CHAPTER 54

EXPECTED DISCHARGE OF IRREGULAR WAVE OVERTOPPING

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ABSTRACT

An experiment was carried out on the overtopping of mechanically generated irregular waves over vertical walls. The experimental discharge was almost in agreement with the expected discharge which had been calculated with the wave height histogram and the data of regular wave overtopping based on the principle of linear summation. The expected values of overtopping discharge were calculated for various laboratory data, which had been represented in a unified form of non-dimensional quantities. The calculation has yielded two diagrams of expected overtopping discharge, one for the sea wall of vertical wall type and the other for the sea wall covered with artificial concrete blocks.

INTRODUCTION

The crest height of a sea wall is the critical factor in its planning. With the heightening of the sea wall the degree of protection which it affords increases, but the construction cost also increases. The selection of the sea wall height based on the minimum sum of expected damage and construction cost has been studied and applied to many coastal sites along Japan (Tsuruta et. al. 1964).

The selection of the sea wall height is also made from its hydraulic performance against the design waves. The arguments on the criterion of selection, however, are divided between the maximum run-up height and the allowable quantity of wave overtopping. If a sea wall can be built higher than the maximum run-up of design waves, the sea wall will afford complete protection against the wave action. Such sea wall must be extremely high, however. Even for a regular train of waves, laboratory data indicate the run-up height being from 1.2 H to more than 4 H. For an irregular train of actual waves, the maximum run-up in that wave train will be $2H_{1/3}$ to more than 10 $H_{1/3}$, since the maximum run-up will be realized by the maximum or near-maximum wave, the height of which may exceed twice the significant wave height. With the design wave height of $H_{1/3}$= 4 to 8 m, which is common along the coasts of Japan, the maximum run-up becomes about 10 to 30 m or higher above the design tide level. In addition, even with such an extreme height the sea wall will not guarantee the perfect protection, because the design wave and tide conditions are not the worst condition possible but only represents an extremely severe condition; we cannot neglect the possibility that the waves and tide surpassing the design conditions may attack the sea wall.

The criterion of allowable discharge of wave overtopping, therefore, becomes the inevitable choice for most case, even though the concept of allowing certain amount of wave overtopping may sound disgraceful for coastal engineers and threatening for people. But the sea wall with appropriate facilities to expel the overtopped water protects well the area behind it even under the attack of the waves and tides surpassing the design condition. The important factor in the planning of a sea wall allowing certain overtopping is the correct estimation of overtopping discharge. This must be done not for regular trains of laboratory wave but for irregular trains of actual waves.

The discharge of overtopping of irregular waves differs from that of regular waves. The difference mainly comes from two sources: the wave height variability and the interference by preceding waves. Some of previous experimental works on irregular wave overtopping have attributed the difference to the interference effect (Paape 1960), or to the general effect of wind including the generation of wave height irregularity (Sibul and Tickner 1956). Although these effects are certainly important, the variability in wave heights distributed from 0 to H_{max} must be the dominating factor, for even the wave interference occurs owning to the presence of wave irregularity. If the effect of wave interference on overtopping is not significant, the overtopping quantity of irregular waves can be estimated as the summation of individual wave overtoppings, the quantities of which are to be determined from the data of regular waves. In order to investigate the applicability of the principle of linear summation for the overtopping of irregular waves, an experiment has been carried out with mechanically generated irregular waves. Supported by the experimental result, the estimation of the expected discharge of irregular wave overtopping has been made for the sea wall of vertical wall type and that covered with artificial concrete blocks as will be described in the following chapters.

EXPERIMENT ON IRREGULAR WAVE OVERTOPPING

WAVE CHANNEL AND IRREGULAR WAVES FOR THE EXPERIMENT

The experiment was carried out in a wave channel of 30 m long, 1.2 m high and 0.5 m wide. The channel was divided into two with a partition wall along the center line which began at 2.5 m away from the wave paddle of piston type. A vertical wall was set in one channel at the distance of 15.5 m from the wave paddle on the slope of 1 to 20, the toe of which was located at 10.5 m from the wave paddle. The water depth was kept at h = 35 cm in front of the vertical wall.

Irregular waves were generated by an oil-pressure pulse motor which is controlled by ten electric oscillators (Tsuruta 1966).

The oscillators with the periods of 0.50, 0.65, 0.83, 1.07, 1.38, 1.77, 2.30, 2.88, 3.84, and 5.00 seconds generate sine waves, the amplitudes of which can be varied with the setting of power adjustment dials. In addition, another power controle dial is provided for the over-all adjustment of composed electric oscillation. By proper combination of ten oscillator amplitudes, power spectra of ocean waves can be simulated. Figure 1 exhibits the power spectrum of one train of test waves with the wave height of $H_{1/3}$ = 10.5 cm. Though the spectrum still exhibits the nature of line spectra, it nevertheless resembles to the spectrum of ocean waves. In the experiment, the dials for the oscillator amplitudes were kept at the pre-determined values and the over-all controls dial was set at one of five different values. By this procedure the magnitude of test waves was varied as shown in Table 1 with out significant change of its statistical property and of shape of power spectra; the ratio of $H_{1/3}/\bar{H}$ varied from 1.4 to 1.5 and $T_{1/3}/\bar{T}$ from 1.0 to 1.1.

Table 1. Characteristics of Irregular Waves Employed

Ident.	Number of waves N	Wave Height $\bar{H}$	$H_{1/3}$	Wave Period $\bar{T}$	$T_{1/3}$
1	188	5.3 cm	7.5 cm	1.43 sec	1.54 sec
2	180	7.0	10.5	1.51	1.56
3	188	8.4	12.6	1.51	1.60
4	174	10.0	14.8	1.60	1.81
5	170	11.4	16.4	1.58	1.77

The cumulative distribution of wave heights is shown in Fig. 2. The distribution of test waves is narrower than the Rayleigh distribution of

$$P(\eta) = 1 - \exp\left[-(\pi/4)\eta^2\right] \qquad (1)$$

where $\eta = H/\bar{H}$ and P denotes the probability of η lying between 0 and η. For the test waves, an empirical form of $\eta^{2.5}$ instead of η^2 gives a better fit.

The wave records analysed for Table 1 and Fig. 2 are not continuous ones but the accumulations of short runs. The use of short runs has been so required because the measurement of wave overtopping must be stopped before the return of the wave front re-reflected by the wave paddle. Though the very front of wave train travelled with the celerity of $\sqrt{gh}$, its amplitude decreased rapidly. Thus, the end of each measurement was determined as 23.5 seconds after the start of wave generator, based on the travel time of wave front with the phase velocity of waves with T = 1.6 seconds. Also, the initial parts of records before the arrival of main wave group which propagated with the group velocity were discarded, because they only contained the long period components of low amplitudes. Thus, the records from 10.3 till 23.5 seconds after the start of wave generator were utilized for the analysis of incident waves and of wave overtopping.

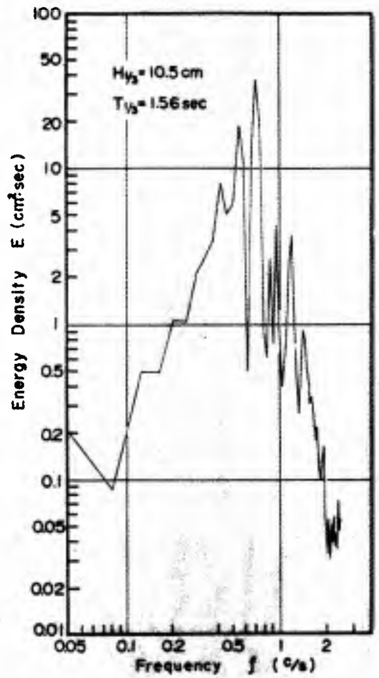

Fig. 1 Power spectrum of one train of test waves

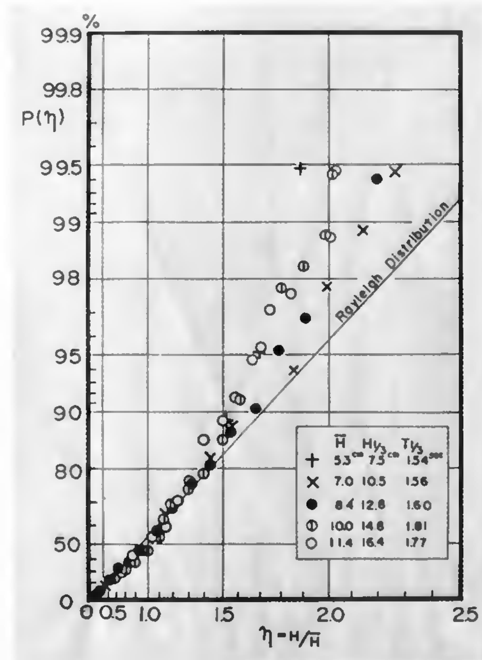

Fig. 2 Cumulative distribution of wave heights of test waves

The tests were repeated twice with and without the model sea wall: the former for the measurement of wave overtopping and the latter for the calibration of incident waves. The records of incident waves were separated into individual waves with the zero-up cross method. The wave height was defined as the distance from a crest to the following trough and the wave period as the time interval between the consecutive zero-up cross points. Each run gave the record of seven to ten waves, depending upon the periods of individual waves, and the test was repeated for twenty runs so that about 180 waves in total could be obtained for both the incident waves and the wave overtopping.

The incident waves was measured with a resistance-wire type wave gauge stationed at the location where the model sea wall was to be set. Another gauge was stationed at the same location in the opposite channel. At the time of overtopping measurement, one gauge was placed at 1 cm away from the vertical wall and the other gauge at the opposite channel. The latter gauge served for the confirmation of the wave overtopping the vertical wall being statistically same with the incident waves which had been analysed before the set of the vertical wall.

MEASUREMENTS OF OVERTOPPING DISCHARGE

The overtopped water was lead into a bucket which was hanged behind a model sea wall as shown in Fig. 3. By means of a movable carriage, the bucket was put into position at the arrival of main wave group and taken away before the return of re-reflected waves. The weight of the bucket with the overtopped water was continuously recorded through a load cell of strain gauge type which was inserted between the bucket and the carriage. When a wave overtopped the vertical wall, the record indicated an increase of the bucket weight about 0.5 second later, which was the time for the water to flow over the deck of model sea wall, as shown in Fig. 4. The difference of weight before and after one wave overtopping gave the amount of the overtopped water produced by that wave.

At the end of a run, the total amount of the overtopped water was measured and divided by the time of measurement period in seconds and the width of the sea wall so as to yield the average discharge of wave overtopping per second per unit width of the sea wall. For regular waves, five runs were repeated and the overtopping discharges of these runs were averaged. For irregular waves, the overtopping amounts of twenty runs were added together and the sum was divided by the total time of measurement periods and the width of the sea wall.

OVERTOPPING DISCHARGE OF REGULAR AND IRREGULAR WAVES

Figure 5 shows the overtopping discharges of regular waves, q (cm3/cm·sec), against the incident wave height, H (cm). Two wave periods of $T = 1.38$ and 1.77 seconds were applied for the vertical walls of 9.4 and 12.8 cm high above the still water level. Since the height of waves with the period of 1.38 seconds was limited to about 10 cm by the generator performance, the waves with the period

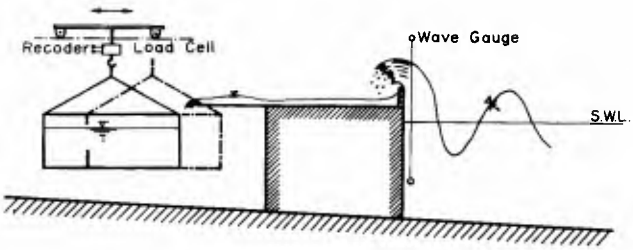

Fig. 3 Experimental set-up for the measurement of wave overtopping

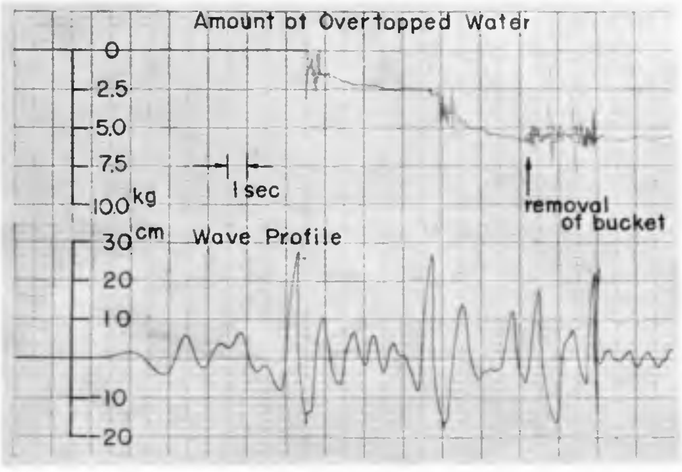

Fig. 4 Sample record of wave overtopping and wave profile

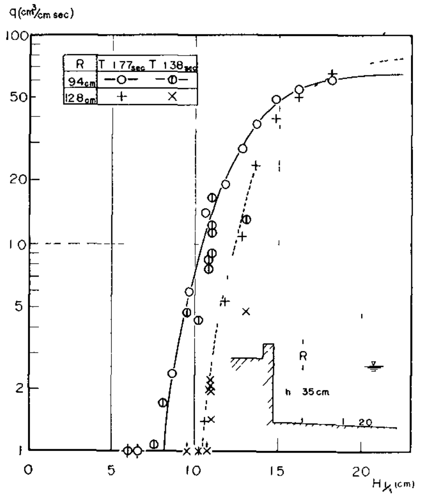

Fig. 5 Overtopping discharge of regular waves on vertical walls

of 1.77 seconds provided the principal source of overtopping data. The overtopping discharge of T = 1.38 seconds was slightly smaller than that of T = 1.77 seconds, but the difference was small. The vertical wall with the crest height of R = 9.4 cm generally produced larger wave overtopping as would be expected. At large wave height, a part of waves having overtopped the vertical wall of R = 9.4 cm flowed back toward the offshore because of the small difference between the top of the wall and the deck; this caused smaller overtopping for R = 9.4 cm than for R = 12.8 cm for the wave height greater than about 17 cm.

The average discharges of irregular wave overtopping were obtained as listed in Table 2. These overtopping discharges of irregular waves were smaller than those of regular waves having the wave

Table 2. Experimental Discharge of Irregular Wave Overtopping

Ident.	Wave Height $H_{1/3}$ (cm)	R = 9.4 cm N	q (cm^3/cm·sec)	R = 12.8 cm N	q (cm^3/cm·sec)
1	7.5	-	-	48	0
2	10.5	-	-	164	1.20
3	12.6	96	5.53	180	5.86
4	14.8	173	10.77	155	7.33
5	16.4	159	16.90	163	12.85

height same with $H_{1/3}$. For example, the overtopping discharge of regular waves with H = 14.8 cm was 37 cm^3/cm·sec for R = 12.8 cm, but the irregular waves with $H_{1/3}$ = 14.8 cm produced the discharge of 7.3 cm^3/cm.sec. The difference between the overtoppings of regular and irregular waves, however, decreases with the decrease of wave height, and the two overtopping discharges become equal at the wave height of $H = H_{1/3}$ = 10.5 cm for R = 12.8 cm. Below this height, the overtopping discharge of irregular waves is estimated to exceed that of regular waves.

The overtopping discharge of individual wave in irregular wave train, on the other hand, does not show much difference with that of regular waves as shown in Fig. 6, where the rate of individual wave overtopping on the vertical wall of R = 12.8 cm is plotted against the wave crest height η_c in front of the vertical wall. Although the data of irregular waves show some scatter, they almost agree with those of regular waves. The scatter is partly due to the difficulty in accurate determination of individual wave overtopping quantity. The interference of preceding waves may have caused additional scatter of the overtopping data, but the tendency of Fig. 6 indicates that the irregular wave overtopping if expressed in terms of wave crest height does not differ much from that of regular waves.

The rise of wave crest of irregular waves in front of the model sea wall was then compared with that of regular waves in Fig. 7. Since the direct comparison was impossible, the wave crest height η_c and the incident wave height H_1 were arranged in order of magnitude

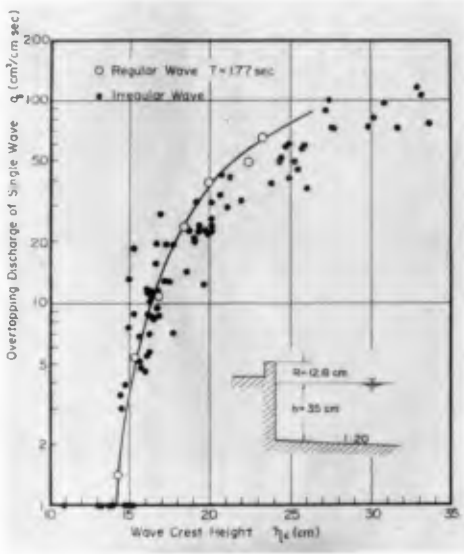

Fig. 6 Overtopping discharge of individual waves in irregular wave train

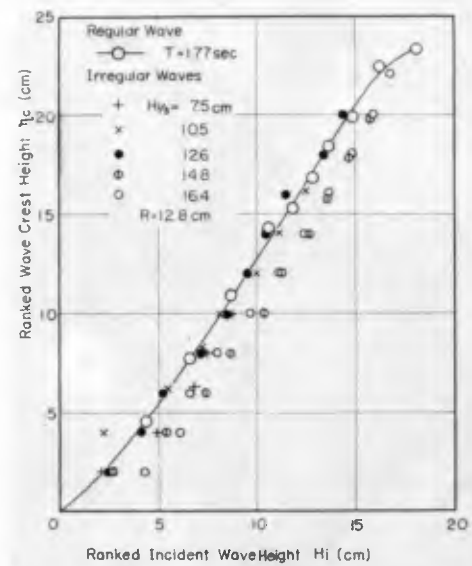

Fig. 7 Ranked wave crest height versus ranked incident wave height of irregular waves

and these ranked heights were compared. Figure 7 shows that the crest heights of irregular waves in front of the vertical wall were a little lower than those of regular waves; the difference is about 1 to 2 cm in terms of H_1.

CALCULATION OF EXPECTED OVERTOPPING DISCHARGE AND COMPARISON WITH EXPERIMENTAL DATA

The results of Figs. 6 and 7 suggests that the overtopping of irregular waves on a vertical wall can be treated separately for each wave. Therefore, the total amount of wave overtopping is estimated as the sum of individual wave overtopping as

$$Q_{total} = \sum_{i=1}^{N} Q_i(H_i) \tag{2}$$

in which Q_i denotes the overtopping quantity of a regular wave with the height of H_i; for waves under the overtopping limit, $Q_i = 0$. Since we are interested in the rate of overtopping, the above quantity is divided by the total time of wave duration, $N \cdot \bar{T}$. The actual calculation was carried out with the division of wave height $\Delta H = 1$ cm as follows

$$q_{EXP} = \frac{Q_{total}}{N \cdot \bar{T}} = \frac{1}{N} \sum_{j=1}^{M} q_j(H_j) \cdot n_j(H_j) \tag{3}$$

in which q_j is the overtopping discharge of regular waves with the height of H_j, n_j denotes the number of waves with the height between $(H_j - \frac{1}{2}\Delta H)$ and $(H_j + \frac{1}{2}\Delta H)$, and M is the number of divisions for wave height. The variation in wave period, though it affects the overtopping discharge to certain extent, was neglected in order to simplify the calculation. If the total number N is let to become very large and the division of wave height ΔH very small, the above equation is rewritten as

$$q_{EXP} = \int_0^{\infty} q(H)\ p(H)\ dH \tag{4}$$

in which p(H) is the probability density of wave height. Since this is exactly the expression for the calculation of an expected value by definition, the overtopping discharge calculated by Eq. 3 is called the expected discharge of irregular wave overtopping.

With the experimental data of regular wave overtopping shown in Fig. 6 and the histograms of wave height of incident waves, the expected discharge of irregular wave overtopping was calculated for the test conditions of Table 2. The results of calculation are compared with the experimental discharge in Fig. 8 for R = 9.4 cm and in Fig. 9 for R = 12.8 cm. The expected values are generally larger than the experimental ones· about 50 to 80% up for R = 9.4 cm and about 30 to 50% up for R = 12.8 cm. The difference is partly attributed to the effect of interference by preceding waves and to the effect of wave period, but the difficulty in maintaining the same statistical

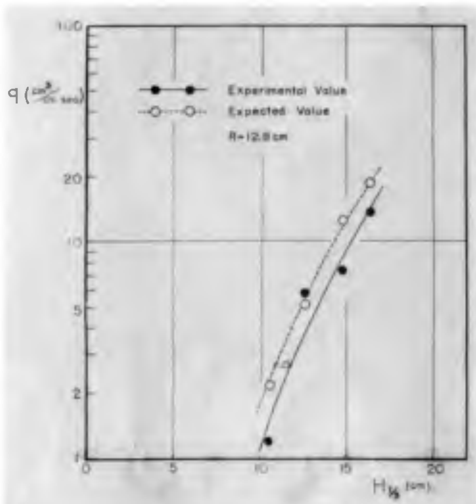

Fig. 8 Comparison of expected and experimental discharges for R=9.4cm

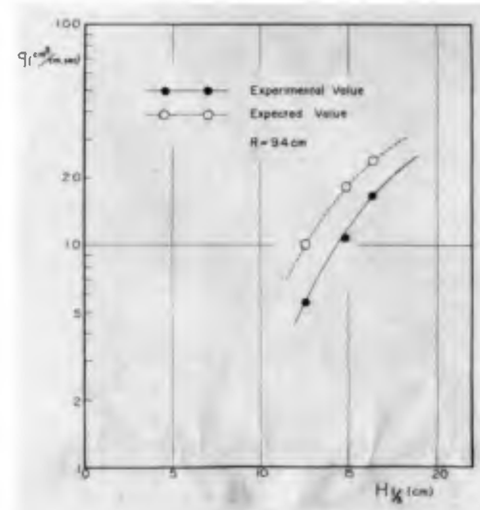

Fig. 9 Comparison of expected and experimental discharges for R=12.8cm

characteristics of irregular waves is another cause of the difference. In spite of these differences, the tendency of expected overtopping discharge agrees with that of experimental data. This agreement supports the applicability of Eq. 3 for the irregular wave overtopping on vertical walls.

ESTIMATION OF EXPECTED DISCHARGE OF IRREGULAR WAVE OVERTOPPING

Non-dimensional representation of wave overtopping data

The expected discharge of irregular wave overtopping can be estimated if the wave height distribution and the q-H curve of regular waves are known. For a specific structure, the estimation will be done based on the result of model tests. For a general planning of coast protection works, however, an over-all information on the expected overtopping discharge is often needed. The information can be obtained if the q-H curves are generalized so as to cover various laboratory data.

The general q-H curves must be in non-dimensional forms. Among various forms of non-dimensional representation, the one by Kikkawa et. al. (1967) of the following seems to be the most reasonable:

$$\frac{q}{\sqrt{2gH^3}} = \phi\left(\frac{R}{kH}\right) = \frac{1}{5}\, m\, k^{3/2} \left(1 - \frac{R}{kH}\right)^{5/2} \tag{5}$$

in which k represents the ratio of wave crest height at the time of overtopping to the incident wave height and m is the discharge coefficient of overflow on the sea wall. Since this representation is based on the similarity of wave overtopping with overflowing on a weir, this will be most applicable to the sea wall of vertical wall type.

Equation 5 is rewritten as in the following for the convenience of calculating the expected overtopping discharge·

$$\frac{q}{\sqrt{2gh^3}} = \phi\left(\frac{H}{h}\,;\ \frac{R}{h},\ \frac{h}{L_0},\ i\right) \tag{6}$$

where i denotes the slope of sea bottom. The parameter of h/L_0 represents the effect of wave period. Various laboratory data on the overtopping of regular waves on vertical walls were then re-analysed in the form of Eq. 6. The major sources of data were those by Beach Erosion Board (Saville and Caldwell 1953, and Saville 1955), University of Kyoto (Ishihara et. al. 1960 and Iwagaki et. al. 1963), and Shiraishi and Endo (1963). The bottom slope ranged from 1/10 to 1/30. The data were arranged into groups according to the value of R/h, and those with the same group were plotted against H_0/h, where H_0 denotes the deep water wave height. By this procedure, experimental curves for various values of R/h could be drawn as shown in Fig. 10.

The critical wave height $(H_0)_c$ under which no overtopping occurrs was estimated from the data of crest height of standing waves calculated by the fourth order theory (Goda and Kakizaki 1966) and of the increase of wave height in the shoaling of finite amplitude waves (Iwagaki and Sakai 1967). The result of estimation is as follows:

$R/h = 0.2 \quad (H_0/h)_c = 0.18 \sim 0.19$
$R/h = 0.4 \quad (H_0/h)_c = 0.28 \sim 0.33$
$R/h = 0.7 \quad (H_0/h)_c = 0.35 \sim 0.47$
$R/h = 1.0 \quad (H_0/h)_c = 0.42 \sim 0.58$

In the above estimation, the deep water wave steepness was takes as in the range of $H_0/L_0 = 0.01 \sim 0.03$, which corresponds to the wave height of $3.5 \sim 10.5$ m for the wave period of 15 seconds. The lower figures of the above correspond to the wave steepness of $H_0/L_0 = 0.01$.

In Fig. 10, the parameter of h/L_0 is not expressed. In general, the wave period causes certain effect on wave overtopping; the data of Tp = 15 seconds with low wave heights in Beach Erosion Board experiments shows a large overtopping discharge, much different from the data of other smaller periods. With the limitation of $H_0/L_0 > 0.01$, however, the effect of wave period becomes insignificant in comparison with the scatter of experimental data. This can be seen in Fig. 10 where the data with $R/h = 1.0$ are shown with classification according to the range of h/L_0. Also, the effect of bottom slope is not visible under the scatter of the data. The curves shown in Fig. 10 are inclusive of these factors, being drawn through the center of these data. Thus, specific data may deviate from the corresponding curve of Fig. 10 to the extent to settle around the adjacent curve, but the curves as a whole are considered to represent the tendency of wave overtopping.

NON-DIMENSIONAL OVERTOPPING DISCHARGE ON SEA WALLS OF BLOCK MOUND TYPE

Along the coasts of Japan where the available land is very limited, sea walls covered with artificial concrete blocks are often constructed, because they afford greater protection with lower crests than the sea walls of vertical wall type. Various laboratory data of model tests on wave overtopping were surveyed and re-analysed in the manner described in the previous section. The non-dimensional overtopping curves thus obtained are shown in Fig. 11. These data are of the model tests of the sea walls for the Yui Coast (Iwagaki et. al. 1963), Wakayama Port (Nagai and Takada 1964), the reclaimed area of Oita-Tsurusaki (Tatsumi 1964), the reclaimed area of Nishi-Kobe (Port and Harbour Research Institute 1965), and the Momotori Coast (Tominaga and Sakuma 1966). The data of generalized experiments by Shiraishi and Endo (1963), Nagai et. al. (1967), and Takada (1967) were also analysed. The height of block mounds above the still water level R' in these data was greater than one half of the heights of parapet walls R · i.e. $R > 0.5R'$. The experimental data of the Shonai Coast (Iwasaki and Numata 1967), though the mound heights were low, seived to give information on the over-all shape of the q-H curves.

The critical wave height $(H_0)_c$ for the block mound type sea walls was estimated from the wave run-up data by Furuya et. al. (1961), Shiraishi and Endo (1963), Morihira and Yarimizu (1967). Consideration was also given to the fact that for the range of small value of H_0/h

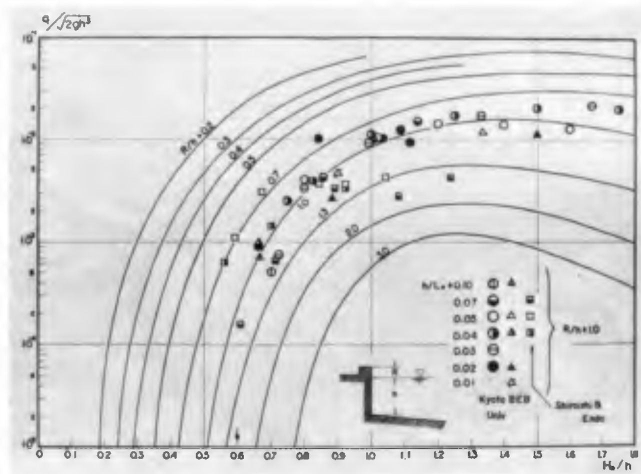

Fig. 10 Non-dimensional overtopping discharge of regular waves on vertical walls.

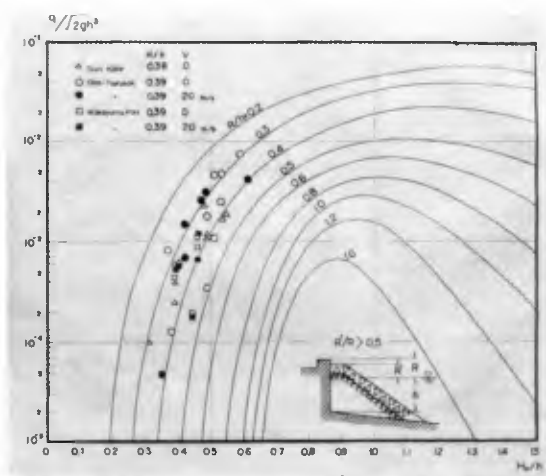

Fig. 11 Non-dimensional overtopping discharge of regular waves on sea walls covered with concrete blocks

the critical wave height is almost equal for the sea walls of vertical wall type and block mound type.

The wave overtopping data on block mound generally show larger scatter than the data on vertical wall. This is due to the fact that a slight modification in the height and width of block mound crest, or in the shape of mound especially around the too often produces significant change in the quantity of wave overtopping. The data shown in Fig. 11 for the relative parapet height of $R/h = 0.38 \sim 0.39$ exhibit rather small scatter. At larger value of R/h, the scatter of overtopping data becomes great, since the data are in the verge of wave run-up on block mound, which is about $(1.0 \sim 1.9)$Ho depending upon the characteristics of incident waves. Under the scatter of experimental data, the effect of wind upon overtopping waves cannot be recognized clearly. Some of the data for $R/h = 0.38 \sim 0.39$ shown in Fig. 11 have been obtained under the action of winds with the prototype velocity of about 20 m/sec, but they do not show marked increase over the data without the wind action. Thus, Fig. 11 represents a gross estimation of wave overtopping discharge on the sea wall covered with artificial concrete blocks. Though individual laboratory data may differ from the result of Fig. 11, the over-all tendency will be in agreement with that of Fig. 11.

CALCULATION OF EXPECTED OVERTOPPING DISCHARGE

Now the q-H curves of wave overtopping having been determined as in Figs. 10 and 11, the next step in the calculation of q_{EXP} is the estimation of wave height distribution. For the ocean waves, the Rayleigh distribution of Eq. 1 by Longuet-Higgins (1952) is accepted to be applicable. Since the effect of wave period on overtopping is not significant as indicated in Fig. 10, the marginal distribution of wave height and period such as proposed by Bretschneider (1959) has not been considered in the present calculation.

The non-dimensional calculation of expected overtopping discharge has been carried out for predetermined values of $R/(H_{1/3})_o$. Along one q-H curve with the parameter of R/h, $q_{EXP}/\sqrt{2gh^3}$ was calculated by Eq. 3. The result was converted into the form of $q_{EXP}/\sqrt{2g(H_{1/3})_o^3}$ with the ratio of $(H_{1/3})_o/h$, which is obtained by dividing R/h with $R/(H_{1/3})_o$. For the asymptotic case of $(H_{1/3})_o/h \rightarrow 0$, Eq. 5 by Kikkawa et. al.(1967) was utilized after rewriting it as follows.

$$\frac{q}{\sqrt{2g(H_{1/3})_o^3}} = 0.1\,\beta^{3/2}\cdot\eta^{3/2}\left[1 - \beta\frac{R}{(H_{1/3})_o}\cdot\frac{1}{\eta}\right]^{5/2} \tag{7}$$

with $\beta = (H_{1/3})_o/\bar{H} = 1.60$ and $\eta = H/\bar{H}$.

The parameter k for η_c/H was taken as 1 since at the limit of $(H_{1/3})_o/h \rightarrow 0$ the sinusoidal wave gives a good approximation to the wave profile. Also the discharge coefficient, m, was given a little over-estimated value of 0.5 in order to cover the difference between the sinusoidal wave and triangular wave profiles, the latter having been employed in the derivation of Eq. 5.

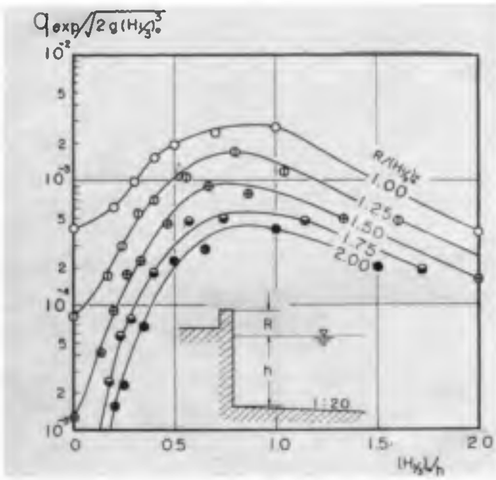

Fig. 12 Expected overtopping discharge on vertical wall

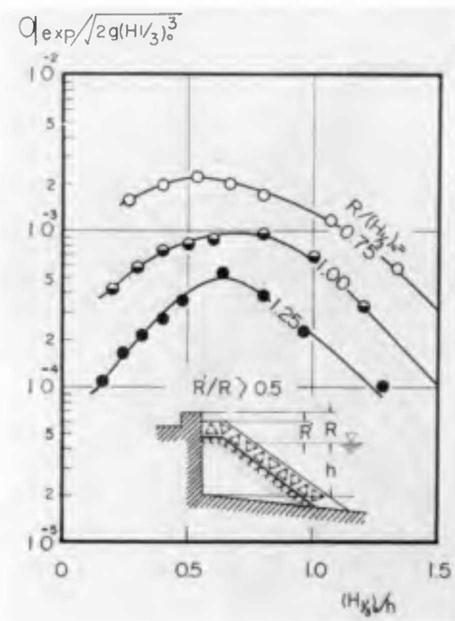

Fig. 13 Expected overtopping discharge on sea walls covered with concrete blocks.

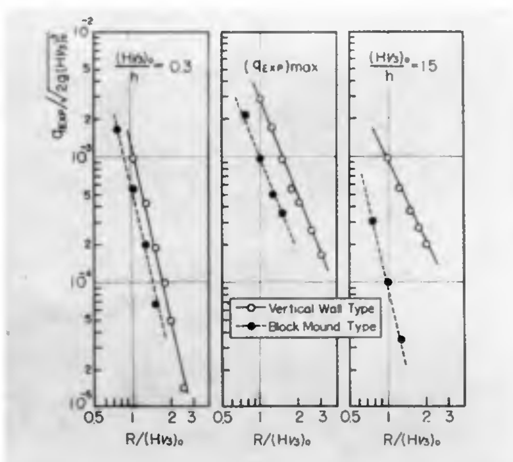

Fig. 14 Sea wall height for a given discharge of irregular wave overtopping

The result of calculation are combined in Fig. 12 for vertical walls and in Fig. 13 for block mound type sea walls. These figures reveal several characteristics of expected overtopping discharge. First, it does not respond sharply to the variation of $(H_{1/3})_o/h$. This is clearly observed for the small value of $R/(H_{1/3})_o$. Second, even with a high parapet of $R/(H_{1/3})_o = 2.0$, the average discharge of overtopping may amount to $0.0004\sqrt{2g(H_{1/3})_o^3}$ for vertical walls. The figure yields the discharge of 0.02 m^3/sec per every one meter of the sea wall for the wave height of $(H_{1/3})_o = 5$ m. If a pumping station for drainage is constructed for every thousand meters of the sea wall, the station must have the capacity greater than 20 ton/sec. Third, the maximum overtopping discharge appears at relatively low wave height i.e., $(H_{1/3})_o/h \fallingdotseq 0.8$ for vertical walls and $(H_{1/3})_o/h = 0.6$ for block mounds. The shift of peak position toward smaller height for block mounds is explained as the result of the promotion of wave breaking by presence of block mounds and of the absorption of after-breaking waves.

The height of sea wall for a given discharge of irregular wave overtopping can be read on Fig. 14 for the non-breaking wave condition of $(H_{1/3})_o/h = 0.3$, the maximum ovetopping condition, and the after-breaking condition of $(H_{1/3})_o/h = 1.5$. Figure 14 indicates that the covering of a sea wall with artificial concrete blocks for the reduction of wave overtopping is most effective for the after-breaking condition and least for the non-breaking condition. The height of block mound type sea wall for a given discharge of allowable wave overtopping can be less than 50% of the vertical wall for the after-breaking condition, but the parapet height of block mound must be about 65% of the vertical wall for the maximum overtopping condition and about 85% for the non-breaking condition.

All the results of expected overtopping discharge do not give suggestions for the amount of wave overtopping which is regarded allowable. The allowable wave overtopping is to be given a priori from the technological, economical and sociological conditions. Once the allowable quantity of wave overtopping is decided, the results of Figs. 12 to 14 will provide basic information for the sea wall height necessary to keep the wave overtopping under the allowable quantity.

CONCLUSIONS

1) The overtopping of irregular waves on a vertical wall can be treated as the linear summation of individual wave overtopping.
2) The effect of wave period on the overtopping discharge is not significant for the deep water wave steepness H_o/L_o greater than 0.01.
3) The expected discharges of irregular wave overtopping on vertical walls calculated from the overtopping data of regular waves and wave height histograms were a little larger than the experimental discharges, but the general tendency was in agreement.
4) Various laboratory data on overtopping of regular waves on sea walls of vertical wall type and block mound type can be represented in a unified form with the non-dimensionarization of $q/\sqrt{2gh^3}$ versus H/h with the parameter of R/h, as shown in Figs. 10 and 11.

5) With the assumption of the Rayleigh distribution for the wave height variability, the expected discharge of irregular wave overtopping has been calculated as shown in Fig. 12 for the sea wall of vertical wall type and in Fig. 13 for the sea wall of block mound type. The result of calculation will provide engineers with basic information for the selection of crest height and the estimation of overtopping quantity in the planning of the sea wall.

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