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Design Process of Coastal Facilities for Disaster Prevention

by

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Abstract

In this paper, the author introduces the current technical issues concerning the disaster prevention described in the present Design Manual of Coastal Facilities. After conducting on intensive review of this manual, numerical analysis is also conducted to develop a crucial proposal for countermeasures that should be implemented in the event of a massive disaster caused by high tide and waves exceeding a specified design level. Results of the analyses clearly indicate that it is more realistic and recommendable to introduce soft countermeasures consisting of, for example, early surveillance, efficient communication and guidance, and refuge control systems into disaster prevention plans than only intending to protect coastal areas by constructing protective facilities along the coastal area.

Keywords: *Disaster prevention, storm surge, wave run-up, wave overtopping, hazard map*

1. Conventional procedure for the protection of land area from storm surges and tsunamis

The procedure for the basic design and selection of structures to protect land area from storm surges and tsunamis is as follows.

1) The threshold needed for the protection of human life and property (planning protection line), the planned[1] tidal level and the planned wave in the area under consideration, must be confirmed.

2) Select a representative cross section in the area under consideration and set up the bottom topography with consideration of future topographic change.

3) The basic crown height of a coastal dike or a seawall is set equal to the wave run-up height of the planned wave plus an allowable height. Also, the basic crown height is set up in such a way that the wave overtopping rate is less than the allowable value for land protection.

4) If the basic crown height is too high, it may hinder the utilization of the coast and exert an adverse effect on the landscape. Also, it is dangerous to rely solely on a vertical dike and seawall, due to serious scouring and erosion in front of the structures. Thus, in order to reduce the basic crown height to the planned crown height, an appropriate planned crown height of the coastal dike and seawall must be derived, together with other measures, with due consideration given to coastal protection.

5) Each measure adopted for the reduction of the crown height can be designed using design manuals (for example, Design Manual for Coastal Facilities[1)]).

6) Evaluate the cost of each measure and select measures which are sufficiently effective in preventing high waves and tide, have the least impact on the surrounding area, and are economically viable.

7) If no appropriate measure is found, the planned height must be reevaluated. If necessary, the basic crown height, as well as the planned tidal level, the planned wave and the planned protection line, must be reevaluated.

Figure 1 shows the flow of the procedures stated above.

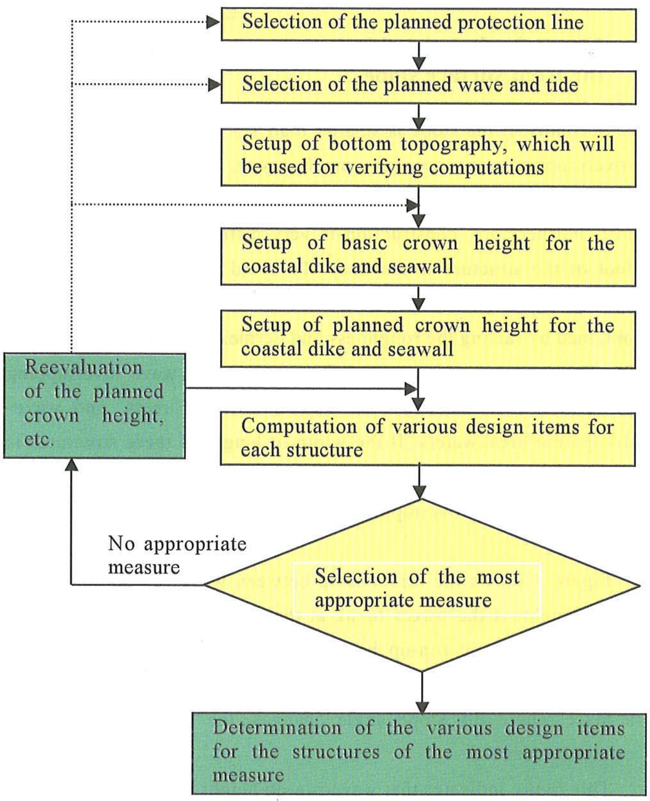

Figure 1 Design procedure of coastal facilities for the protection of land area from storm surge and tsunami [1)]

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[1] 'Planned' and 'planning' values refer to those assumptions made for a specific coast. 'Design' values refer to those assumptions made for a specific structure.

Besides the procedures stated above, the following facts should also be taken into account prior to making the concrete plan of shore protection facilities.

1) The basic structure for the protection of land area from higl waves is a combination of dikes, seawalls, and wave dissipating facilities, but if these facilities cause difficulties in beach access, selection of a dike and seawall with a mild front surface slope should be examined. Constructing a dike and seawall with a mild front surface is a means of lowering the crown height and reducing scouring at the foot of the front surface. In this case, the following two points must be taken into account.

In figure 2, past experiment data are used to show the relationship between run-up height and frontal surface slope of a coastal structure. The following facts can be inferred from this figure. First, the larger the frontal surface slope of the structure is, the higher is the wave run-up height (wave overtopping rate). Furthermore, the wave run-up height is highest for a wall whose slope namely vertical to horizontal ratio is in a range between 1:2 and 1:3.

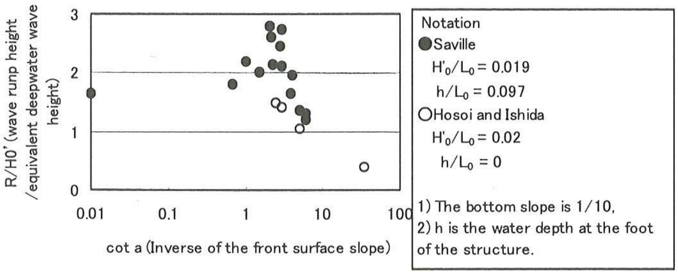

Figure 2 Relationship between wave run-up height and the front surface slope [1)]

Secondly, if the slope is smaller than 1:5, the wave run-up height (overtopping rate) becomes much lower, but the front surface extends too far seawards, and sometimes the whole sand beach is lost. Furthermore, construction is very difficult if the water at the foot of the structure is too deep. To avoid all these problems, the appropriate slope of the front surface of the structure must be obtained by raising the roughness and permeability of the surface.

2) The structures for wave dissipation and wave overtopping prevention are detached breakwaters, artificial reefs, and wave-dissipating breakwaters. If the width or height of these structures is enhanced, wave-dissipating efficiency is increased, but wave run-up height and wave overtopping rate are not always proportionally reduced.

Figure 3 shows the relationship between wave run-up height and wave height; if the waves break at the point where the slope angle changes, the wave run-up height quickly peaks. Therefore, if the dissipated wave height exceeds the height of the wave that breaks at a place where the slope angle changes, the wave run-up height cannot be reduced.

This figure indicates that wave run-up height increases as offshore wave height increases although waves break not at a point where the slope angle changes, but further offshore when the offshore wave height becomes too large. Since irregular waves include waves with

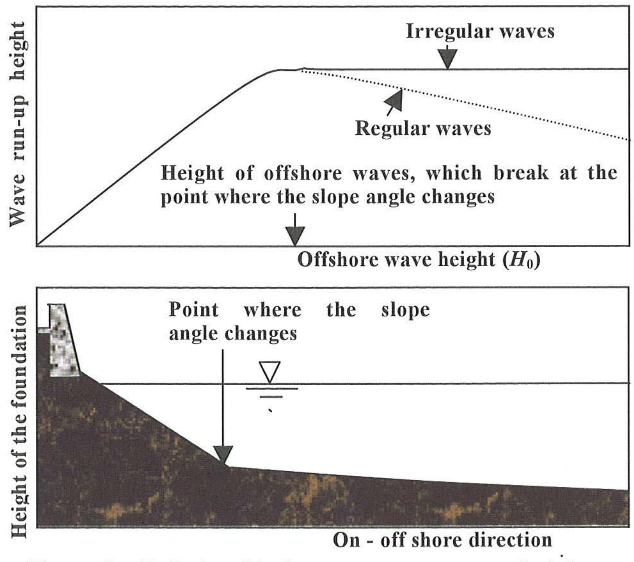

Figure 3 Relationship between wave run-up height and offshore wave height [1)]

heights lower than the significant wave height, an actual wave run-up height is not reduced after attaining a peak value.

3) When there is a port in a bay or inlet, construction of a large dike and seawall is not always profitable. Breakwaters constructed slightly offshore can prevent storm surges and tsunamis.

2. Determination of crown height of dikes and seawalls in Japan

2.1 Basic matters in determination of crown height

The crown height here is the required height of the dike and seawall to prevent wave overtopping, and is defined as follows.

Crown height[1)]	=	planned tidal level + necessary height corresponding to planned wave + allowed height

1) The following points should be considered when dealing with the planned tidal level.

When the water depth at the structure foot is less than the breaking water depth of the design waves, the wave run-up height becomes higher as the water depth becomes greater. Thus, the highest high-water level (H.H.W.L.) is used as the planned tidal level.

When the water depth at the structure foot is greater than the breaking water depth of the design waves, the planned tidal level is considered as follows. The two cases where the H.H.W.L. and the water depth at which the design waves break at the foot of the structure (if the water depth at the structure foot is coincident with the wave breaking water depth, the wave run-up height easily becomes largest) are selected as the planned tidal level should be examined, and then the wave overtopping rates of the two cases should be compared.

2) The following points must be considered when checking the necessary height corresponding to the planned wave.

The wave run-up height for the planned wave height does not always become maximum. If waves break at the point where the angle of the bottom surface slope of the dike or seawall changes, the wave run-up height quickly peaks. Then, if the wave run-up height, determined by employing the height of waves that break where the bottom surface slope angle changes, is used as the necessary height, one can expect good safety. In reality, since such a wave height is not easily determined, the design wave height is usually evaluated by varying the wave height in the range of the planned equivalent deep-water wave height to a sufficiently low wave height, then the wave run-up height corresponding to each wave height is computed. Finally, the highest wave run-up height is determined.

Since the wave run-up height becomes higher when the wave period becomes longer, it is favorable to use the planned wave period for the design wave period. Since even the height of the planned waves decreases due to dissipation, the waves with the planned wave period remain, and unless the wave steepness becomes too low, it is not necessary to make the wave period shorter.

If the waves approach the dike or seawall at a more inclined angle, the wave run-up height decreases. Therefore, the design wave direction is set as perpendicular as possible to the surface of the dike or seawall.

The cross section of the dike or seawall becomes exceedingly large if the crown height is set such that it completely prevents wave overtopping for all waves; this is inappropriate in light of construction costs and beach utilization. Therefore, of the two engineering methods explained in sections 2.2 and 2.3 for the determination of the necessary height corresponding to the planned wave, the one suitable for the coast under consideration must be used.

The cross-sectional topography used in the determination of the necessary crown height is determined using the newest bottom-sounding data. However, it is sometimes necessary to predict short-term erosion, long-term erosion, and deposition trends when dikes and seawalls are used.

3) The allowed height is set in the range of 1m, with consideration of the uncertainty in the design crown height, such as the occurrence of dangerous tidal levels higher than the planned tidal level, and the density of population and property in the back land.

In the above design process of the crown height, factors such as land subsidence and multiyear tidal level changes should be appropriately considered if it is possible to clearly predict them.

2.2 Evaluation of necessary height using wave run-up height

In 1959, when a huge typhoon hit Ise Bay, the damage was small in coastal areas where the front, crown, and back surfaces of the dike and seawall were covered with concrete and designed on the basis of the following standard.

Necessary height corresponding to planned wave[1]	=	Wave run-up height, corresponding to the significant waves, evaluated at the planned tide level

1) The crown heights of dikes and seawalls evaluated according to the above standard are sufficient if the three surfaces of the structures are covered by concrete.
2) For computational methods based on regular waves, if the incident wave height is matched to the significant wave height, the wave run-up height is assumed to correspond to the significant wave run-up height (averaged from 1/3 the highest wave run-up height computed using a series of incoming waves). If this assumption is not satisfied or the horizontal configuration is complex, then the wave run-up height must be checked with a physical model using irregular waves.
3) Even if the tidal level, significant wave parameters, and cross sectional topography are accurately estimated, the wave run-up height after wave breaking, evaluated using regular waves, is usually underestimated. The reason is that the irregularity of waves and the influence of wind on the wave run-up height are not taken into account. These treatments are described below.

1) Influence of irregularity of waves

Each band in figure 4 expresses the range of the ratio of the wave run-up height (computed using significant wave heights from Saville's graph[2] based on regular waves or Yamamoto & Horikawa's equation[3] based on regular and irregular waves) to the observed significant wave run-up height.

If the average bottom slope between the wave-breaking point and the wave run-up height is small, the wave run-up heights computed using regular waves are significantly underestimated; the errors can not be negligibly small, particularly if the average bottom slope is smaller than 1/20[4,5].

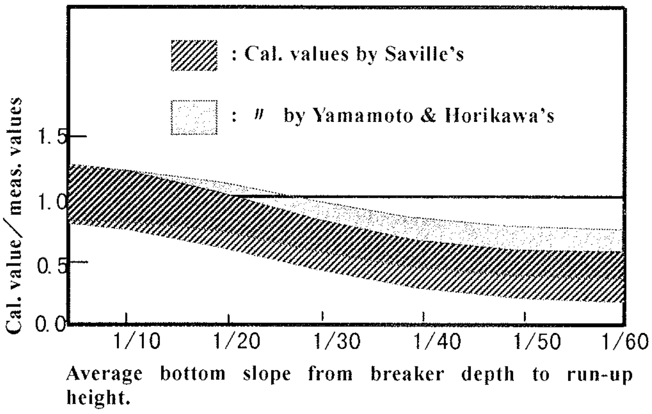

Figure 4 Comparison of significant wave run-up height observed with the computed one

It is not possible to explain this phenomenon if the significant wave run-up height is evaluated in such a way that each wave run-up height is calculated by methods based on regular waves and treated by a statistical superposition method. As pointed out by Iwata[6] and treated by Yamamoto and coworkers[4,5], it is necessary to account for the influence of long-period waves and the absorption of small waves by large waves.

A specific group of sea waves of irregular patterns are termed "wave groups", and their run-up wave patterns change depending on the bottom surface slope. This is because each wave breaks at a point farther offshore and becomes smaller as bottom slope becomes smaller, and thus the influence of long-period waves (which are hard to break) on run-up waves becomes more significant. This is particularly true if long-period waves generated by periodic variation of the mean water level becomes predominant in the breaker zone, which are caused by periodic movement of breakpoint due to wave grouping.

Nakaza et al.[7], studying with reef coasts, and Yamamoto and his coworkers[5, 8], studying wide surf zones of usual coasts, pointed out that long-period waves with the same phase as wave groups can easily produce peak run-up waves. Others reported field survey results[9),10]. If the average bottom slope between the breaker line and the wave run-up line is less than 1/20, the influence of long period waves must be accounted for. The method for evaluating this influence can be found in various research papers [5, 8, 11, 12].

2) Influence of wind

A notable influence of wind velocity on the wave run-up height[13] is often apparent even for wind velocities that do not exceed 20m/s. In addition, because notable increases in water levels due to winds with velocities higher than 10m/s were recorded at numerous observational points, it is necessary to consider the influence of wind on wave run-up height[8] in cases where wind with velocity exceeding 10m/s blows in the same direction as wave propagation.

However, results obtained in wind tunnel experiments are not sufficient for application to all coasts. It is indispensable to carry out wind tunnel experiments and field observation and store the obtained data as much as possible.

2.3 Evaluation of necessary height by overtopping rate

If the land use density of the back land is high, it is inadequate to set only the standard of non-breakable seawall; in consideration of the importance of land use in the back land, a crown height with an allowable overtopping rate must be designed. The standards for determining the necessary crown height with consideration of the overtopping rate are as follows.

Necessary height corresponding to planned wave[1]	=	Height at which the overtopping rate due to irregular waves, evaluated at the planned tide level, is smaller than the allowable value [14]

Since the overtopping rate is proportional to the square of the overtopping height (overtopping height equals to the wave run-up height minus the crown height of the dike or seawall), the peak overtopping rate follows conditions similar to those of wave run-up height. Thus, the treatment of the design wave and design tide is similar to that of the wave run-up height. However, in spite of the significant wave run-up height on a real coast being lower than the crown height, the overtopping rate on the real coast sometimes is not zero but more than the allowable overtopping rate. Therefore, calculation methods of the overtopping rate based on irregular waves must be used.

Goda et al.'s graph[15] can be used for a vertical dike or seawall (with slope of the front surface steeper than 1:1) built on a narrow beach or on a place where there is no beach at all. Takayama and his coworkers[16),17] have shown that the overtopping rate decreases when waves arrive at a more inclined angle. If the water depth at the foot of the structure is larger than the incoming significant wave height, the average overtopping rate for multidirectional irregular waves is about 0.7 to 0.8 that for unidirectional irregular waves[18, 19].

For a beach, or for a sloping dike and seawall, the formula of Yamamoto et al. [20] can be used. However, when the average bottom slope from the wave-breaking point to wave run-up height is less than 1/20, the influence of long-period waves must be accounted for[8),11),12].

Numerical models, such as the Boussinesq approximation model[8] or the VOF (Volume of Fluid) method[21], have enabled significant progress.

If wind blows in the same direction as wave propagation, then the overtopping rate increases, and it becomes necessary to treat it in the same way as wave run-up. There are several empirical formulas for the relationship between wind and the increase in the overtopping rate[22]. However, it is necessary to confirm the applicability of these formulas using field data.

3. Countermeasures to prevent major disasters resulting from high tides and waves that exceed the design level

In Section 1 and Section 2, the basic concept of the design method for coastal protection facilities, and the newest method to evaluate the influence of wind and irregular waves, were introduced.

In Section 3, review will be made on the effects of the phenomena that the accumulation of population and property has advanced into lowland areas and that global warming brings continuous rise of sea elevation and appearance of extreme weather patterns. Since the population and property in Japan are concentrated in coastal zones, Japanese people have frequently suffered damage from high tides and waves. The Suruga coast in Shizuoka Prefecture is a typical example. The Shizuoka Office of River (Ministry of Land and Infrastructure and Transport), which manages this coast, is concerned about preventing calamities resulting from high tides and waves. The author, Vu Thanh Ca, and Satoru Kawashima of INA Cooperation have carried out a study on how coastal protection facilities should be planned in preparation for the occurrence of high tides and waves exceeding the design level. This study was conducted under a contract with the Shizuoka Office of River.

3.1 Case study of Suruga coast

The author and his colleagues improved the numerical model PWRI[23] in order to predict a unsteady flood due to storm surge, and performed a flood calculation for cases where high tides and waves exceeding the design level hit the Suruga coast. We used an improved model, the details of which are described below.

1) Basic equations of numerical model

Basic equations of the numerical model are expressed as follows.

The continuity equation of a flood is

$$\frac{\partial f_y q_x}{\partial x}+\frac{\partial f_x q_y}{\partial y}+\frac{\partial S\eta}{\partial t}=0, \tag{1}$$

the motion equation in the x-direction is

$$\frac{\partial q_x}{\partial t}+\frac{1}{S}\frac{\partial}{\partial x}\left(\frac{Sq_x^2}{d}\right)+\frac{1}{S}\frac{\partial}{\partial y}\left(\frac{Sq_xq_y}{d}\right)+gd\frac{\partial\eta}{\partial x}-\frac{1}{S}\frac{\partial}{\partial x}\left[dv_tS\frac{\partial(q_x/d)}{\partial x}\right] \tag{2}$$
$$-\frac{1}{S}\frac{\partial}{\partial y}\left[dv_tS\frac{\partial(q_x/d)}{\partial y}\right]+\frac{f_c}{d^2}Qq_x=0,$$

and the motion equation in the y-direction is

$$\frac{\partial q_y}{\partial t}+\frac{1}{S}\frac{\partial}{\partial x}\left(\frac{Sq_yq_x}{d}\right)+\frac{1}{S}\frac{\partial}{\partial y}\left(\frac{Sq_y^2}{d}\right)+gd\frac{\partial\eta}{\partial y}-\frac{1}{S}\frac{\partial}{\partial x}\left[dv_tS\frac{\partial(q_y/d)}{\partial x}\right] \tag{3}$$
$$-\frac{1}{S}\frac{\partial}{\partial y}\left[dv_tS\frac{\partial(q_y/d)}{\partial y}\right]+\frac{f_c}{d^2}Qq_y=0,$$

where q_x and q_y are the horizontal fluid fluxes in the x- and y-directions respectively, η is the surface elevation, f_x and f_y are the x- and y-direction ratios of the wet portion in a calculation mesh, S is the area of the wet portion in the calculation mesh, d is the water depth, g is acceleration due to gravity, ν_t is the eddy viscosity coefficient, and f_c is the resistance coefficient due to houses, buildings and trees,

$$f_c=\frac{gn^2}{d^{\frac{1}{3}}}. \tag{4}$$

Here, n is the roughness coefficient and follows the model PWRI[23].

2) Setup of calculation scheme and calculation time

In order for this numerical model to be applicable to different calculations by the Crank-Nicholson method, the calculation time interval ⊿t is decided according to C.F.L.'s stability condition, and the calculation is repeated until it fully converges.

The coordinate system and difference meshes are shown in Figure 5.

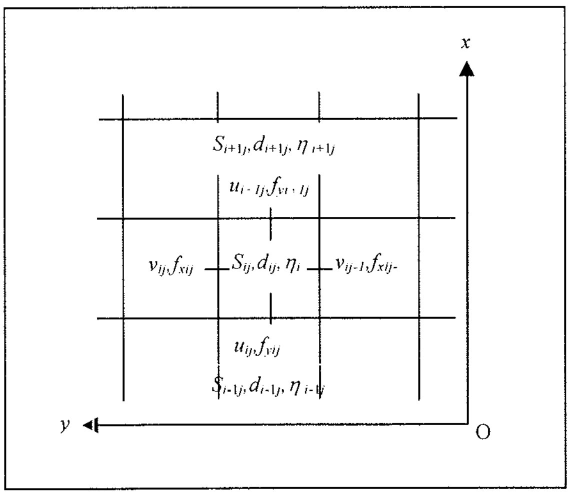

Figure 5 Coordinate system and difference meshes

The differential equations (1)～(3) can be converted into difference equations shown in equations (5)～(7).

$$0.5\left(\frac{f^n_{yi+1j}q^n_{xi+1j}-f^n_{yij}q^n_{xij}}{\Delta x}+\frac{f^n_{xij}q^n_{yij}-f^n_{xij-1}q^n_{yij-1}}{\Delta y}\right)$$
$$+0.5\left(\frac{f^o_{yi+1j}q^o_{xi+1j}-f^o_{yij}q^o_{xij}}{\Delta x}+\frac{f^o_{xij}q^o_{yij}-f^o_{xij-1}q^o_{yij-1}}{\Delta y}\right)+\frac{S^n_{ij}\eta^n_{ij}-S^o_{ij}\eta^o_{ij}}{\Delta t}=0, \tag{5}$$

$$\frac{q^n_{xij}-q^o_{xij}}{\Delta t}+0.5AD_{xx}q^n_x+0.5AD_{xx}q^o_x+0.5AD_{xy}q^n_x+0.5AD_{xy}q^o_x+g\frac{d^n_{ij}+d^n_{i-1j}}{2}\frac{\eta^n_{ij}-\eta^n_{i-1j}}{\Delta x}$$
$$+g\frac{d^o_{ij}+d^o_{i-1j}}{2}\frac{\eta^o_{ij}-\eta^o_{i-1j}}{\Delta x}$$
$$-\frac{2}{\left(S^n_{ij}+S^n_{i-1j}\right)\Delta x^2}\left[S^n_{ij}d^n_{ij}\nu^n_{tij}\left(\frac{q^n_{xi+1j}}{d^n_{ij}+d^n_{i+1j}}-\frac{q^n_{xij}}{d^n_{ij}+d^n_{i-1j}}\right)-S^n_{i-1j}d^n_{i-1j}\nu^n_{ti-1j}\left(\frac{q^n_{xij}}{d^n_{ij}+d^n_{i-1j}}-\frac{q^n_{xi-1j}}{d^n_{i-1j}+d^n_{i-2j}}\right)\right]$$
$$-\frac{2}{\left(S^o_{ij}+S^o_{i-1j}\right)\Delta x^2}\left[S^o_{ij}d^o_{ij}\nu^o_{tij}\left(\frac{q^o_{xi+1j}}{d^o_{ij}+d^o_{i+1j}}-\frac{q^o_{xij}}{d^o_{ij}+d^o_{i-1j}}\right)-S^o_{i-1j}d^o_{i-1j}\nu^o_{ti-1j}\left(\frac{q^o_{xij}}{d^o_{ij}+d^o_{i-1j}}-\frac{q^o_{xi-1j}}{d^o_{i-1j}+d^o_{i-2j}}\right)\right]$$
$$-\frac{2}{\left(\bar{S}^n_{ij}+\bar{S}^n_{ij-1}\right)\Delta y^2}\left[\bar{S}^n_{ij}\bar{d}^n_{ij}\bar{\nu}^n_{tij}\left(\frac{q^n_{xij+1}}{d^n_{ij+1}+d^n_{i+1j+1}}-\frac{q^n_{xij}}{d^n_{ij}+d^n_{i-1j}}\right)-\bar{S}^n_{ij-1}\bar{d}^n_{ij-1}\bar{\nu}^n_{ij-1}\left(\frac{q^n_{xij}}{d^n_{ij}+d^n_{i-1j}}-\frac{q^n_{xij-1}}{d^n_{i-1j-1}+d^n_{ij-1}}\right)\right]$$
$$-\frac{1}{\left(\bar{S}^o_{ij}+\bar{S}^o_{ij-1}\right)\Delta y^2}\left[\bar{S}^o_{ij}\bar{d}^o_{ij}\bar{\nu}^o_{tij}\left(\frac{q^o_{xij+1}}{d^o_{ij+1}+d^o_{i+1j+1}}-\frac{q^o_{xij}}{d^o_{ij}+d^o_{i+1j}}\right)-\bar{S}^o_{ij-1}\bar{d}^o_{ij-1}\bar{\nu}^o_{ij-1}\left(\frac{q^o_{xij}}{d^o_{ij}+d^o_{i+1j}}-\frac{q^o_{xij-1}}{d^o_{i+1j-1}+d^o_{ij-1}}\right)\right]$$
$$+\frac{2f^n_c}{\left(d^n_{ij}+d^n_{i-1j}\right)^2}Q^n_{xij}q^n_{xij}+\frac{2f^o_c}{\left(d^o_{ij}+d^o_{i-1j}\right)^2}Q^o_{xij}q^o_{xij}=0\ , \tag{6}$$

$$\frac{q^n_{yij}-q^o_{yij}}{\Delta t}+0.5AD_{yx}q^n_y+0.5AD_{yx}q^o_y+0.5AD_{yy}q^n_y+0.5AD_{yy}q^o_y+g\frac{d^n_{ij}+d^n_{ij+1}}{2}\frac{\eta^n_{ij+1}-\eta^n_{ij}}{\Delta x}$$
$$+g\frac{d^o_{ij}+d^o_{ij+1}}{2}\frac{\eta^o_{ij+1}-\eta^o_{ij}}{\Delta x}$$
$$-\frac{2}{\left(S^n_{ij}+S^n_{ij+1}\right)\Delta y^2}\left[S^n_{ij+1}d^n_{ij+1}\nu^n_{tij+1}\left(\frac{q^n_{yij+1}}{d^n_{ij}+d^n_{ij+1}}-\frac{q^n_{yij}}{d^n_{ij}+d^n_{ij+1}}\right)-S^n_{ij}d^n_{ij}\nu^n_{tij}\left(\frac{q^n_{yij}}{d^n_{ij}+d^n_{ij+1}}-\frac{q^n_{yij-1}}{d^n_{ij}+d^n_{ij-1}}\right)\right]$$
$$-\frac{2}{\left(S^o_{ij}+S^o_{ij+1}\right)\Delta y^2}\left[S^o_{ij+1}d^o_{ij+1}\nu^o_{tij+1}\left(\frac{q^o_{yij+1}}{d^o_{ij}+d^o_{ij+1}}-\frac{q^o_{yij}}{d^o_{ij}+d^o_{ij+1}}\right)-S^o_{ij}d^o_{ij}\nu^o_{tij}\left(\frac{q^o_{yij}}{d^o_{ij}+d^o_{ij+1}}-\frac{q^o_{yij-1}}{d^o_{ij}+d^o_{ij-1}}\right)\right]$$
$$-\frac{1}{\left(\bar{S}^n_{ij}+\bar{S}^n_{i+1j}\right)\Delta x^2}\left[\bar{S}^n_{i+1j}\bar{d}^n_{i+1j}\bar{\nu}^n_{ti+1j}\left(\frac{q^n_{yi+1j}}{d^n_{i+1j}+d^n_{i+1j+1}}-\frac{q^n_{yij}}{d^n_{ij}+d^n_{ij+1}}\right)-\bar{S}^n_{ij}\bar{d}^n_{ij}\bar{\nu}^n_{ij}\left(\frac{q^n_{yij}}{d^n_{ij}+d^n_{ij+1}}-\frac{q^n_{yi-1j}}{d^n_{i-1j}+d^n_{i-1j+1}}\right)\right]$$
$$-\frac{1}{\left(\bar{S}^o_{ij}+\bar{S}^o_{i+1j}\right)\Delta x^2}\left[\bar{S}^o_{i+1j}\bar{d}^o_{i+1j}\bar{\nu}^o_{ti+1j}\left(\frac{q^o_{yi+1j}}{d^o_{i+1j}+d^o_{i+1j+1}}-\frac{q^o_{yij}}{d^o_{ij}+d^o_{ij+1}}\right)-\bar{S}^o_{ij}\bar{d}^o_{ij}\bar{\nu}^o_{ij}\left(\frac{q^o_{yij}}{d^o_{ij}+d^o_{ij+1}}-\frac{q^o_{yi-1j}}{d^o_{i-1j}+d^o_{i-1j+1}}\right)\right]$$
$$+\frac{2f^n_c}{\left(d^n_{ij}+d^n_{ij+1}\right)^2}Q^n_{yij}q^n_{yij}+\frac{2f^o_c}{\left(d^o_{ij}+d^o_{ij+1}\right)^2}Q^o_{yij}q^o_{yij}=0\ , \tag{7}$$

where

$$\bar{S}_{ij}=\frac{S_{ij}+S_{i-1j}+S_{ij+1}+S_{i-1j+1}}{4}$$

$$Q_{xij}=\sqrt{\bar{q}_{yij}^2+q_{xij}^2}$$

$$Q_{yij}=\sqrt{\bar{q}_{xij}^2+q_{yij}^2}$$

$$\bar{q}_{yij}=\frac{q_{yij}+q_{yij-1}+q_{yi-1j}+q_{yij-1}}{4}$$

$$\bar{q}_{xij}=\frac{q_{xij}+q_{xi+1j}+q_{xi+1j+1}+q_{xij+1}}{4}$$

$$AD_{xx}q_x=AD_{xi+1j}q_{xi+1j}+AD_{xij}q_{xij}+AD_{xi-1j}q_{xi-1j}.$$

In the case of $q_{xij}<0$,

$$AD_{xi+1j}=\frac{S_{ij}+S_{i+1j}}{S_{ij}\Delta x}\frac{q_{xi+1j}}{d_{i+1j}+d_{ij}}$$

$$AD_{xij} = -\frac{S_{ij} + S_{i-1j}}{S_{ij}\Delta x}\frac{q_{xij}}{d_{i-1j} + d_{ij}}$$

$$AD_{xi-1j} = 0.$$

In the case of $q_{xij} > 0$,

$$AD_{xi+1j} = 0$$

$$AD_{xij} = \frac{S_{ij} + S_{i-1j}}{S_{i-1j}\Delta x}\frac{q_{xij}}{d_{i-1j} + d_{ij}}$$

$$AD_{xi-1j} = -\frac{S_{i-1j} + S_{i-2j}}{S_{i-1j}\Delta x}\frac{q_{xi-1j}}{d_{i-1j} + d_{i-2j}}$$

$$AD_{xy} q_x = AD_{xyij+1} q_{xij+1} + AD_{xyij} q_{xij} + AD_{xyij-1} q_{xij-1}.$$

In the case of $\bar{q}_{yij} < 0$,

$$AD_{xyij+1} = \frac{\bar{S}_{ij} + \bar{S}_{ij+1}}{\bar{S}_{ij}\Delta y}\frac{\bar{q}_{yij+1}}{d_{i+1j+1} + d_{ij+1}}$$

$$AD_{xyij} = -\frac{\bar{S}_{ij} + \bar{S}_{ij-1}}{\bar{S}_{ij}\Delta y}\frac{\bar{q}_{yij}}{d_{i-1j} + d_{ij}}$$

$$AD_{xyij-1} = 0$$

In the case of $\bar{q}_{yij} > 0$,

$$AD_{xyij+1} = 0$$

$$AD_{xyij} = \frac{\bar{S}_{ij} + \bar{S}_{ij-1}}{\bar{S}_{ij-1}\Delta y}\frac{\bar{q}_{yij}}{d_{i-1j} + d_{ij}}$$

$$AD_{xyij-1} = -\frac{\bar{S}_{ij-2} + \bar{S}_{ij-1}}{\bar{S}_{ij-1}\Delta y}\frac{\bar{q}_{yij-1}}{d_{i-1j-1} + d_{ij-1}}$$

$$AD_{yx} q_y = AD_{yxi+1j} q_{yi+1j} + AD_{yxij} q_{yij} + AD_{yxi-1j} q_{yi-1j}.$$

In the case of $\bar{q}_{xij} < 0$.

$$AD_{yxi+1j} = \frac{\bar{S}_{i+2j} + \bar{S}_{i+1j}}{\bar{S}_{i+1j}\Delta x}\frac{\bar{q}_{xi+1j}}{d_{i+1j} + d_{i+1j+1}}$$

$$AD_{yxij} = -\frac{\bar{S}_{ij} + \bar{S}_{i+1j}}{\bar{S}_{i+1j}\Delta x}\frac{\bar{q}_{xij}}{d_{ij} + d_{ij+1}}$$

$$AD_{yxij-1} = 0.$$

In the case of $\bar{q}_{xij} < 0$,

$$AD_{yxi+1j} = 0$$

$$AD_{yxij} = \frac{\bar{S}_{ij} + \bar{S}_{i+1j}}{\bar{S}_{ij}\Delta x}\frac{\bar{q}_{xij}}{d_{ij} + d_{ij+1}}$$

$$AD_{yxij-1} = \frac{\bar{S}_{i-1j} + \bar{S}_{ij}}{\bar{S}_{ij}\Delta x}\frac{\bar{q}_{xi-1j}}{d_{i-1j} + d_{i-1j+1}}$$

$$AD_{yy} q_y = AD_{yij+1} q_{yij+1} + AD_{yij} q_{yij} + AD_{yij-1} q_{yij-1}.$$

In the case of $q_{yij} < 0$,

$$AD_{yij+1} = \frac{S_{ij+2} + S_{ij+1}}{S_{ij+1}\Delta y}\frac{q_{yij+1}}{d_{ij+2} + d_{ij+1}}$$

$$AD_{yij} = -\frac{S_{ij} + S_{ij+1}}{S_{ij+1}\Delta y}\frac{q_{xij}}{d_{i-1j} + d_{ij}}$$

$$AD_{yij-1} = 0.$$

In the case of $q_{yij} > 0$,

$$AD_{yij+1} = 0$$

$$AD_{yij} = \frac{S_{ij} + S_{ij+1}}{S_{ij+1}\Delta y}\frac{q_{xij}}{d_{i-1j} + d_{ij}}$$

$$AD_{yij-1} = -\frac{S_{ij-1} + S_{ij}}{S_{ij}\Delta y}\frac{q_{yij+1}}{d_{ij+2} + d_{ij+1}}.$$

3) Handling a flood tip

At the flood tip, the wet portion in a mesh is part of the mesh, and the ratio S of the wet portion of the mesh area is one or less. In order to estimate S, it is necessary to find the velocity of the flood tip in the mesh.

Figure 6 shows this calculation scheme. Assuming that water enters from four directions in the mesh, the propagation speed of each flood tip is expressed as follows:

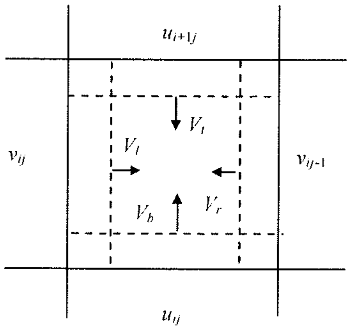

Figure 6 Treatment of flood tip

$$V_t = \max\left[\sqrt{g\left(d_{ij} + d_{i+1j}\right)} - u_{i+1j}, 0\right] \tag{8}$$

$$V_b = \max\left[\sqrt{g\left(d_{ij} + d_{i-1j}\right)} + u_{ij}, 0\right] \tag{9}$$

$$V_l = \max\left[\sqrt{g\left(d_{ij} + d_{ij+1}\right)} - v_{ij}, 0\right] \tag{10}$$

$$V_r = \max\left[\sqrt{g\left(d_{ij} + d_{ij-1}\right)} + v_{ij-1}, 0\right]. \tag{11}$$

When the distance from the mesh boundary in each direction is obtained using each propagation speed, the ratio of the wet portion area can be found by

$$S = 1 - \max\left[\left(1 - l_t / dx - l_d / dx\right), 0\right] \times \max\left[\left(1 - l_l / dx - l_r / dx\right), 0\right]. \tag{12}$$

However, if the water depth in the mesh becomes 1mm or less, ratio S is regarded as zero.

4) Flood calculation procedure

The following is the procedure for flood calculation.

1) Calculate wave overtopping quantity on the coastline corresponding to the first incident wave and the first tide level using Goda's diagrams[1].

2) Insert the wave overtopping quantity to the numerical model and simulate a landward flood.

3) Calculate wave overtopping quantity corresponding to the next incident wave and the next tide level and repeat the flood simulation.

5) Predicted cases and results

The topography and main planning items for the Suruga coast are shown in Figure 7 and Table 1, respectively.

Flood simulation was conducted for two cases (see Table 2) for the Suruga coast using the numerical model described above.

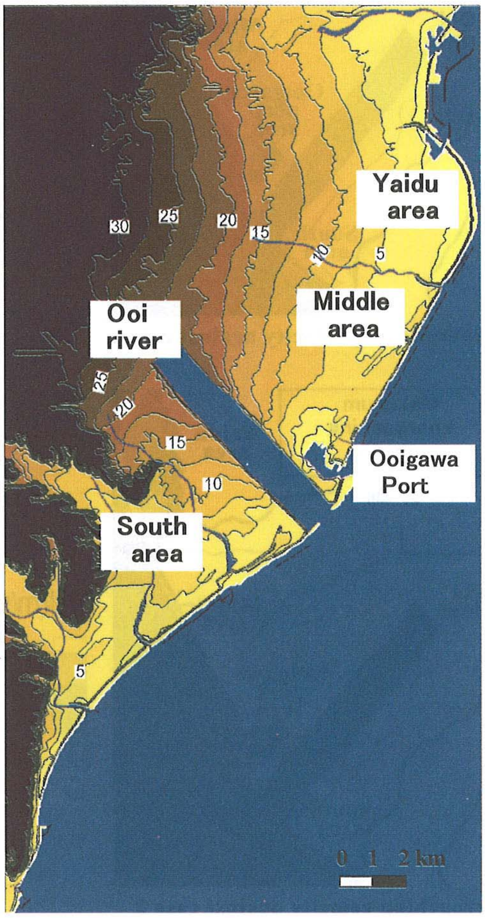

Figure 7 Topography of Suruga coast

Table 1 Main planning items for Suruga coast

Planning wave and tide				
Offshore wave height	Wave period	Wave direction		Tidal level
9m	14s	SW		T.P.+1.66m
Return period of about 25 years (the significant wave at the time of the Ise Bay typhoon was estimated)				H.H.W.L.
Main planning items of present coastal protection works				
Name of area	Yaidu	Middle	Ooigawa port	South
Crown height of dike	T.P.+8.2m	T.P.+6.2m	T.P.+6.2m	T.P.+6.2m
Beach width at the time of T.P.+0.0m	>120m	=80m	=30～300m	>120m
The number of Detached breakwater	0	28	2	5

Table 2 Calculation conditions of predicted cases

Case	Coastal facilities	Peak values of wave height and period	Peak tidal level
1	Present coastal protection works are completed	9m and 14s (present design wave)	H.H.W.L +48.5cm
2	Same as above	11.3m and 16s (wave with a return period of 50 years)	H.H.W.L. +48.5cm

The incident wave height increases consistently from half the peak value, to the peak value for 12 hours, and then decreases to half the peak value again for 12 hours. The incident period is fixed to the peak value. The tidal level increases consistently from H.W.L. plus 48.5cm, to H.H.W.L. plus 48.5cm for 12 hours, then decreases to H.W.L. plus 48.5cm again in the next 12 hours. The value of 48.5cm is the average rise of water level predicted by IPCC[24] in 2001. The reduction ratio of transmitted height due to detached breakwaters is 0.6.

Results are shown in Figures 8 and 9. The respective figures show the maximum submerged depth and the maximum encroaching velocity at each point within the calculation time. In Case 1, the flood region deeper than 50cm spreads out in the Yaidu area and the Ooigawa Port area where the lowland ground level is 2-5m. In Case 2, the flood region deeper than 50cm is double that of Case 1, and spreads into the middle and southern areas. Even though coastal protection works have been completed, it is clear that the lowlands will eventually be afflicted by severe disaster. It would cost an additional 30 billion yen to construct detached breakwaters at Yaidu, Ooigawa Port, and in the southern areas. This clearly indicates that it would not be realistic to protect whole coast of Japan by protection facilities.

3.2 Countermeasures against natural disasters with force exceeding the planned values

Assuming high tides and waves are generated only once approximately every 50 years, it is not realistic to completely prevent disaster by paying immense costs necessary to construct large protection facilities.

The following countermeasures are more economical and realistic than the construction of large protection facilities.

1) Large dikes are used only to enclose and protect high-density zones in terms of population and property.
2) Important buildings, such as public offices and hospitals, should be made flood-proof by raising foundations and floors.
3) Raised floor shelters should be placed at appropriate locations.
4) An effective early warning/evacuation system should be designed. This should consist of surveillance networks that include, for example, remote control cameras connected by optical fiber, multiple communications and alarm networks connected by cables and radio, hazard maps, and evacuation guides.
5) Lastly, training should be carried out repeatedly until all residents become accustomed to the details of the evacuation plan.

4. Concluding remarks

This study revealed that it is more realistic to use countermeasures consisting of early surveillance, communication and guidance, and evacuation control systems, together with the construction of protection facilities, rather than to rely solely on the construction of massive protection facilities, to protect coastal areas. It was emphasized, however, that training should be carried out repeatedly until all lowland residents become accustomed to the evacuation plan.

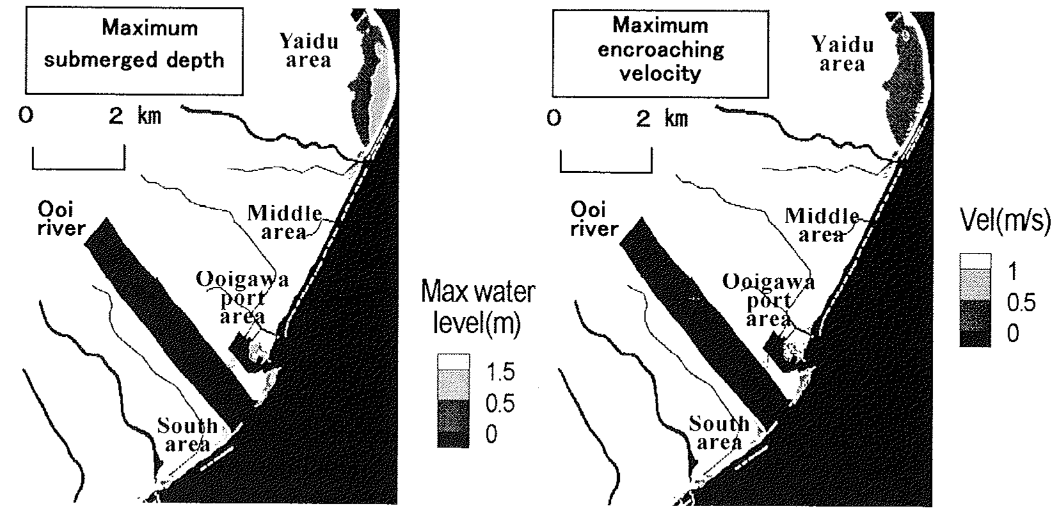

Figure 8 Distribution of submerged depth and encroaching velocity against case 1

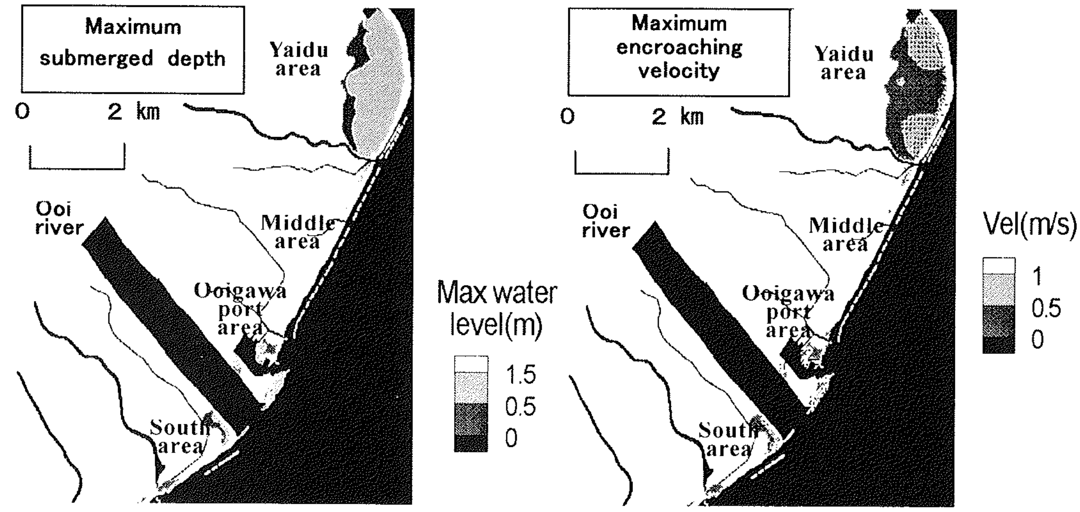

Figure 9 Distribution of submerged depth and encroaching velocity against case 2

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