

## CHAPTER 73

### SOME CONTRIBUTIONS TO HYDRAULIC MODEL EXPERIMENTS IN COASTAL ENGINEERING

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#### ABSTRACT

This paper presents some aspects of the hydraulic model experiments in coastal engineering made at the Ujigawa Hydraulic Laboratory, Disaster Prevention Research Institute, Kyoto University, including the experiments performed by using an estuary model basin and a high speed wind wave channel. In particular, the problems to which attentions should be paid from view point of similitude between model and prototype will be discussed in addition to the presentation of experimental results.

#### INTRODUCTION

Various kinds of experimental facilities are used for hydraulic studies in coastal engineering at present. The most general type among these facilities is of a wave tank or wave basin having a short period wave generator to carry out the experiments involving the transformation of waves and wave actions on shore structures and bottom materials. This type of the facility and the experimental technique have become quite popular recently except the model study of bottom sand movement due to waves.

There are two specialized experimental facilities for coastal engineering researches at the Ujigawa Hydraulic Laboratory, Disaster Prevention Research Institute, Kyoto University; one is an estuary model basin which is used for the model experiment involving tidal motion and the other is a high speed wind wave channel, which is for the experimental studies of wave overtopping on seawalls and the wind wave generation and development.

Since the land reclamation works along coasts have become very active since several years ago in Japan for industrial and agricultural developments,

the necessity of predicting the change of tidal motion nearshore resulting from the change of coastal configuration due to land reclamations has occurred in addition to the protection of those areas by shore structures from a storm surge and associated waves. Thus, the hydraulic model experiment involving tidal motion have been adopted as one of the best ways to solve these problems. Besides such an experiment, the problem of wave overtopping on a seawall has become important in designing the seawall of a reclaimed land to be constructed at comparatively deep water depth. From this reason, the precise and comprehensive study of wave overtopping, in particular the effect of wind on wave overtopping has been required.

The experimental studies by the estuary model basin and the high speed wind wave channel satisfy the requirement described above and in fact a number of experiments have been performed by using the both facilities. The purpose of this paper is to introduce the experimental results obtained by these two facilities and in addition discuss some problems to which attentions should be paid from view point of similitude between model and prototype, and practical applications.

#### EXPERIMENTS BY THE ESTUARY MODEL BASIN

##### EXPERIMENTAL FACILITIES

The model basin is 25 m wide, 35 m long and 0.4 m deep. The tide is generated by a pneumatic tide generator consisting of an air chamber, a roots blower of 7.5 HP and a control valve as shown in Fig. 1, which is operated by an automatic control system. The tidal range is 4 cm in maximum and sinusoidal waves of 1 to 60 min. period or waves of optional forms are provided. In addition, river flow can also be provided up to 20 l/sec by a pump of 7.5 HP and lateral current can be generated up to 80 l/sec by two pumps of 10 HP.

The water level is measured by wave meters of electric resistance type and the current velocity is observed by photographing intermittently a number of floats distributed on the water surface.

##### SIMILITUDE

The most important problem in the hydraulic model experiment is to satisfy the dynamical similitude between the prototype and the model. In treating the behaviors of long period waves and associated currents such as tides and tidal currents, it is necessary to satisfy the following equations in order to hold the dynamical similitude including the frictional effect:

$$t_r = x_r / z_r^{1/2} \quad (1)$$

$$C_{fr} = z_r / x_r \quad (2)$$

in which  $x$  is the horizontal length,  $z$  the vertical length,  $t$  the time,  $C_f$  the frictional coefficient and the suffix  $r$  denotes the ratio of the quantity in the prototype to that in the model.

If the flow in the model for the experiment of tidal current is lamnar, and it is assumed that  $C_f = 1.328 \text{ Re}^{-1/2}$ , Eq. (2) is rewritten as

$$z_r^{\frac{5}{4}} = \frac{C_{fp}}{1.328} \sqrt{x_r \text{Re}_p} \quad (3)$$

and on the other hand, if the flow is turbulent and the Manning formula is applied as the law of resistance, Eq. (2) becomes

$$n_r = x_r^{-\frac{1}{2}} z_r^{\frac{2}{3}} \quad (4)$$

in which the suffix p denotes the quantity in the prototype, Re is the Reynolds number constructed by the maximum velocity of tidal current  $U_{\max}$  and the length of tidal excursion L, and n the Manning roughness coefficient. When the time variation of the tidal current is sinusoidal, the length of tidal excursion is expressed with the maximum velocity and the tidal period T as follows:

$$L = (1/\pi) U_{\max} T \quad (5)$$

Then the Reynolds number becomes

$$\text{Re} = U_{\max} L / \nu = U_{\max}^2 T / \pi \nu \quad (6)$$

in which  $\nu$  is the kinematic viscosity.

Strictly speaking, it is impossible to satisfy completely Eq. (3), even if the flow in the model is in laminar regime in the whole area, because  $C_{fp}$  is a function of the time and the space, and also  $\text{Re}_p$  is a function of the space. However, the space and the time under consideration are restricted or the representative space and time are selected as an approximation,  $C_{fp}$  and  $\text{Re}_p$  can be evaluated, so that the horizontal and vertical scales can be decided from Eq. (3).

#### KINDS OF MODEL EXPERIMENTS

The model experiments carried out by this basin are of the tidal currents in Hiroshima Bay by Hayami, Higuchi and Yoshida (1958) and Nagoya Harbor by Higuchi and Yoshida (1964), and the sea level oscillations in Sakai Channel by Higuchi (1961) and Nagoya Harbor by Higuchi (1964) as shown in Table 1. Fig. 2 shows a layout of the Nagoya Harbor model as an example of the models constructed in the estuary model basin. In addition to the model experiment of Nagoya Harbor using this basin, the preliminary experiment was carried out, in which the horizontal and vertical scales are 1/2000 and 1/667 respectively, and a plunger type tide generator was used (Higuchi and Yoshida (1964)).

#### SUMMARY OF EXPERIMENTAL RESULTS

Hiroshima Bay - From the comparison between the flow patterns of tidal currents in the models of distortion ratios 2, 4 and 8 and the prototype, it was found that the flow pattern in the model of distortion ratio 2 is most similar to that in the prototype. This fact can be explained theoretically based on Eq. (3) under the assumption that the frictional coefficient  $C_{fp}$  and the Reynolds number  $\text{Re}_p$  in the prototype are  $4 \times 10^{-3}$  to  $5 \times 10^{-3}$  and  $1.4 \times 10^8$  ( $U_{\max} = 9.7$  cm/sec and  $L = 1.4 \times 10^5$  cm) respectively.

Sakai Channel - Sakai Channel is 7.5 km long, 200 to 800 m wide and 5 m deep.

Table 1. Kinds of models and experiments

Place	Model scale		Kinds of experiments	Flow regime
	Horizontal	Vertical		
Hiroshima Bay	1/500	3 kinds	Flow pattern and the effect of model distortion	laminar
Sakai Channel	1/500	5 kinds	Frequency response of sea level oscillation, and the effect of model distortion	turbulent
Nagoya Harbor	1/700	1/500	Flow pattern, current velocity at the opening of the new breakwater, the effect of land reclamation, frequency response of sea level oscillation and its non-linear effect	laminar except at opening of breakwater

This Channel connects Lake Nakaumi, of which the area is about 102 km<sup>2</sup> and the mean depth of water is 4.6 m, with Miho Bay facing Japan Sea. The purpose of this experiment is to find the character of frequency response of the water level in the channel after the construction of a gate at the entrance of the lake for the reclamation works. From this experiment, it was clarified that the characteristics of frequency response without a reservoir are considerably different from those with a reservoir, and in particular the 130 minutes oscillation in the channel becomes predominant. Besides, by the examination of the scale effect, a comparison between the amplitude ratios in the channel of the models of five distortion ratios, 2, 4, 6, 8 and 16 and of the prototype showed that the values in the model of distortion ratio 4 agree best with observed values. This means that the Manning roughness coefficient required in the model  $n_m = 0.0196$  (m-sec unit) is almost satisfied for a given coefficient in the prototype  $n_p = 0.022$ .

Nagoya Harbor - The purpose of this experiment is to investigate the change in the behavior of tidal currents by the construction of new breakwaters and the reclamation of new lands inside the breakwaters. The horizontal and vertical scales of the model were decided based on Eq. (3) by assuming that  $C_{r_p}$  is equal to  $5 \times 10^{-3}$  and using the observed maximum velocity of tidal current 20 cm/sec. The flow patterns inside and outside the breakwaters and the maximum current velocities at the main and sub-entrances of the harbor were observed for three widths of the openings in two different geographical configurations shown in Table 2.

From the experiments, it was found that the estimated maximum value of the tidal current at the opening based on the experimental results is in good agreement with the observed value; however, the problem in such a model experiment is the correction of the velocity resulting from the difference of discharge coefficients of the opening between the prototype and the model. A discussion of this problem will be given in the following section. With respect to the harbor oscillation, it was clarified that the experimental

Table 2. Conditions of experiments

Run	Width of opening		Condition of reclama- tion works	Surface area inside breakwater (km <sup>2</sup> )
	Main (m)	Sub (m)		
TCA	350	50	Semi-complete	71.16
TCB	400	200	"	"
TCC	500	300	"	"
TDA	350	50	Complete	50.27
TDC	500	300	"	"

values of response factor and phase lag coincide quantitatively with the theoretical ones derived by Love (1959).

#### COMPARISON OF TIDAL CURRENT BETWEEN PROTOTYPE AND MODEL

In beginning the model experiment, first of all, the reproductivity of the prototype in the model must be checked using the observed data of tidal current velocities. Fig. 3 shows the velocity pattern of the maximum rising current in the prototype of Nagoya Harbor observed at 3 m below the water surface in spring tide.

The flow pattern in the model corresponding to Fig. 3 is presented in Fig. 4 which was obtained by the preliminary experiment using a small model of the horizontal scale 1/2000 and the vertical scale 1/667. From the comparison between the two flow patterns, it seems that the reproductivity is fairly good even in the small model.

The observed data of the maximum current velocities at the main- and sub-entrances of the harbor are plotted against the tidal range in Fig. 5. The full lines and broken lines in the figure represent the theoretical relationship based on the following formula derived under the assumption of sinusoidal variation of the current velocity:

$$U_{\max} = \frac{\overline{U_{\max}}}{C} = \frac{\pi}{2} \frac{SH}{CA(T/2)} \quad (7)$$

in which  $U_{\max}$  is the spatial maximum current velocity at the opening,  $\overline{U_{\max}}$  the spatial mean velocity at the cross section of the opening,  $C$  the discharge coefficient of the opening,  $H$  the tidal range,  $S$  the surface area of the basin inside the harbor and  $A$  the cross sectional area at the opening.

From this figure, it is pointed out that the value of the discharge coefficient in the model will be 0.6 to 0.7. This estimation was confirmed by measuring the discharge coefficient of the opening of the breakwater directly in steady flow. Fig. 6 shows the relationship between the discharge

coefficient and the Reynolds number with a parameter of the water depth. It is found from the figure that the value of the discharge coefficient is about 0.65 in maximum in the range of the experiment and increases with an increase in the Reynolds number, which means that the value in the prototype will be greater than 0.65. This tendency was further verified with the detailed experiment by Higuchi (1966).

In estimating the maximum current velocity at the opening in the prototype based on the experimental data, it is necessary to know the value of the discharge coefficient in the prototype. Fig. 7 shows the observed data of the spatial maximum current velocity at the openings of the breakwater in the prototype compared with the computed values by Eq. (7). It is obviously pointed out from the figure that the value of the discharge coefficient in the prototype will be approximately from 0.8 to 1.0 and in average 0.9. This fact agrees well with that the discharge coefficient increases with an increase in the Reynolds number, as described previously. Thus it becomes possible to estimate the maximum current velocity in the prototype through the experimental data by using the discharge coefficients in both the prototype and the model.

#### EXPERIMENTS BY THE HIGH SPEED WIND WAVE CHANNEL

##### EXPERIMENTAL FACILITIES

The wind wave channel consists of three parts: the first is a wind tunnel to generate wind with a blower of 100 HP, the second a smoke tunnel to study the wind resistance of structures and the third a wind wave tank to study hydrodynamical characters of wind wave and mechanism of wave overtopping on seawalls as shown in Fig. 8. The downstream end of the wind tunnel is connected with a wave tank 40 m long, 2.3 m to 4.0 m high and 0.8 m wide. The maximum wind speed is 35 m/sec at the entrance of the wave tank with a section of 0.8 x 0.8 m. A wave generator of piston type is set near the wave tank and connected with the bottom of the wave tank, which is driven by a motor of 10 HP, so that waves can be generated without operating the blower. The period  $T$  and height  $H$  of waves generated by the wave generator are variable from 0.75 sec to 3.0 sec and up to 28 cm when  $T = 2.3$  sec respectively. At the end of the wave tank, a uniform model beach of 1 on 15 slope is set, which is made of wood.

##### WAVE OVERTOPPING ON VERTICAL SEAWALLS IN CALM CONDITION

Some experiments on wave overtopping on vertical seawalls in calm conditions were firstly carried out by Saville and Caldwell (1953), and consequently by Saville (1955) and Sibal (1955) for seawalls with various slopes and shapes. In Japan, Ishihara, Iwagaki and Suzuki (1955) asserted that a design method considering the allowable quantity of wave overtopping to some extent, should be used for practical purpose, and then Ishihara, Iwagaki and Mitsui (1960) performed systematically some basic experiments for vertical and inclined seawalls, and proposed a dimensionless expression for the rate of wave overtopping to that of water moving on shore per wave period in deep water. Moreover, they plotted the experimental results obtained by themselves and Saville, based on the above expression, and found

the relationships between the characteristics of incident waves, the water depth at the toe of a seawall, the height of the seawall from still water level and the rate of wave overtopping. Recently, Iwagaki, Shima and Inoue (1965) discussed the effects of wave characteristics, water depth and water level on the quantity of wave overtopping on vertical seawalls based on their experimental results, in addition to those by the Beach Erosion Board. The experiments by Ishihara and others were carried out only in the case of incident wave steepnesses of 0.03 to 0.08, but the experiments for the wave steepnesses of less than 0.03 have not been carried out yet. For this reason, the experiments of wave overtopping on vertical seawalls were firstly made in the case of incident wave steepnesses of 0.01, 0.02 and 0.03 and calm conditions using the high speed wind wave channel before investigating the effect of wind.

The model of a vertical seawall made of steel plate was set on a beach of 1/15 slope. With regard to the width of the model seawall, to measure the incident wave height as exactly as possible, the width of wave tank, 80 cm, was divided into two parts only near the seawall model, 30 cm and 50 cm, and the model was set in the part of 30 cm. The incident wave height was measured in the part of 50 cm where the waves are not reflected by the seawall. The results of experiments are presented in Fig. 9 for the case of the wave steepness 0.02, in which  $Q$  is the quantity of wave overtopping per wave period,  $H_0$  the wave height,  $L_0$  the wave length,  $H_c$  the crest height of a seawall above the still water level, and  $h$  the water depth at the toe of the seawall. This figure shows that the quantity of wave overtopping becomes maximum when a seawall is constructed at the place where incident waves break just in front of the seawall.

#### EFFECT OF WIND ON WAVE OVERTOPPING

Wave Overtopping on Vertical Seawalls - In estimating the quantity of wave overtopping on seawalls, the effect of wind on it may not be ignored because in most cases wave overtopping becomes a problem in the event of a strong wind. Sibul and Tickner (1956) carried out model experiments for sea dikes with slopes of 1/3 and 1/6 put on a model beach with a slope of 1/10 to find the additional quantity of wave overtopping due to the action of wind, compared with conditions in calm weather. Paape (1961) made clear, by using a wind wave tunnel, that the irregularity of incident wind waves increases the quantity of wave overtopping considerably, compared with the results in the case of regular waves. However, the influence of wind on wave overtopping has not yet been made clear quantitatively. For this reason, a basic study of wave overtopping to find the influence of wind has been begun by Iwagaki and others and some results have been presented already by Iwagaki, Tsuchiya and Inoue (1965), Iwagaki, Inoue and Ohori (1966) and Iwagaki, Tsuchiya and Inoue (1966).

Firstly the quantity of wave overtopping was measured in calm conditions and secondly, the same measurements were undertaken in the case of wind. The wave steepnesses used in the experiments were 0.01 and 0.02.

In plotting the experimental data, the following expression was derived by means of the dimensional analysis for a vertical, smooth seawall concerned with the phenomenon of wave overtopping with wind, if the effect of viscosity

of water is neglected:

$$2\pi Q/H_0 L_0 = f ( H_0/L_0, H_c/H_0, h/L_0, V/\sqrt{g H_0} ) \quad (8)$$

in which  $V$  is the wind velocity,  $g$  the acceleration of gravity and  $V/\sqrt{g H_0}$  the dimensionless wind velocity.

Fig. 10 is a plot of the experimental data to show the effect of wind on wave overtopping on a vertical seawall in the case of the wave steepness 0.02.

The experimental results reduce the following conclusions:

- (1) When incident waves do not break in front of the seawall, because the water depth at the toe of the seawall is large compared with the incident wave height, the quantity of wave overtopping begins to increase suddenly with an increase in the wind velocity at a certain wind velocity ( see the case  $h/L_0 = 0.03$  and  $H_c/H_0 = 2.08$  in Fig. 10 ),
- (2) When incident waves break just in front of the seawall, the quantity of wave overtopping shows a complicated change with an increase in the wind velocity for the wave steepness of 0.01, and little change over a low wind velocity for the wave steepness of 0.02 (see the case  $h/L_0 = 0.02$  and  $H_c/H_0 = 2.00$  in Fig. 10). But the additional quantity of wave overtopping due to wind action is generally small,
- (3) When incident waves break before they reach the seawall, the effect of wind on wave overtopping is not remarkable quantitatively, and when the seawall is constructed at the shoreline or on shore, the quantity of wave overtopping rather decreases a little at a high wind velocity (see the cases  $h/L_0 = 0.01$ ,  $H_c/H_0 = 1.48$  and  $h/L_0 = 0$ ,  $H_c/H_0 = 0.56$  in Fig. 10).

Wave Overtopping on Model Seawalls - The model experiments of wave overtopping on the seawalls at Sakai Harbor and Yui Coast were carried out by Iwagaki, Tsuchiya and Inoue (1964) in a scale of 1/15.

Fig. 11 shows a cross section of the seawall at Sakai Harbor, in which the design deep-water wave is 2 to 3 m in height and 6.5 sec in period, and the water depth at the toe of the seawall is 13.3 m at the design sea level of O.P. + 4.30 m; that is, the case when the water depth at the toe of the seawall is very large compared with the incident wave height. Firstly the experiments were carried out in calm conditions and it was found that the rate of wave overtopping increases with an increase in the wave height. Secondly the effect of wind on wave overtopping was investigated by using the high speed wind wave channel. Fig. 12 represents the experimental results, which shows that the quantity of wave overtopping increases gradually with an increase in the wind velocity until the value of  $V/\sqrt{g H_0}$  reaches about 5 and after that it increases suddenly in the same manner as in the case of a vertical seawall.

Fig. 13 is a cross section of the seawall at Yui Coast, in which the design deep-water wave is 14.5 m in height and 18 sec in period, and the water depth at the toe of the seawall is 6.4 m at the design sea level of T.P. +1.66 m. According to the model experiments of wave overtopping in calm conditions for various deep-water wave heights by Iwagaki, Tsuchiya and Inoue

(1963) in a scale of  $1/25$ , it was found that the maximum rate of wave overtopping appeared at the deep-water wave height of about 6 m, which was the case when incident waves break just in front of the seawall. The effect of wind on wave overtopping for a constant wave height is shown in Fig. 14. In this figure, it can be seen that there is much difference in the effect of wind between the cases when incident waves reach the seawall after breaking ( $H_0/L_0 > 0.0135$  in Fig. 14) and without breaking ( $H_0/L_0 < 0.0128$  in Fig. 14). It should be noted, therefore, that the effect of wind on wave overtopping is varied with the characteristics of incident waves.

#### CONCLUSIONS

The authors described some results of the hydraulic model experiments, which are of tidal currents and wave overtopping on seawalls, by using the estuary model basin and the high speed wind wave channel respectively, and discussed the problems to which attentions should be paid from view point of similitude between the model and the prototype, in particular, taking an example of the experiment of tidal current at Nagoya Harbor. In addition, based on the experimental data obtained by the wind wave channel, it was shown that the effect of wind on wave overtopping is quite different due to the characteristics of incident waves, and it cannot be ignored in the case when the water depth at the toe of a seawall is large compared with the incident wave height.

#### ACKNOWLEDGEMENTS

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#### REFERENCES

- Hayami, S., Higuchi, H. and Yoshida, K. H. (1958). On the Similitude of Hydraulic Models Involving Tidal Motion: Disaster Prevention Research Institute, Kyoto University, Annuals No. 2, pp. 83-95 (in Japanese).
- Higuchi, H. (1961). Hydraulic Model Experiment on the Oscillation of Water Level in Sakai Channel: Coastal Engineering in Japan, Vol. 4, pp. 35-45.
- Higuchi, H. and Yoshida, K. H. (1964). Hydraulic Model Experiments on Tidal Current At Nagoya Harbor: Coastal Engineering in Japan, Vol. 7, pp. 133-147.
- Higuchi, H. (1964). On the Characteristic of the Oscillation of Water Level in Nagoya Harbor (Continued): Disaster Prevention Research Institute, Kyoto University, Annuals No. 7, pp. 400-409 (in Japanese).
- Higuchi, H. (1966). On the Similitude at the Opening of Breakwater in Hydraulic Model Experiment Involving Tidal Current: Disaster Prevention Research Institute, Kyoto University, Annuals No. 9, pp. 763-770 (in Japanese).

- Ishihara, T., Iwagaki, Y. and Suzuki, Y. (1955). Design of Seawalls --- Especially on the Effective Height ---: Proc. of 2nd Conf. on Coastal Engineering in Japan, pp. 57-70 (in Japanese).
- Ishihara, T., Iwagaki, Y. and Mitsui, H. (1960). Wave Overtopping on Seawalls: Coastal Engineering in Japan, Vol. 3, pp.53-62.
- Iwagaki, Y., Tsuchiya, Y. and Inoue, M. (1963). Model Experiments on the Seawall at Yui Coast : Disaster Prevention Research Institute, Kyoto University, Annuals No. 6, pp.328-337 (in Japanese).
- Iwagaki, Y., Tsuchiya, Y. and Inoue, M. (1964). Some Problems on Prevention of Wave Overtopping on Seawalls and Seadikes: Disaster Prevention Research Institute, Kyoto University, Annuals No. 7, pp. 387-399 (in Japanese).
- Iwagaki, Y., Tsuchiya, Y. and Inoue, M. (1965). Studies on the Effect of Wind on Wave Overtopping on Seawalls (First Report): Disaster Prevention Research Institute, Kyoto University, Annuals No. 8, pp. 397-406 (in Japanese).
- Iwagaki, Y., Shima, A. and Inoue, M. (1965). Effects of Wave Height and Sea Water Level on Wave Overtopping and Wave Run-up: Coastal Engineering in Japan, Vol. 8, pp. 141-151.
- Iwagaki, Y., Inoue, M. and Ohori, K. (1966). Studies on the Effect of Wind on Wave Overtopping on Seawalls (Second Report): Disaster Prevention Research Institute, Kyoto University, Annuals No. 9, pp. 715-727 (in Japanese).
- Iwagaki, Y., Tsuchiya, Y. and Inoue, M. (1966). On the Effect of Wind on Wave Overtopping on Vertical Seawalls: Bulletin of the Disaster Prevention Research Institute, Kyoto University, Vol. 16, Part 1, No. 105, pp. 11-30.
- Love, R. W. (1959). Tidal Response of a Bay with a Constricted Opening to the Sea: Thesis for M. S., A and M College of Texas, Dept. of Oceanography, pp. 1-66.
- Paape, A. (1961). Experimental Data on the Overtopping of Seawalls by Waves: Proc. of 7th Conf. on Coastal Engineering, Vol. 2, pp. 674-681.
- Saville, T. Jr. and Caldwell, J. M. (1953). Experimental Study of Wave Overtopping on Shore Structures: Proc. of Minnesota International Hydraulics Convention, pp. 261-269.
- Saville, T. Jr. (1955). Laboratory Data on Wave Run-up and Overtopping on Shore Structures: Beach Erosion Board, Tech. Memo., No. 64, pp. 1-32.
- Sibul, O. J. (1955). Flow over Reefs and Structures by Wave Action: Trans. AGU, Vol. 36, No. 1, pp. 61-71.
- Sibul, O. J. and Tickner, E. G. (1956). Model Study of Overtopping of Wind Generated Waves on Levees with Slopes 1 : 3 and 1 : 6: Beach Erosion Board, Tech. Memo., No. 81, pp. 1-27.

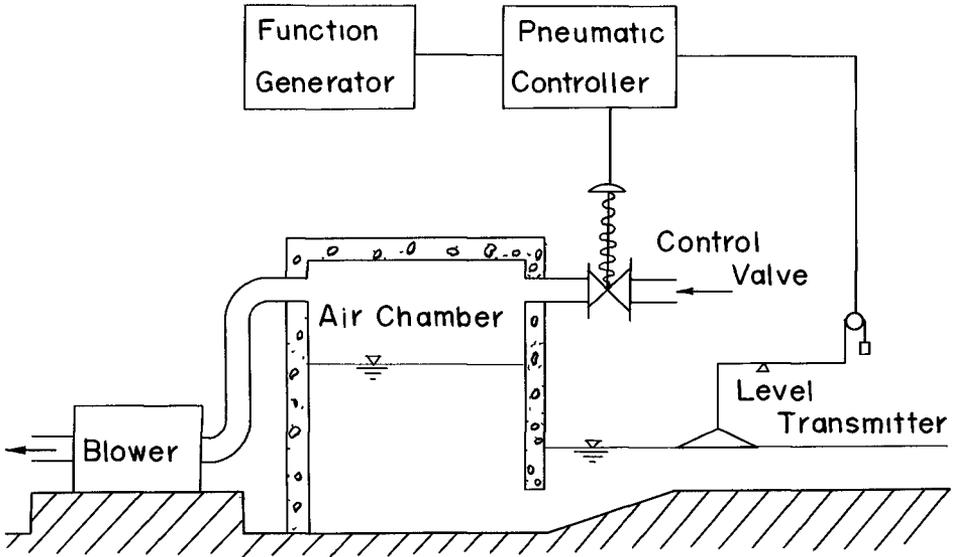


Fig. 1. Pneumatic tide generator.

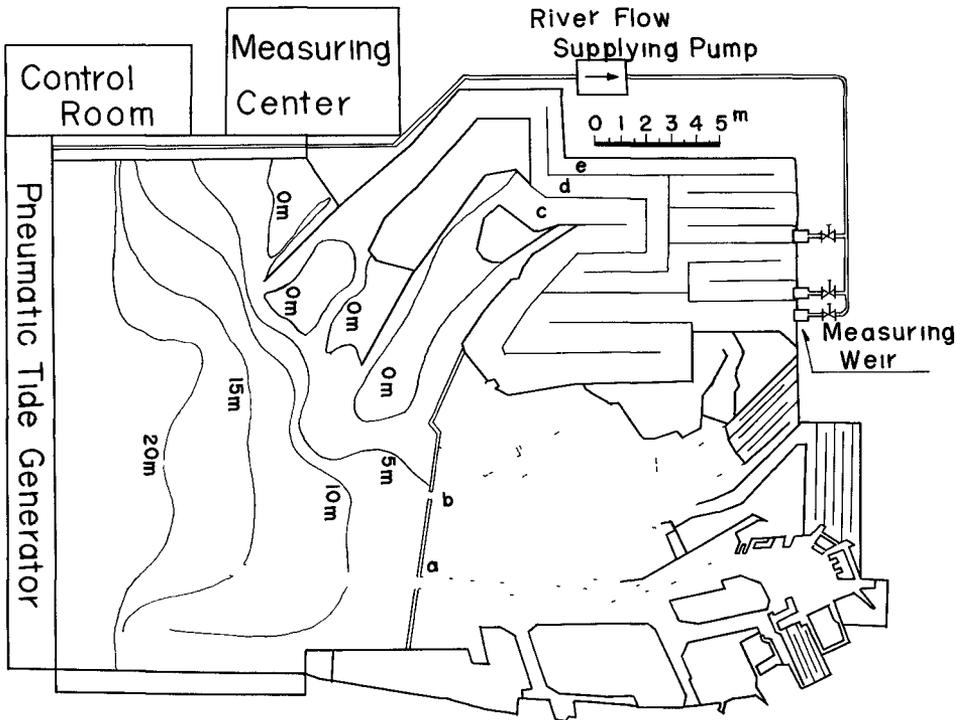


Fig. 2. Layout of Nagoya Harbor model.  
 a. Main-entrance b. Sub-entrance  
 c. Kiso River d. Nagara River  
 e. Ibi River

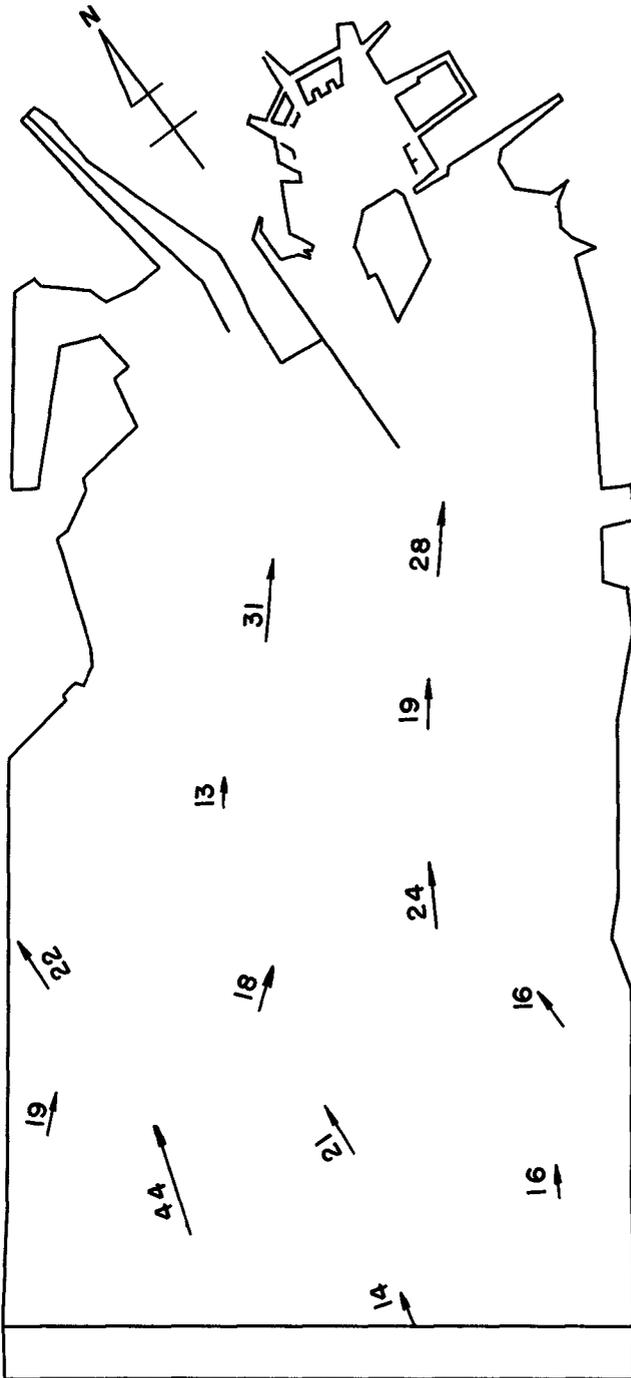


Fig. 3. Velocity pattern of maximum rising current in prototype in cm/sec. (Spring tide and 3 m below water surface)

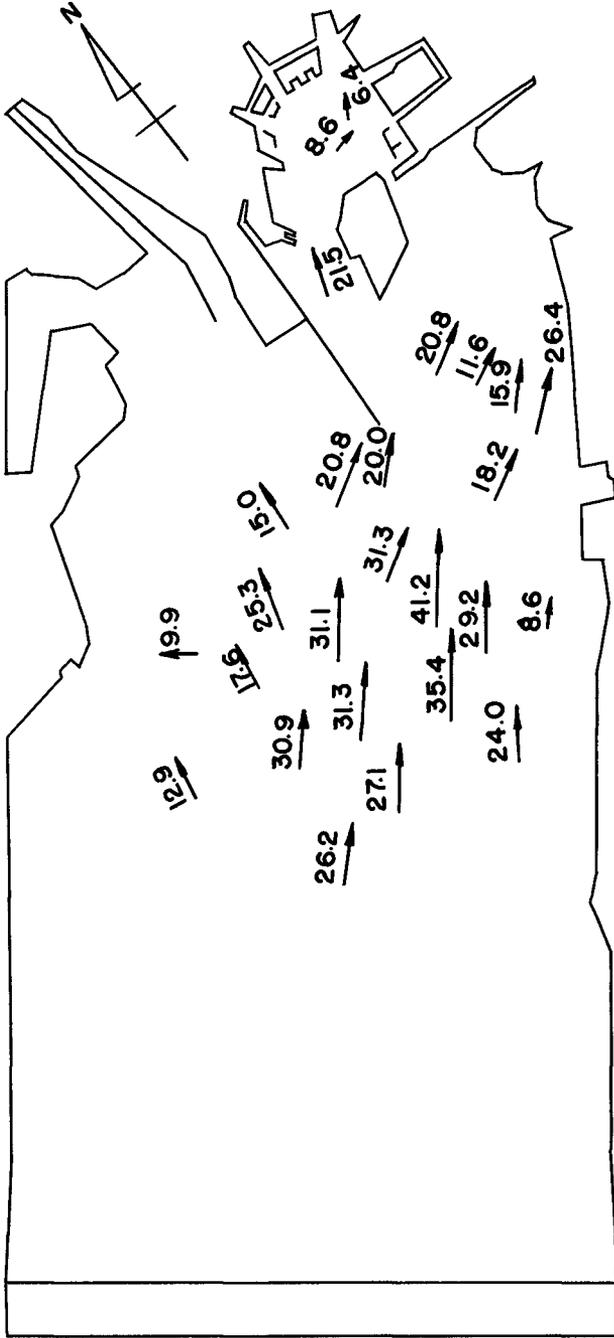


Fig. 4. Velocity pattern of maximum rising current in model in cm/sec. (Tidal range: 2.6 m)

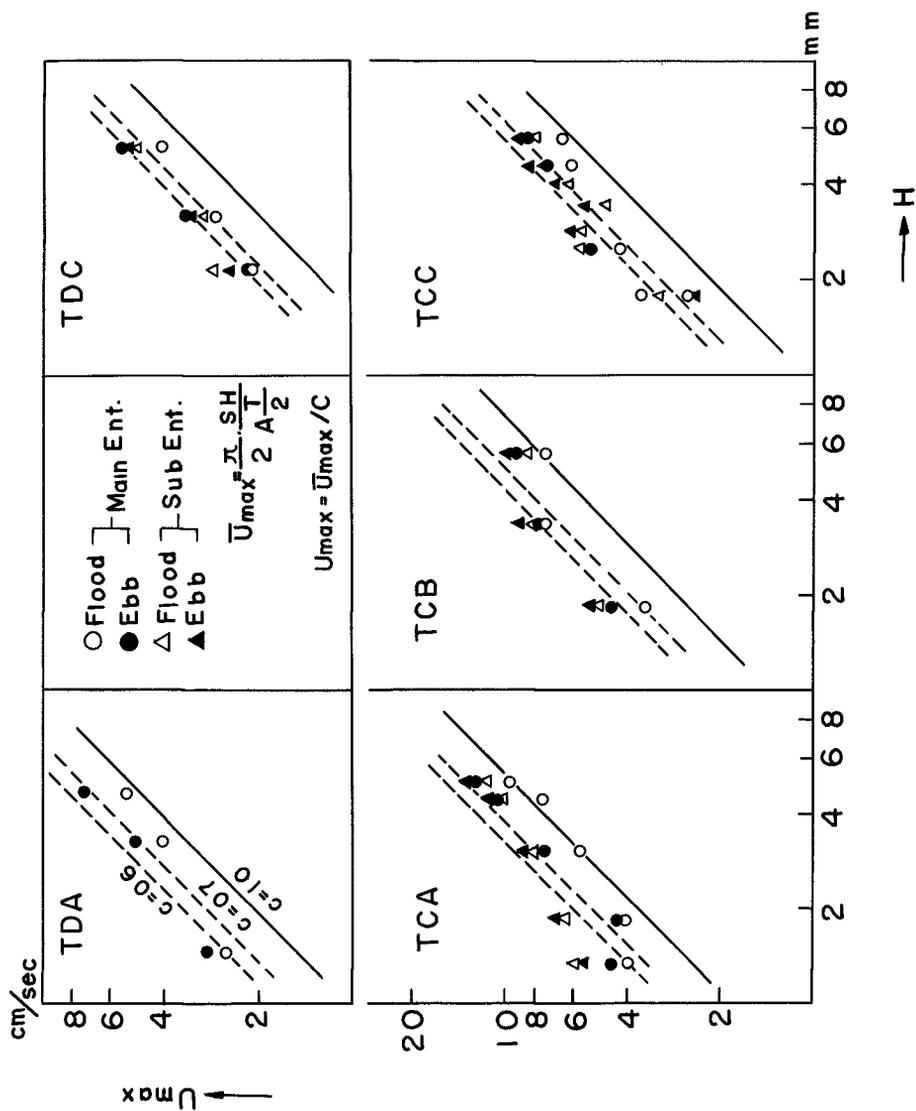


Fig. 5. Maximum current velocity at openings against tidal range in model.

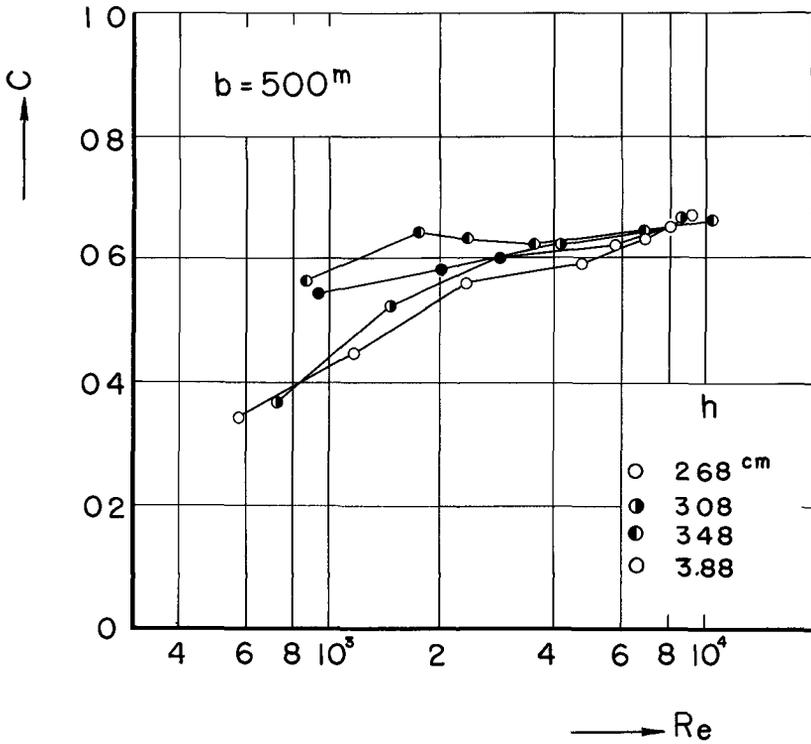


Fig. 6. Discharge coefficient of opening against Reynolds number in model.

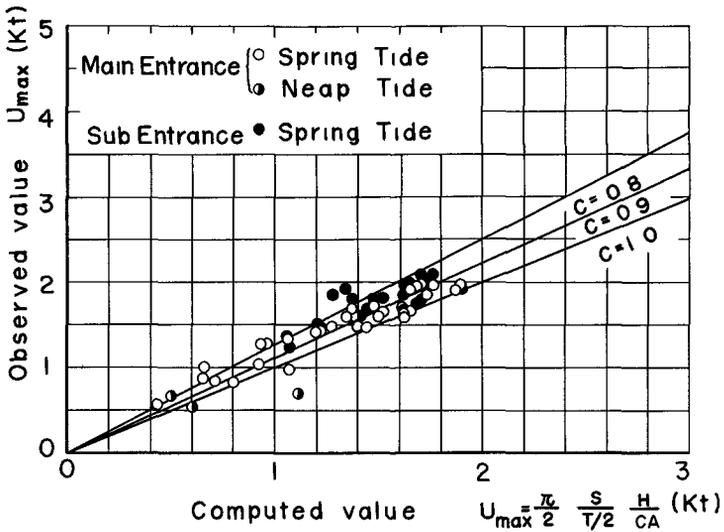


Fig. 7. Maximum current velocity at openings in prototype observed by Ministry of Transportation.

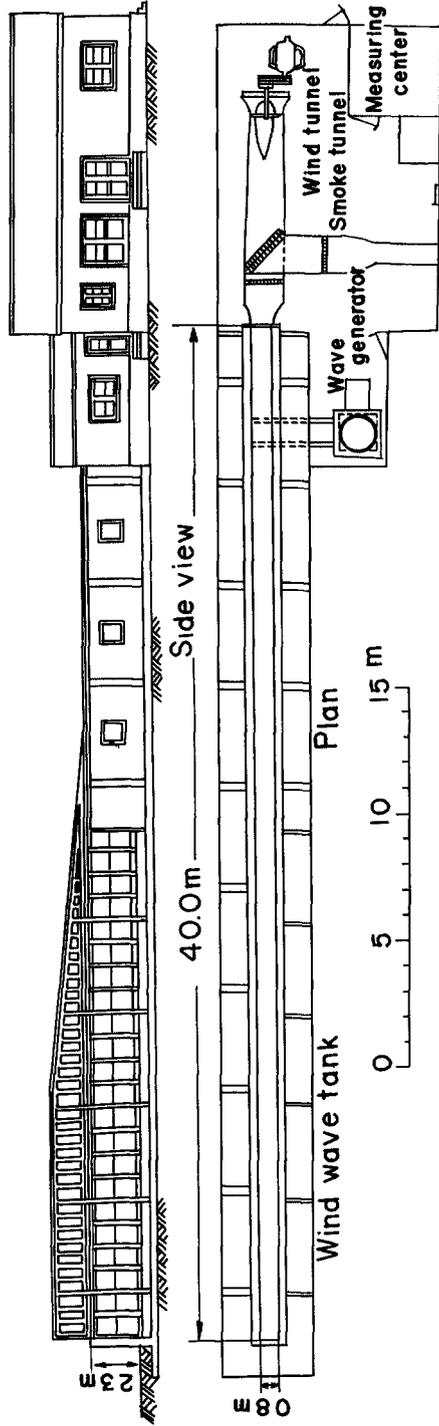


Fig. 8. Sketch of high speed wind wave channel.

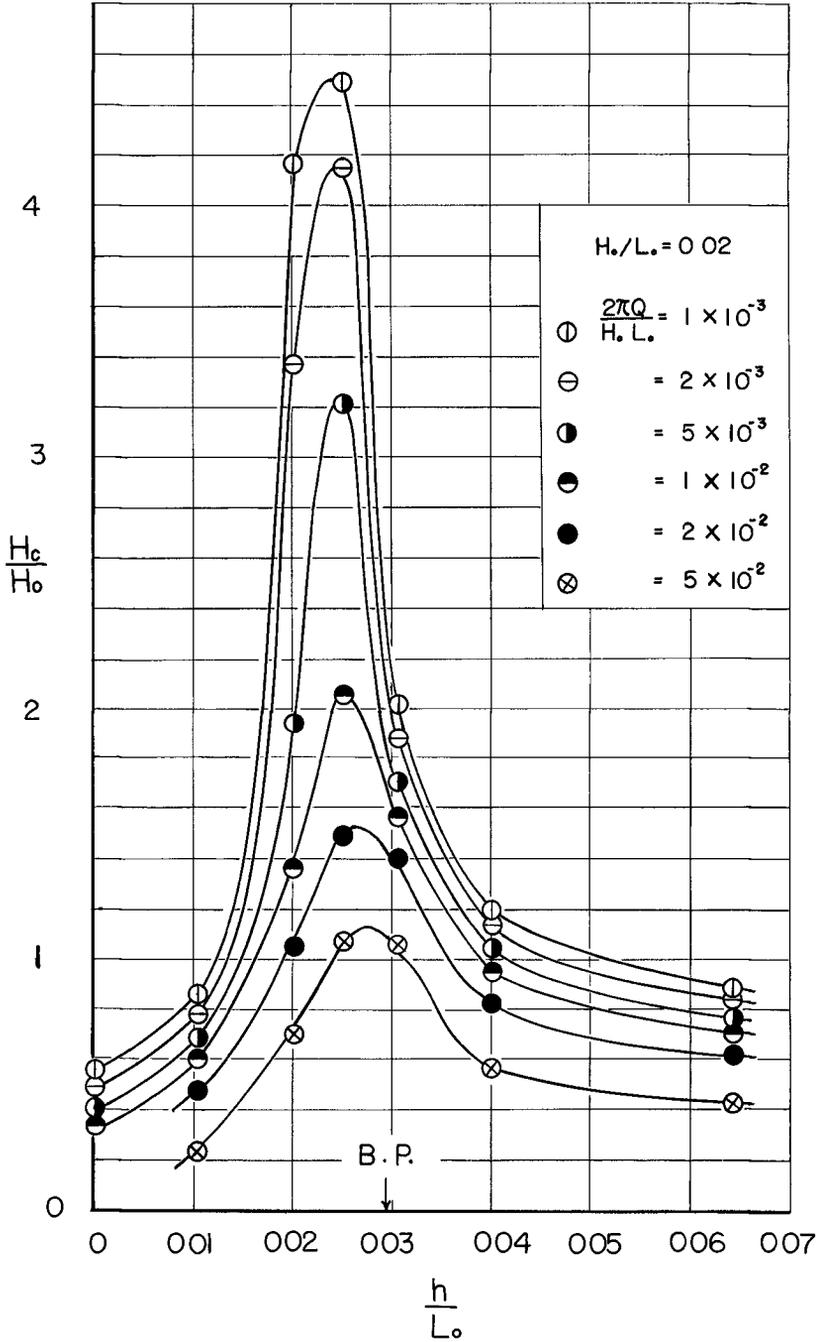


Fig. 9. Dimensionless plots of rate of wave overtopping on vertical seawalls in calm condition when wave steepness is equal to 0.02.

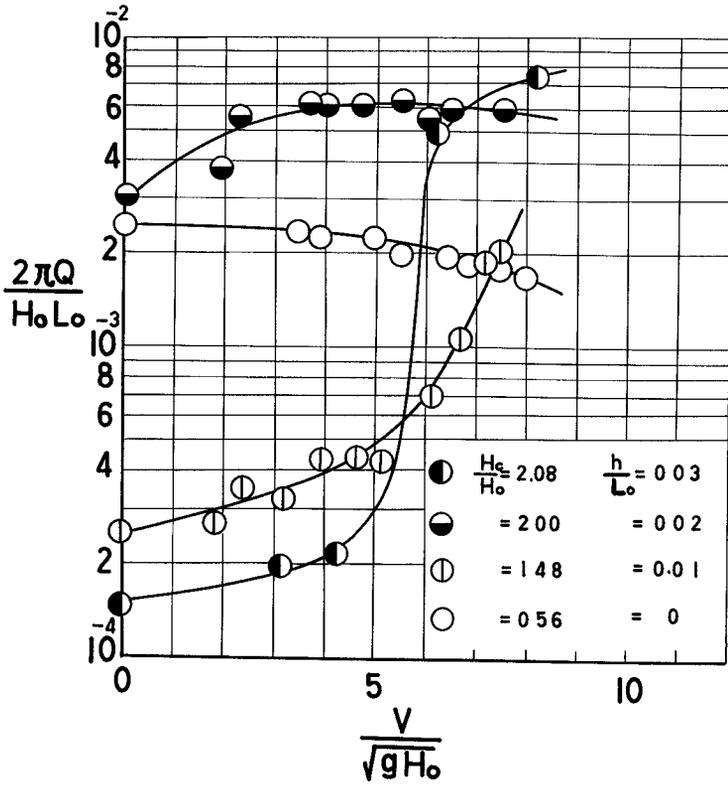


Fig. 10. Effect of wind on wave overtopping on vertical seawalls when wave steepness is equal to 0.02.

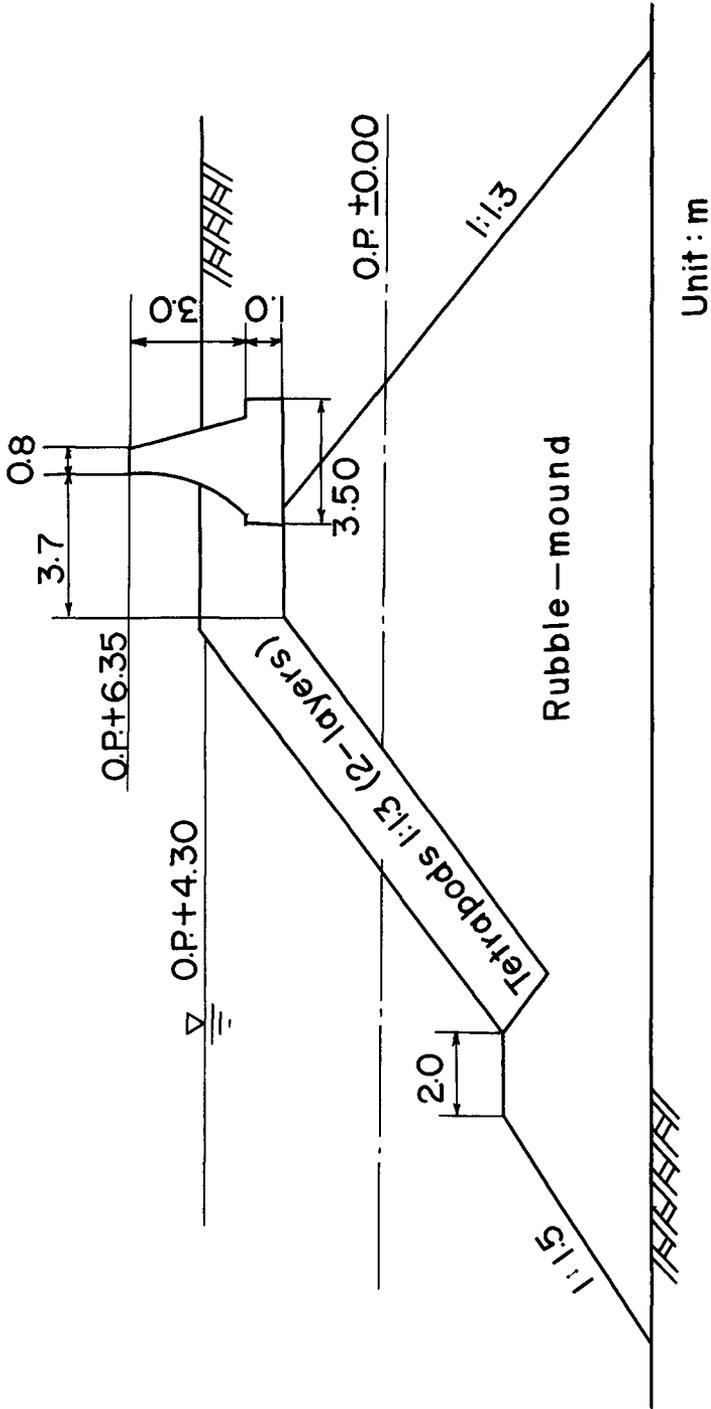


Fig. 11. Cross section of seawall at Sakai Harbor.

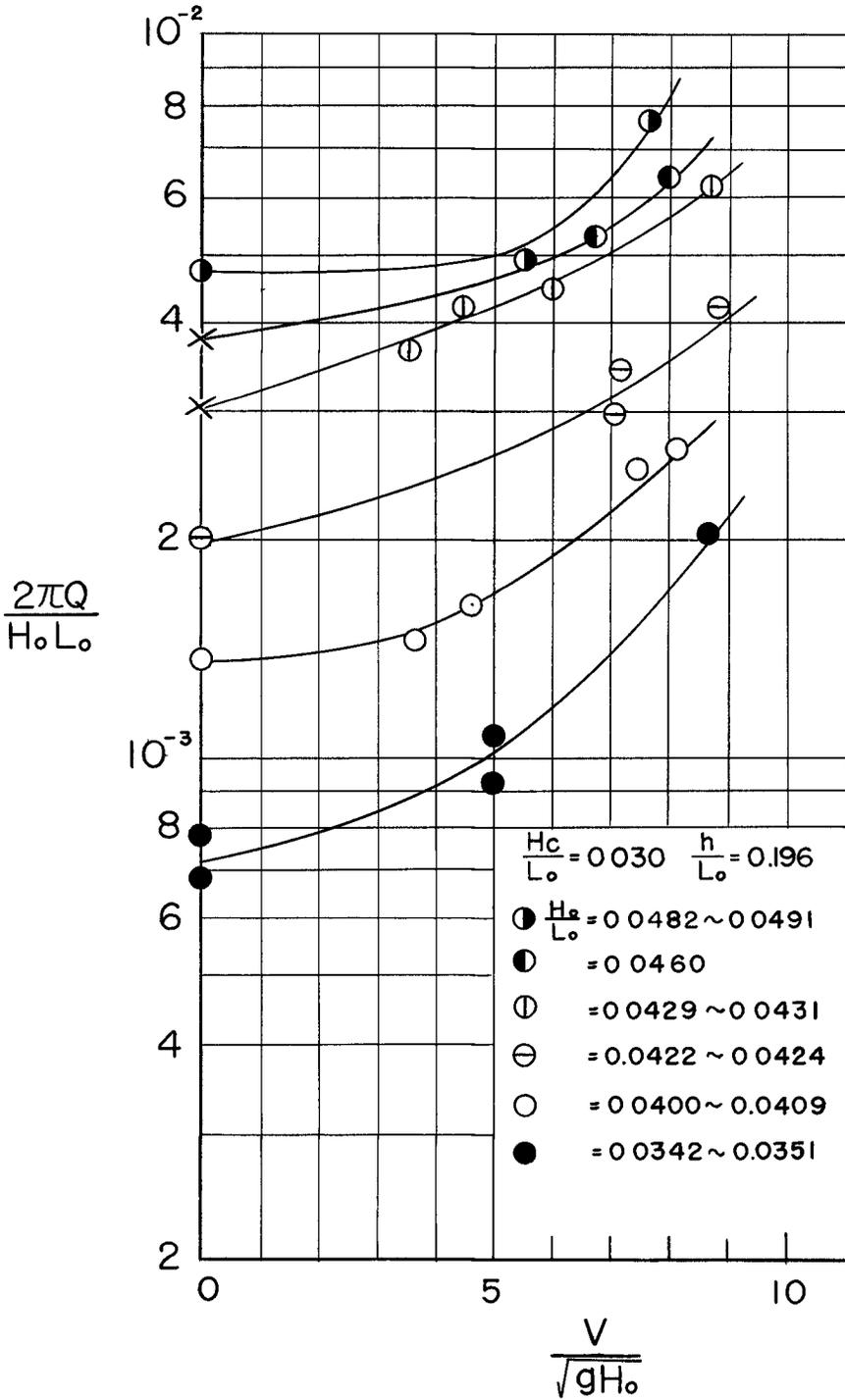


Fig. 12. Effect of wind on wave overtopping for seawall at Sakai Harbor.

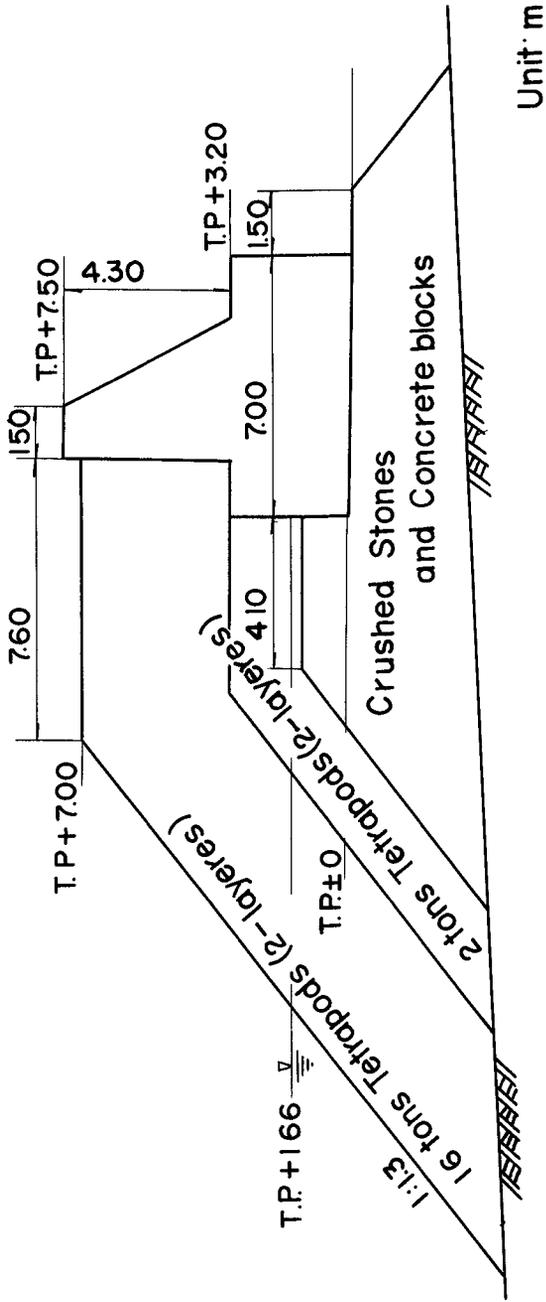


Fig. 13. Cross section of seawall at Yun Coast.

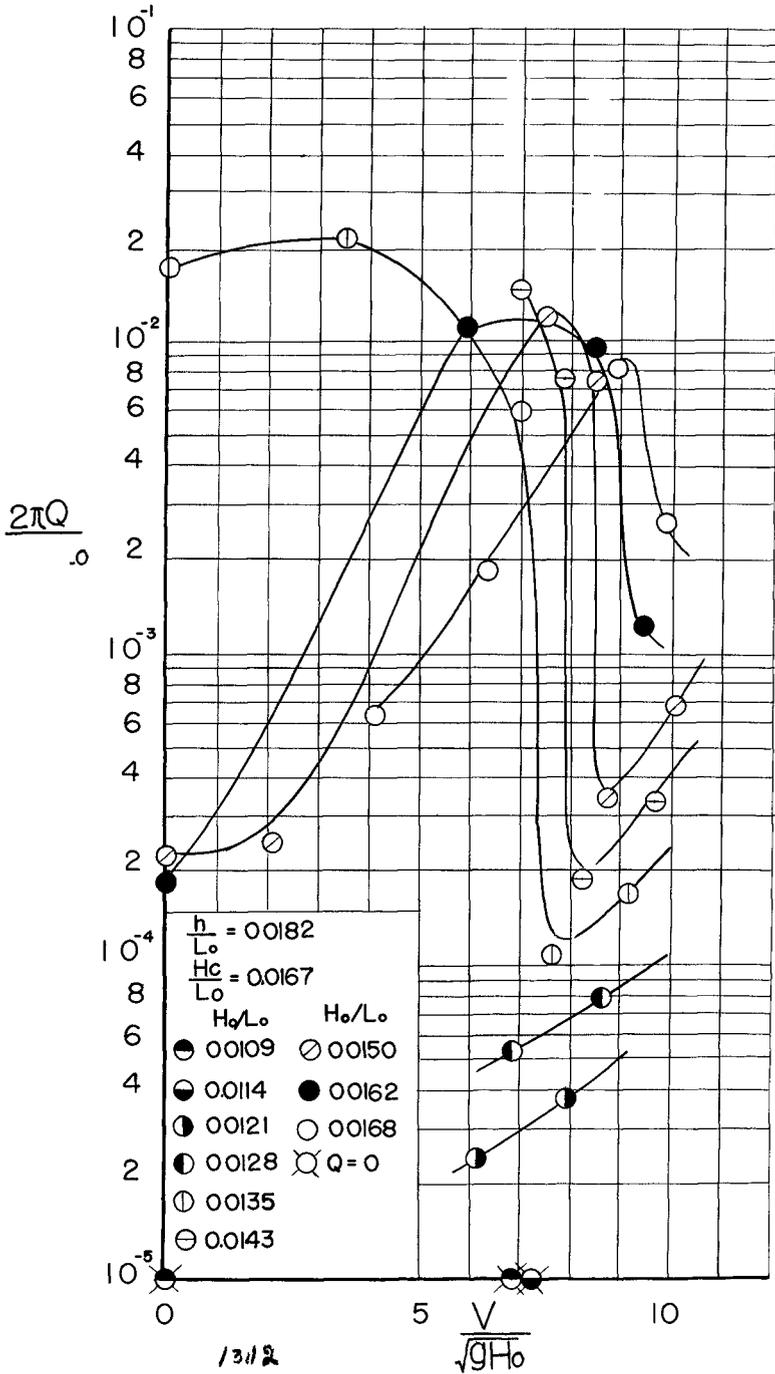


Fig. 14. Effect of wind on wave overtopping for seawall at Yui Coast.