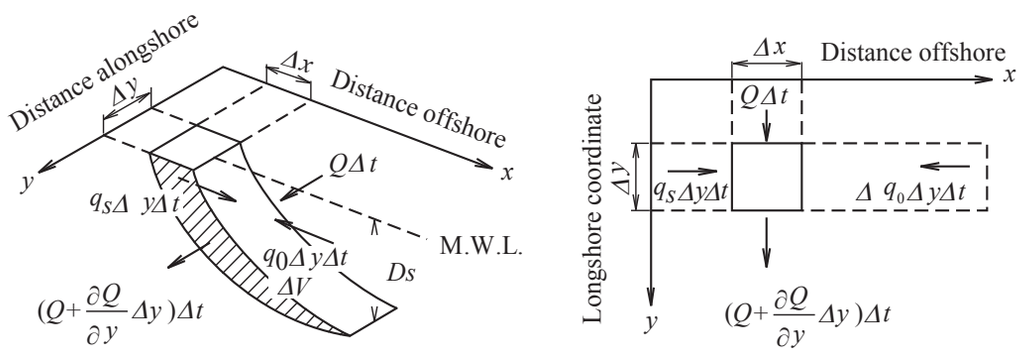
Beach sediment is transported by waves and currents both in the offshore and onshore directions and in the alongshore direction. Because littoral drift is caused mainly from the direct action of waves, littoral drift during storm periods will be predominantly towards offshore, and the coast will be eroded with a retreat of shoreline. When the sea becomes calm, however, the sediment will be carried towards the shore and the shoreline will advance. Along with these movements the beach profile will also change. This topographical change in the shoreline location and beach profiles caused by the onshore-offshore transport is normally a seasonal one. When looked at on the average profile over a long period of time, the changes caused by onshore-offshore transport can mostly be ignored when compared with those caused by longshore transport. Thus, when focusing on beach erosion or accretion over a period of several years, one can assume there is no change in the shape of beach profile and that beach erosion and accretion will correspond to the retreat and advance of the shoreline. A prediction of changes in the shoreline location can then be based on the balance of the deposition and removal of sediment volume primarily from longshore transport.

Fig. 6.4.2 sketches the calculation principles of a shoreline change prediction model. As shown in the Fig., the shoreline should be split along the alongshore direction of the shoreline into sections having the width Δy, and the inflow and outflow of sediment volume between those widths are considered. That is, when the inflow of sediment volume $Q\Delta t$ and the outflow of sediment volume during time period Δt are compared, accretion will occur if the former is larger and erosion will take place if the latter is larger. By introducing the assumption that the beach profile remains unchanged over time and any imbalance in the sediment inflow and outflow simply shifts the beach profile parallel to offshore or onshore, it is possible to express the advance and retreat of the shoreline as the result of the imbalance. When this is expressed in the continuity of sediment flux, the result is equation (**6.4.1**).

$$\frac{\partial x_s}{\partial t} + \frac{1}{D_s}\left(\frac{\partial Q}{\partial y} - q\right) = 0 \tag{6.4.1}$$

where

x_s : shoreline location (m)
t : time (s)
y : coordinate in alongshore direction (m)
D_s : width of the littoral drift movement zone (m)
Q : longshore sediment transport rate (m^3/s)
q : cross-shore inflow ($q > 0$) or outflow ($q < 0$) of the sediment transport rate across the onshore-offshore boundary per unit width in the alongshore direction ($m^3/m/s$)



(a) Model view (b) Plan view

Fig. 6.4.2 Relationship between Shoreline Change and Sand Movement

The longshore sediment transport rate Q, is often estimated using an equation including the alongshore component of the incident wave energy flux at the wave breaking point, which is obtained from the wave height and direction. One of the equations frequently used to estimate Q is equation **(6.4.2)**, which is based on Ozawa and Brampton [45]. This equation incorporates the influence of the current induced by the alongshore gradient of breaking wave height, which is often observed behind coastal structures.

$$Q = \frac{H_B{}^2 C_{gB}}{16s(1-\lambda)}\left(K_1 \sin 2\theta_B - \frac{2K_2}{\tan\beta}\cos\theta_B \frac{\partial H_B}{\partial y}\right) \tag{6.4.2}$$

where

H_B : breaking wave height (m)
C_{gB} : group velocity at the wave breaking point (m/s)
θ_B : angle formed by the wave crest line and the shoreline at the breaker point (°)
$\tan\beta$: equilibrium beach slope
s : $s = (\rho_s - \rho_0)/\rho_0$
ρ_s : density of sediment (g/cm^3)
ρ_0 : density of seawater (g/cm^3)
λ : void ratio of sediment
K_1, K_2 : coefficients

The width of sediment movement zone Ds is the distance perpendicular to the shoreline from the wave runup point on the beach to the offshore boundary where longshore sediment transport activity becomes significant. The distance D_s is determined basically by investigating the volume of beach profile area change from the bathymetric data of the coast in question. When the available data are inadequate, an energy-averaged representative wave is estimated and its dimensions are substituted into the equations for the runup height and the threshold depth of sediment movement as a method to conveniently find the distance D_s. Because equation **(6.4.2)** cannot be solved analytically except in extremely simple cases, a computer is required to perform the numerical computation. In the numerical computation Q must be evaluated at each measuring line. For this purpose the wave height and angle, and water depth at the wave breaking point at each measuring line must be calculated using a separate wave deformation calculation.

(7) Three-Dimensional Deformation Models or Prediction Model for Bathmetric Change

Prediction model for bathmetric change, which predict the change in the water depth at each point in a calculation domain, consider not only longshore sediment transport but also cross-shore sediment transport. Several examples have been presented in prediction model for bathmetric change, but in every model the method of calculation is to first calculate the fields of waves and nearshore currents and then determine the bathmetric changes. The determination of nearshore currents may mean the depth average for nearshore current fields, or may also include their vertical distributions.

Prediction model for bathmetric change are divided into two main types depending on the method of predicting the bathmetric changes. One type of model is based on local sediment transport rates calculated from hydraulic factors and sediment particle diameters at the location in question, and the other type of model considers convection and diffusion of sediment.

The models based on local sediment transport rates determine bathmetric changes based on the difference between the incoming amount and the outgoing amount of the local sediment transport, and one of these models is that of Watanabe et.al.[46], which uses the sediment transport rate formulas of Watanabe et.al. Other local sediment transport rate formulas are those of Bijker, [47] and Bailard [48] which separately calculate the bedload and the suspended-load. The model of Watanabe et.al.[46] has been improved several times and has developed into a model that considers the grain size distribution [49].

Models that consider the convection and diffusion of sediment, either in three dimensions or just for a two-dimensional, have been proposed, for example, by Sawaragi et.al.[50] (hereafter, the Sawaragi model) and by Lesser et.al.[51] (hereafter, the Delft 3D-flow model). In these models, the bathmetric changes are dominated by the difference between the amount of uplifting of suspended sand and its settling rate, and by the difference between the incoming amount of bedload and its outgoing amount. The concentration of suspended-load at reference points near the sea bottom, which is used in the calculation of uplifted amounts of suspended sand, may be found from the formula of Deguchi and Sawaragi [52] used in the Sawaragi model, or from the van Rijn formula [53] used in the Delft 3D-flow model. An example of a formula for bedload is the bedload formula of van Rijn [54] (used in the Delft 3D-flow model).

The main areas of application of prediction model for bathmetric change have been cases where offshore topographical changes are important, such as the problem of siltation in channels and basins, and bathmetric changes due to large-scale submerged breakwaters.

6.5 Fluid Force due to Current [55]

(1) General

The fluid force due to the currents acting on members and facilities in the water or near the water surface such as a pile-supported structure including a pier, a pipeline, or the armor material of a mound is proportional to the square of the flow velocity. It may be divided into the drag force acting in the direction of the current and the lift force acting in the direction perpendicular to the current. Generally drag and lift forces are calculated using the following equations. Note also that a thin, tube-like object in the water may be subject to vibrations excited by induced vortices.

① Drag force

$$F_D = \frac{1}{2} C_D \rho_0 A U^2 \tag{6.5.1}$$

where

F_D : drag force acting on the object in the direction of the current (kN)
C_D : drag coefficient
ρ_0 : density of water (t/m^3)
A : projected area of the object in the direction of the current (m^2)
U : flow velocity (m/s)

② Lift force

$$F_L = \frac{1}{2} C_L \rho_0 A_L U^2 \tag{6.5.2}$$

where

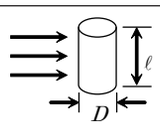
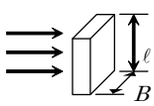
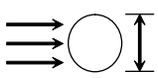
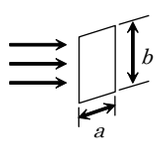
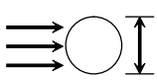
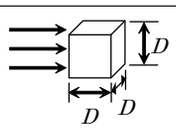
F_L : lift force acting on the object in the direction perpendicular to the current (kN)
C_L : lift coefficient
A_L : projected area of the object in the direction perpendicular to the current (m^2)

(2) Drag Coefficient

The drag due to currents is expressed as the sum of the surface resistance due to friction, and is expressed as in equation (**6.5.1**). The drag coefficient varies according to the shape and the roughness of the object, the direction, and the Reynolds number of the current, and thus the value appropriate to the conditions in question must be used.

When the Reynolds number is greater than about 10^3, the values listed in **Table 6.5.1** may be used as standard values for the drag coefficient. Note that for a circular cylinder or sphere with a smooth surface, the value of the drag coefficient drops suddenly when the Reynolds number is around 10^5. However, for a circular cylinder with a rough surface, this drop in drag coefficient is not particularly large, and the drag coefficient settles down to a constant value that corresponds to the relative roughness. The data for the cube have been obtained from wave force experiments carried out by Hamada, Mitsuyasu and Hase.[56] The values for rectangular cylinders and L-shaped member placed diagonally to the current can be found in the reference 57).

Table 6.5.1 Drag Coefficients

Object shape		Standard area	Drag coefficient
Circular cylinder (rough surface)		$D\ell$	$1.0(\ell > D)$
Rectangular cylinder		$B\ell$	$2.0(\ell > B)$
Circular disk		$\frac{\pi}{4}D^2$	1.2
Rectangular plate		ab	*When* a/b=1 1.12 〃 〃 2 1.15 〃 〃 4 1.19 〃 〃 10 1.29 〃 〃 18 1.40 〃 〃 ∞ 2.01
Sphere		$\frac{\pi}{4}D^2$	0.5–0.2
Cube		D^2	1.3–1.6

(3) Lift Coefficient

As with the drag coefficient, the lift coefficient varies with the shape of the object, the direction of the current, and the Reynolds number. (see **4.7.3 Wave Force Acting on Submerged Members and Isolated Structures**).

(4) Current Force Acting on Coping of Submerged Dike at the Opening of tsunami Protection Breakwater

As for the current force acting on the coping of the submerged dike at the opening of tsunami protection breakwater, Iwasaki et al.[58] have measured the pressure and obtained the values of 0.94 for the drag coefficient and 0.48 for the lift force coefficient. Tanimoto et al.[59] have carried out similar measurements, and obtained the values 1.0 to 1.5 for the drag coefficient and 0.5 to 0.8 for the lift coefficient. They have also pointed out that when the flow velocity in the breakwater opening is large, the effect of the water surface gradient causes the coefficient values to increase.

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7 Other Meteorology Items to be Considered

7.1 Items to be Considered

The following meteorology items should be considered with regard to design and performance verification of port facilities:

① Rain is a factor in determining the capacity of drainage facilities within the port, and can interfere with cargo handling and other port operations.

② Fog interferes with ship navigation and the entering and leaving of the harbor, and is a factor that can decrease the usability of port facilities.

③ Snowfall may need to be considered with regard to its surcharge on port facilities.

④ Atmospheric temperature may affect the stress distribution on port facilities, creating temperature stress.

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8 Meteorological and Marine Observations and Investigations

8.1 Meteorological Observations and Investigations

(1) Overview

Port facilities must be designed to have the required performance with regard to natural phenomena such as strong winds. Therefore, in the performance verification of port facilities, it is necessary to examine items relevant to that purpose by observing of meteorological elements or conducting numerical simulations.

(2) Planning of Meteorological Investigation and Setting of Natural Conditions

Meteorological investigation includes various methods such as statistical analysis of past data, analysis through numerical simulations, and on-site meteorological observations, and it is necessary to formulate a plan by generally considering the following items 1 through 6 in order to decide which methods are desirable:

① Determination of required meteorological elements

② Necessity for real-time on-site meteorological data

③ Possibility to obtain meteorological observation data from the past

④ Possibility to use observational data from the closest meteorological stations or the AMeDAS observation stations

⑤ Necessity for numerical simulations

⑥ Necessity for on-site meteorological observations

Based on these investigation results, determine which of the following methods to use in order to specify the natural conditions:

① Statistical analysis of past data

② Analysis by numerical simulations

③ On-site meteorological observations

8.2 Tide Level Observations and Investigation

(1) Purpose of Tide Level Observations

Tide level observations are continuous observations of the ocean surface level 1, eliminating relatively short frequency variations such as waves. Tide level observations have various purposes, as listed below, so it is preferable for the observations to be done as appropriate for the purpose.

① Standard Water Level

Through all the stages of planning, design and construction, an appropriate standard water level must be provided from tide level observations.

② Mean Water Level Monitoring

Recently, ocean surface rise has become a great concern related to global warming. However, there are great variations in the predictions of the amount of ocean surface rise, so the importance of mean water level monitoring based on long-term tide level observation has been recognized.

③ Understanding of Tsunamis, Storm Surge and Long-period Waves

When structures along the shore suffer a disaster, the understanding of marine conditions, including tide level records, is the first step in the process of understanding the cause and planning recovery measures.

④ Construction Management

Tide level observations are important for performing safe and reliable maritime construction.

⑤ Environmental Monitoring

It is also important to understand tide level variations for environmental monitoring.

(2) Analysis of Long-Term Mean Water Level Variations

Fig. 8.2.1 is an example of a graph of monthly mean tide level variations based on long-term tide level observation. It is known that the mean tide level varies with the season, being higher in the summer and lower in the winter. In addition to this seasonal variation, the long-term mean tide level increases slowly. The amount of this long-term mean tide level rise is about 4.4 mm/year.

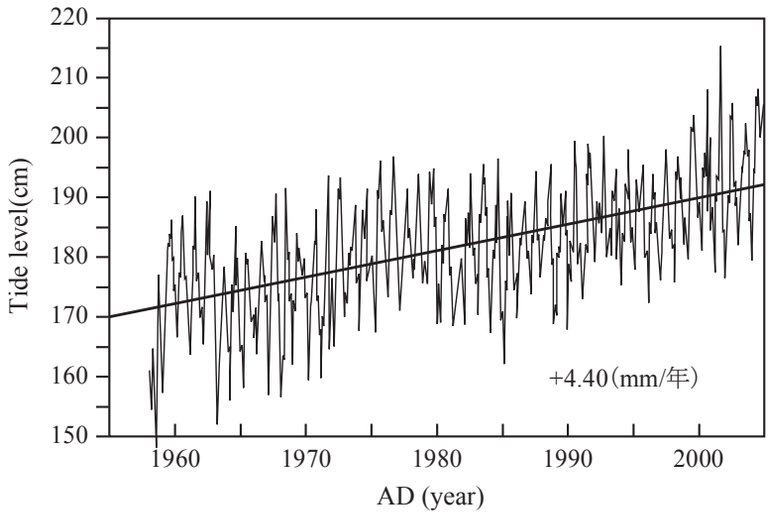

Fig. 8.2.1 Example of Long Term Fluctuation of Monthly Mean Sea Level (Kurihama Tidal Observation Station)

8.3 Wave Observations and Investigation

(1) Overview

In shallow water, waves are deformed by processes such as refraction, breaking and shoaling, so offshore observations are necessary to understand their actual conditions. Mankind will have more interactions with the seas in the future than it has had in the past, so it will be necessary to gather wave observation data over longer periods of time. In this section, an explanation on wave gauges, which measure wave height and period, being the two most fundamental parameters of waves is introduced. Then measurements of wave direction and the directional spectrum, and measurements of long-period waves components which have recently been pursued as important topics are also introduced. [17), 18)]

(2) Wave Gauges for observation of wave height and period

① Pressure Type Wave Gauges

Prior to the 1950's, wave measurements in Japan generally used methods such as step-resistance type wave gauges and pressure type wave gauges, but the pressure type wave gauges, which measure water pressure variations on the sea bottom, became the preferred method because they didn't require facilities such as observation towers.

However, the movement of water particles induced by deepwater waves does not reach to the sea bottom. Consequently, water pressure type wave gauges are less sensitive to short-period waves. So, the wave profiles recorded by these gauges are not the surface waveforms themselves but rather shapes that are the surface waveforms from which the short-period components have been eliminated. Also, the detection of water pressure variation is difficult in relatively deep locations, so this type of gauge has the disadvantage of being unsuitable for taking wave measurements in places of deep water.

More recently, methods of precisely calculating surface wave profile from water pressure variations have been developed and improved, so the scope of application for the water pressure type wave gauges, which offer a simple and inexpensive way of measuring waves, has started to grow again. [19), 20), 21)]

② Ultrasonic Type Wave Gauges

Ultrasonic type wave gauges, USW, were developed in the 1960's. USW have an advantage over water pressure type wave gauges in that they can obtain the surface wave profile directly, and their use has greatly increased.

An ultrasonic type wave gauge is composed of a sensor placed on the sea bottom, an underwater cable that connects the sensor with a land observation station, and an amplifier, the main body of the wave gauge, located at the land observation station. Ultrasonic signal pulses are emitted vertically upward from a sensor that is placed in a fixed location, either on the sea bottom or in the sea at a certain depth, and the ultrasonic signal reflected by the ocean surface is received at the same sensor location. The time that elapses between emission and reception is proportional to the distance of the sensor from the ocean surface, so it is possible to measure the surface wave profile by measuring this time in short intervals, about 0.5 seconds.[22), 23)]

A problem is that the detection of the ocean surface is difficult when many bubbles are gulfed near the surface, such as due to breakers, however for normal wave measurements this type of gauge has the advantage

of allowing comparatively inexpensive, accurate, and direct measurement of surface waveforms, without the need for equipment such as towers. It can also be applied to wave measurements where the sea is deep, and there are many examples where it has been set up in locations with depths of as much as 50 meters. On the other hand, a type of gauge has also come into use that is placed on the bottom of the superstructure of pier and measures the water level variations directly underneath by emitting sound through the air.[24)]

③ Buoy type wave gauges

It is possible to measure the vertical motion of the water surface with a vertical acceleration meter placed on a buoy. This has the advantage that it is also possible to measure the wave direction and directional spectrum by attaching a horizontal acceleration meter. Other advantages are that many buoys can send data wirelessly to the land observation station, so that cables are not required, and such meters are easy to use even when the water is deep. However, they cannot detect long-period components such as tsunamis and storm surge that have small accelerations.

Recently, GPS buoy systems have been developed, so that instead of using the acceleration meter method, the position coordinates of an antenna placed on a buoy are directly measured by a GPS, with short sampling intervals of one second or less, making it possible to measure not just waves but also long period components such as tsunamis, storm surge, and tidal motion.

④ Step-resistance type wave gauges

Step-resistance type wave gauges have electrodes that are insulated from each other, arranged vertically with constant spacing, that electrically turn on and off as the electrodes sink into the water or are exposed to the air, thereby detecting the water level in steps. They have the same advantage as ultrasonic type wave gauges in that they directly obtain the water surface wave profile, and furthermore they don't need calibration, but their problem is that a structure such as an observation tower is needed to fix the electrodes in place.

⑤ Capacitance type wave gauges and resistance wire type wave gauges

The principle of a capacitance type wave gauge is to vertically stretch an electric wire covered by a dielectric from under the water to the air above the sea surface, so that the electrical capacity between the wire and the sea water will vary as the up and down of the water level. This method converts variations in electrical capacity into a carrier voltage, which is amplified and recorded after the wave is detected, so its output has good linearity, and there is also is good response because a high frequency electric wave is used as the transmission wave.

On the other hand, the principle of a resistance wire type wave gauge is to extend a resistance wire vertically from the air into the water and measure the variation in the shorting distance of the resistance wire due to the up and down movement of the water level. As in the case of a capacitance type wave gauge, this also is characterized by good output linearity and response.

⑥ Optical measurement method

These include methods such as taking measurements by stereo photography from the land or air, and float tracing by memo motion cameras. Recently, surface observation by HF radar has started to be used, and there is a good possibility that technology will be developed in the future for measuring waves in the coastal area with remote sensing from artificial satellites.

(3) Observation of Wave Direction and Directional Spectra

① Overview

Although wave direction is one of the three main components of a wave, along with wave height and period, its measurement is more difficult than that of wave height, so less data has been collected than for wave height.

② Ultrasonic current meter type directional wave recorder (CWD)

The measurement principle of ultrasonic current meter type directional wave recorder is that, when ultrasonic wave is transmitted between two points in the water, and there is a flow of water, the transmission velocity will change depending on the current velocity components. The equipment measures the movement of water particles where the sensor is located, hence measurement is not possible if the wave height is small or the wave has a short period. Therefore, for the purpose of wave direction measurement, the water depth to be able to place the sensor is usually 30 meters or less. Hence, it has a major drawback of only being able to measure the directions of waves in shallow areas affected by refraction and not waves that are offshore, but at present, these meters have become the most generally used wave direction meters for waves along the coast.

③ Observation with a wave gauge array

Because CWD have the problem mentioned above, in order to observe offshore directional spectra with high precision and reliability, observations are being performed off the coasts of Iwaki and Niigata in Japan with arrays of three or four wave gauges.[26), 27), 28)]

Array observations use cross-spectra of water level variation records of the various wave gauges to estimate directional spectra. Precision and reliability are excellent, but this is very expensive compared to the usual wave observations, so it hasn't been possible to introduce the array method at many observation locations.

④ Doppler-typed wave Directional Meter (DWDM)
An oceanographic meter combines the functions of wave gauges, wave direction meters and current direction-and-speed meters in a single instrument. It is a wave observation device that employs the Doppler effect on ultrasonic signals in the water, detecting the difference in frequency between a signal that is emitted and then received by a sensor on the sea bottom, to detect the current velocity of an arbitrary intermediate layer between the sea bottom and the surface, and it is possible for a single sensor to precisely calculate the direction spectrum of an offshore wave without the need for large-scale array observations.[29), 30)]

⑤ Buoys
By measuring the vertical and horizontal components of a buoy's acceleration it is possible to measure wave directions and direction spectra. It is also possible to measure wave directions and directional spectra by measuring the horizontal displacement of buoys fitted with GPS sensors. However, one must remember that an acceleration observation method cannot detect long-period components that have small accelerations. Furthermore, the same is true of both the acceleration observation method and of the GPS method that a buoy's movement does not necessarily match the movement of the water particles that accompany waves due to the effect of winds and mooring systems, so there are cases whereby it may not be possible to measure wave directions and direction spectra.

⑥ Radar for wave direction observation
Methods for determining wave direction using high resolution millimeter wave radar to display the distribution of wave crest lines in regions of the sea have been developed, however data analysis remains a problem.

Technology to observe surface direction spectra with HF radar is still under development.

(4) Observation of Tsunamis and Long-period Waves with Offshore Wave Meters

① Recording of offshore tsunami profile
Previously, the only actual measured data at the sites of tsunamis were run-up traces and tide level recordings. Of course, such data is important for clarifying the actual conditions of tsunamis, but it is not sufficient. In particular, the tide level variation recordings measured at tide measurement wells within harbors were records of water level changes through inlet pipes, so in actuality it was extremely difficult to clearly measure vibration components with short periods of 10 minutes or less. For this reason, obtaining of offshore tsunami wave profile recordings has a significant meaning.

② Observation of long period waves
Information concerning long-period waves from continuous observation data taken using offshore wave gauges is expected to be useful not just when tsunamis occur but also at normal times.

It has recently come to be understood that it is impossible to correctly evaluate harbor calmness on the basis of significant wave height alone, as has been the custom to date. Even when the significant wave height is low and when the waves appear calm in the port, there have been many reports of cases where mooring ropes broke making cargo operations impossible or where maritime construction work was unavoidably interrupted because of large oscillations of ships and operating ships within the port due to the effects of long-period waves. Long-period waves may greatly affect the calmness of a port even if the wave height is small. This is because they can cause resonance with the natural periods of the harbor associated with the port's topography on the order of several minutes to several tens of minutes, or with the natural period of mooring systems generated by the mass of the ships and the spring constants of the mooring ropes, again, on the order of several minutes to several tens of minutes.[35)]

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