Baco Raton Inlet, Florida

PART 3. COASTAL STRUCTURES AND RELATED PROBLEMS

St. Johns River, Florida
CHAPTER 88

RESEARCHES ON SEA-WALLS

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ABSTRACT

In the recent decade, very wide areas of sea where the depths of water are from several meters to ten meters or more during storms have been reclaimed for industrial firms and port facilities in many places in Japan. As the incident wave energy in such cases is very large at the sea-walls, the protection of the reclaimed lands from wave overtopping by the conventional sea-walls of vertical type or composite-slope and berm type is generally impossible from an economical point of view. In Japan, a special type of sea-wall, which is of such a type that a rubble-mound covered with specially shaped precast concrete armor blocks is built in front of the sea-wall to absorb most of the incident wave energy, has been constructed to protect the reclaimed lands from wave overtopping. Most of the sea-walls have been proved satisfactory after passing of typhoons over or near the sea-walls. The design of the sea-walls is presented here in by showing the comparisons between the experiments and prototypes during typhoons.

INTRODUCTION

Until about fifteen years ago most of sea-walls had been built on the shore line or in shallow water for the protection of low lands or coastal areas from the attack of high tides and storm waves. In such cases incoming wave energy is generally not large and the sea-walls can be designed only with the estimations of the change of the incoming wave characteristics before the waves reach the sea-walls and of the wave run-up on the sea-walls. In a recent decade, however, very wide areas of sea where the depths of water are several meters to ten meters or more during typhoons or storms have been forced to reclaim at many places in Japan, because of rapid expanding industries and increasing population. In the latter cases the incident wave energy at the sea-walls is much larger than that in the former cases, and the protection of the reclaimed land from wave overtopping during typhoons is so difficult that a conventional sea-wall of vertical type would need very high crowns with heights of two to several times the design wave height above the design sea level or a sea-wall of composite-slope and berm type would require a very large cross-section. Therefore the sea-walls of these two types are seldom possible to be designed from an economical point of view.

In Japan a special type of sea-wall has been constructed since around 1961 to protect reclaimed lands and coastal areas from wave overtopping. It is of such a type that a rubble-mound covered with specially shaped precast concrete armor blocks is built in front of the sea-wall in order to absorb most of the incoming wave energy. The comprehensive experiments of the sea-walls of this type have been performed since 1960 in wind channels with a wind blower in Osaka City University, and numerous sea-walls have been designed and constructed in seas since 1961 by the use of the experimental results. Most of the
COASTAL ENGINEERING

Sea-walls have proved after attacks of storm waves during typhoons that the results obtained in the experiments were in a good agreement with prototype

PARAMETERS RELATED WITH WAVE OVERTOPPING
AND CLASSIFICATION OF OVERTOPPING

In the investigation to determine the quantity of wave overtopping from the sea-walls of special type mentioned above the following variables should be considered:

- \( q \) = quantity of overtopping over the unit length of the sea-wall for a period,
- \( H \) = height of the incoming wave,
- \( L \) = length of the incoming wave,
- \( h_1 \) = depth of water at the toe of the rubble-mound,
- \( H_C \) = height of the crown of the sea-wall above the design sea level,
- \( H_R \) = height of the crown of the rubble-mound above the design sea level,
- \( i \) = slope of the sea bottom
- \( \tan \alpha \) = slope of the rubble-mound,
- \( B \) = width of the crown of the rubble-mound,
- \( V \) = wind velocity.

These symbols are shown in Fig. 1.

If \( q_o \) defines the volume of water transported shoreward by a shallow water wave for a period, it is given by the small amplitude wave theory:

\[
q_o = \frac{HL}{2\pi} \int_0^{\frac{T}{2}} \left( \int_{h_1}^0 u \, dz \right) dt
\]

where

\[
q/o = f \left( \frac{h_1}{H}, \frac{h_1}{L}, \frac{H_C}{H}, \frac{H_R}{H}, \frac{B}{W}, \frac{V}{\sqrt{gH}}, \frac{\tan \alpha}{\tan \alpha} \right) \ldots (1)
\]

when the permeability and roughness of the rubble-mound are kept constant.

The dissipation of the energy of a wave striking the sea-wall with a rubble-mound covered with specially shaped precast concrete armors depends, to a considerable extent, on the characteristics of the armor blocks, that is, the permeability, the distribution of the voids of the armor layers and the roughness of the armors, as well as the slope of the rubble-mound, \( \tan \alpha \), and the crown width, \( B \). In practical designs the value of \( \tan \alpha \) has mostly been taken 1.15 or 1.2 from an economical point of view and the stability of the armor blocks used on the slope. The crown width, \( B \), of the rubble-mound also has usually been taken the width of two to three rows of the armor blocks from the same reasons as mentioned above.
Therefore tanα was kept 1.15 in our experiments, and most of B were taken four to six meters in prototype-scale. The velocity of wind has a great effect on the quantity of overtopping of a sea-wall when it exceeds about ten meters per second, but it was kept constant V = 20 to 25 meters per second in the experiments. According to the results of the experiments, in which B/H ranged from 1.0 to 4.0, and V/√gH were proved to have a negligibly small effect on overtopping. Finally the relative overtopping was shown as a function of the following five major dimensionless parameters

\[
\frac{q}{q_0} = f \left( \frac{h_1}{H}, \frac{H}{L}, \frac{H_C}{H}, \frac{H_T}{H}, \lambda \right),
\]

when the shape and hydraulic characteristics of the rubble-mound is kept constant.

When the depth of water at the toe of the rubble-mound, h₁, is so small that the incoming wave breaks offshore the rubble-mound, it is rather easy to construct a sea-wall which can completely prevent the overtopping even during windy storms. However, when h₁ is equal to or larger than the depth of breaking of the incoming wave, it is generally seldom possible from an economical point of view to design a sea-wall of no-overtopping. As h₁ increases compared with H, the difficulty increases much more, and it cannot be helped to permit some quantity of overtopping from the sea-wall. The quantity of overtopping to be allowed depends upon the economical value of the land to be protected, the purpose of use of the area, the stability of the sea-wall and the scale of the drainage channel. According to the results of the experiments, the state and quantity of the overtopping can be classified into the four cases shown in Table 1 by the value of q/q₀.

\[
\begin{array}{|c|c|c|}
\hline
\text{Classification} & \text{Behavior of overtopping} & \frac{q}{q_0} & \text{Property} \\
\hline
I & \text{Only spray overtops (very well absorption of wave)} & 0 \text{ to } 10^{-4} & \text{Adequate for a sea-wall} \\
\hline
II & \text{Lumps of water overtop (higher limit applicable to a sea-wall)} & 10^{-4} \text{ to } 5 \times 10^{-3} & \\
\hline
III & \text{A substantial part of wave overtops (imperfect absorption of wave)} & 5 \times 10^{-3} \text{ to } 10^{-2} & \text{Inadequate for a sea-wall} \\
\hline
IV & \text{Large volume of wave overflows (poor absorption of wave)} & 10^{-2} \text{ to } 10^{-1} & \\
\hline
\end{array}
\]
The experiments were performed by dividing into two groups, one was concerned with sea-walls constructed in comparatively shallow waters with depths less than several meters, and the other concerned with sea-walls built in deeper waters with depths from about 8 meters to 14 meters. The former group of the experiments were conducted in 1963 and 1964, and the latter in 1966 and 1967. The wave channel used for both groups of the experiments is 50 m long, 1 m wide, and 1.65 m high, and has a wind blower by which winds of velocities up to 6.0 m per sec can be blown over the water waves generated by a wave-generator of flutter-type. The scale of the experiments used in both the groups is 1/20 horizontally and vertically. The characteristics of the waves tested and other conditions used in the experiments are summarized in Table 2. The 10 m-length of the bottom in front of the sea-wall has a slope of 1, and the remaining part of the bottom is flat.

### Table 2: Conditions Used in Experiments

<table>
<thead>
<tr>
<th>Group of Experiment</th>
<th>Water depth model (h1)m cm</th>
<th>Wave characteristics model (h1)p m</th>
<th>Wave characteristics prototype Hm cm, Tm sec, Hp m,Tp sec</th>
<th>Slope of bottom</th>
<th>Wind velocity model Vm cm/sec, Vp m/sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow waters</td>
<td>2 to 33</td>
<td>0.4 to 6.6</td>
<td>3 to 14, 3.0 to 4.4, 6.0 to 13.4</td>
<td>1/10 and 1/40</td>
<td>4.5 to 20</td>
</tr>
<tr>
<td>Deeper waters</td>
<td>40 to 70</td>
<td>8.0 to 14.0</td>
<td>10 to 12, 2.8 to 5.0, 5.0 to 13.0</td>
<td>1/100</td>
<td>4.5 to 20</td>
</tr>
</tbody>
</table>

The rubble-mound was made of quarry stones with diameters of about 2 cm to 3.5 cm (40 cm to 70 cm in prototype), and covered with two layers of precast concrete armor blocks such as hollow square, hollow tetrahedron, and N-shape blocks with dead weights of 250 grams and 750 grams (2 tons and 6 tons in prototype). The slope of the rubble-mound was kept 1.15.
as shown in Fig 1 The characteristics of the armor blocks used in the experiments are summarized in Table 3

<table>
<thead>
<tr>
<th>Armor block</th>
<th>Weight in ton</th>
<th>Placing</th>
<th>NO of blocks per 100 m$^2$</th>
<th>Void ratio</th>
<th>$K_a$ for no damage condition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Reg placing</td>
<td>Pell-mell</td>
</tr>
<tr>
<td>Hollow square</td>
<td>2</td>
<td>2 layers</td>
<td>80</td>
<td>49</td>
<td>20</td>
</tr>
<tr>
<td>Hollow tetrahedron</td>
<td>6</td>
<td>2 layers</td>
<td>42</td>
<td>66</td>
<td>11</td>
</tr>
<tr>
<td>Hollow N$_1$</td>
<td>6</td>
<td>2 layers</td>
<td>45</td>
<td>53</td>
<td>20</td>
</tr>
<tr>
<td>Hollow N$_3$</td>
<td>6</td>
<td>2 layers</td>
<td>49</td>
<td>61</td>
<td>20</td>
</tr>
</tbody>
</table>

The crown width of the rubble-mound, $B$, was taken the width required for placing 2.5 or 3 rows of the armor blocks used, from the experiences in seas of the stability and absorption of wave energy of the armor blocks.

EXPERIMENTAL RESULTS

Effect of Water Depth at the Toe of the Rubble-Mound of the Sea-Wall

The volume of wave overtopping is greatly affected by the point of breaking of the incident wave, which can be divided into the following three cases:

1. When the incident wave breaks offshore from the toe of the rubble-mound. This case may be termed "Offshore Breaking", in which the overtopping is the minimum of the three cases.
2. When the incident wave breaks at or near the toe of the rubble-mound. This case may be termed "Breaking at Toe". The relative overtopping and relative run-up are the maximum of the three cases as seen in Fig 3.
3. When the incident wave breaks on the slope of the rubble-mound. This case is termed "Breaking on Slope". This case occurs when the rubble-mound is located in larger depth of water than the depth of breaking of the incident wave.
The effect of $h_1/L$ on $q/q_o$ is shown in Fig. 3 in the cases when the relative crown heights of the sea-wall and rubble-mound, $H_c/H$ and $H_r/H$, as well as the steepness of the incident wave, $H/L$, are kept constant. Fig. 3 shows that $q/q_o$ is the maximum when the toe of the rubble-mound is located at a little larger depth of water than or near the point of breaking, i.e., $h_b = 1.28 H_p$, which is the breaking depth of solitary wave. The reason is attributed to the fact that the point of breaking of the incident wave somewhat moves toward offshore due to the existence of the rubble-mound.

However, when $h_1/H > 1.7$, in which the incident wave always breaks on the slope of the rubble-mound, the parameters of $h_1/H$ and $h_1/L$ have little effect of overtopping and run-up. The experimental results obtained at the Waterways Experiment Station (1) and Coastal Engineering Research Center (2) also showed roughly the fact that when the ratio of $h_1/H$ is between 2.18 and 3.12 or as long as the waves break on the slope of a structure, $h_1/H$ had little effect on wave run-up.

**Effect of the Steepness of the Incident Wave**

Fig. 4 shows the effect of $H/L$ of the incident wave on $q/q_o$ when $H_c/H$, $H_r/H$, and $h_1/H$ are kept constant. Though the relative overtopping seems the maximum near $H/L$ of about 0.025, sea-walls in general are designed by the characteristics of design waves decided from storm conditions at the site.

**Effect of the Bottom Slope**

According to comparisons of the results of the experiments, Figs. 5 and 6, in which the bottom slope of the wave channel was changed 1/10 and 1/40, it was noted that the cases of $i = 1/10$ generally caused larger overtopping than those of $i = 1/40$, but the effect of the bottom slope on the overtopping was smaller than that of the other parameters.

**Effects of the Relative Crown Heights of the Sea-Wall and Rubble-Mound**

The effect of the relative crown height of the sea-wall, $H_c/H$, is shown in Figs. 7 and 8 for the various values of the relative crown heights of the rubble-mound, $H_r/H$. It may be seen in Figs. 7 and 8 that the value of $H_c/H$ must be taken larger than 1.0 at least in order to be $q/q_o$ less than $5 \times 10^{-3}$ which will be the higher limit applicable to the sea-walls. Figs. 9 and 10 show the effect of $H_r/H$ on $q/q_o$ for the scope of $1.0 \leq H_c/H < 1.3$, and also the effect of the bottom slope on $q/q_o$. It may be understood in Figs. 9 and 10 that in order to be $q/q_o$ less than $5 \times 10^{-3}$, $H_r/H$ must be taken larger than 0.7 for the scope of the "Breaking at Toe" and $i = 1/40$, and $H_r/H > 1.1$ for the same scope of $i = 1/10$.

**Relationships among $q/q_o$, $H_c/H$ and $H_r/H$ for the Case of "Breaking on Slope"**

As has been mentioned, when the value of $h_1/H$ exceeds 1.7 the effect of $h_1/H$ or $h_1/L$ on $q/q_o$ is negligibly smaller than the other parameters, and in the scope of the "Breaking on Slope" the following relationship was found.
\[ \frac{H_0}{H} + \frac{H_0}{X} = C \] ............................ (3)

in which \( C \) is a constant for a value of \( q/q_0 \) and a kind of armor block used on the slope of the rubble-mound. Figs. 11 and 12 show the relationships in the scope of \( h/L = 0.090 \) to 0.450 for the various values of \( q/q_0 \) and the two kinds of armor block such as N1- and N2- blocks. Tables 4 and 5 show the values of \( C \).

**TABLE 4  VALUES OF C FOR N1-BLOCK**

<table>
<thead>
<tr>
<th>( q/q_0 )</th>
<th>( 10^{-4} )</th>
<th>( 10^{-3} )</th>
<th>( 5 \times 10^{-3} )</th>
<th>( 10^{-2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>2.5</td>
<td>2.0</td>
<td>1.75</td>
<td>1.6</td>
</tr>
</tbody>
</table>

**TABLE 5  VALUES OF C FOR N3-BLOCK**

<table>
<thead>
<tr>
<th>( q/q_0 )</th>
<th>( 10^{-4} )</th>
<th>( 10^{-3} )</th>
<th>( 5 \times 10^{-3} )</th>
<th>( 10^{-2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>2.3</td>
<td>1.8</td>
<td>1.55</td>
<td>1.4</td>
</tr>
</tbody>
</table>

**Effect of the Permeability of the Rubble-Mound**

It has been well realized that the permeability and the shape of voids of rubble-mounds play a great role of the absorption of waves running up the slope of the rubble-mound. In order to prove the effect of the permeability and the shape of voids of the rubble-mound on the overtopping of sea-wall, the three kinds of N-shape armor block, shown in Fig. 13, which have a same shape but different void ratios of 53 per cent for N1, 55 per cent for N2, and 61 per cent for N3, were used as the armor block of the rubble-mound, and tetrahedron blocks which have a different shape of voids and a void ratio of 66 per cent were also used. The experimental results are shown in Fig. 14. Fig. 14 shows that for a same shape of voids of armor layers the capacity to absorb waves increases as the void ratio increases, but for different shapes of voids of armor layers the capacity to absorb wave energy is not always proportional to the void ratio.
EFFECT OF RECURVATURE OF A SEA-WALL AND SIMILARITY ON OVERTOPPING

Taking the origin of the rectangular co-ordinates at the top of the recurvature of a sea-wall, as shown in Fig. 15, the x-axis as positive toward the offshore direction, and the z-axis vertically upward, the equation of motion of a water mass, m, exerted by wind force are given by

\[ m \frac{d^2x}{dt^2} = -F \]  \hspace{1cm} \text{(4)}

\[ m \frac{d^2z}{dt^2} = -mg + F \]  \hspace{1cm} \text{(5)}

in which \( F \) denotes the horizontal component of the wind pressure acting from the offshore side, \( F_u \) is the vertical component of the wind pressure, and \( g \) defines the acceleration of gravity. \( F \) is given by

\[ F = \zeta \cdot \frac{V^2 \cdot A}{2g} \]  \hspace{1cm} \text{(6)}

in which \( V \) denotes the wind velocity, \( A \) is the area of the water mass exerted by \( F \), \( \zeta = \rho_a \cdot g = \text{unit weight of air} \), and \( \zeta \) defines the coefficient of drag which is a function of Reynolds number and the shape of the water mass.

(1) A water mass of sphere

Letting the water mass be a sphere with a diameter of \( d \), and let us calculate Reynolds number for \( d = 0.01 \) to \( 0.10 \) m, a wind velocity of 20 meters per second and at temperature of 20 degrees in Centigrade

\[ R_e = \frac{Vd}{\nu} = 1.33 \times 10^{-8} \text{ to } 1.33 \times 10^{-5} \]

The drag coefficient is nearly constant for the Reynolds number, \( \zeta = 0.4 \) to \( 0.5 \)

Taking \( \zeta = 0.5 \), and \( m = (4/3) \rho \pi r^3 \) in which \( \rho \) is the density of water, and \( r \) is the radius of the water sphere,

\[ \frac{d^2x}{dt^2} = -\frac{3}{16} \cdot \frac{\rho_a}{\rho} \cdot \frac{V^2}{r} \]  \hspace{1cm} \text{(7)}

Let the velocity of the water mass be \( V \) at \( t = 0 \) and \( x = 0 \), and the angle between the direction and the x-axis be \( \theta \),

\[ \frac{dx}{dt} = -\frac{3}{16} \cdot \frac{\rho_a}{\rho} \cdot \frac{V^2}{r} t + V \cos \theta \]  \hspace{1cm} \text{(8)}

and

\[ x = -\frac{3}{32} \cdot \frac{\rho_a}{\rho} \cdot \frac{V^2}{r} t^2 + Vt \cos \theta \]  \hspace{1cm} \text{(9)}
The time, $t_0$, which a water mass spends until it comes back again on the z-axis after leaving the top, $O$, of the sea-wall, is obtained from Eq 9

$$t_0 = \frac{v_0 \cos \theta_0}{32} \cdot \frac{\rho a}{\rho} \cdot \frac{v^2}{r}$$  \hspace{1cm} (10)$$

If $V_u$ represents the upward component of wind velocity at the top of the sea-wall, $P_u$ is given by

$$P_u = \frac{1}{3} \rho a \cdot \pi r^2 \cdot V_u$$

Assuming $V_u = \frac{1}{3} V,$

$$P_u = \frac{1}{36} \rho a \cdot \pi r^2 \cdot V^2$$ \hspace{1cm} (12)$$

Substituting Eq 12 into Eq 5,

$$\frac{d^2 z}{dt^2} = -g + \frac{1}{48} \frac{\rho a}{\rho} \frac{v^2}{r}$$ \hspace{1cm} (13)$$

as

$$\frac{dz}{dt} = v_0 \sin \theta_0, \text{ for } t = 0$$

Integrating

$$z = v_0 t \sin \theta_0 + \frac{1}{2} \left( -g + \frac{1}{48} \frac{\rho a}{\rho} \frac{v^2}{r} \right) t^2$$ \hspace{1cm} (14)$$

If the third term of the right hand side of Eq 14 is neglected, since it is approximately 20 per cent of the second term,

$$z = v_0 t \sin \theta_0 - \frac{1}{2} gt^2$$ \hspace{1cm} (15)$$

If $t_z$ represents the time which a water mass spends until it falls again onto the x-axis by the gravity force after leaving the top of the sea-wall,

$$t_z = \frac{2 v_0 \sin \theta_0}{g}$$ \hspace{1cm} (16)$$

Denoting by $t$ the time which the water mass spends until it falls down into the land over the sea-wall after leaving the top of the sea-wall,

$$t_z > t > t_0$$ \hspace{1cm} (17)$$
The values of $t_o$, $t_z$, and $(1/2)t_o v_o \cos \theta_o$, which represents the horizontal flying distance of the water mass in the time of $t_o$ are tabulated in Table 6 for $V = 10$, 15 and 20 m/sec, $v_o = 4$ meters per second, $r = 0.5$, 1.0 and 5.0 cm, and $\theta_o = 50^\circ$ and $60^\circ$.

**Table 6** Values of $t_o$, $t_z$, and $\frac{1}{2}t_o v_o \cos \theta_o$ ($\rho_a/\rho = 1/827$)

<table>
<thead>
<tr>
<th>$V$ (m/sec)</th>
<th>$\theta_o$</th>
<th>$r = 0.5$ cm</th>
<th>$r = 1$ cm</th>
<th>$r = 5$ cm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$t_o$ (sec)</td>
<td>$t_z$ (sec)</td>
<td>$\frac{1}{2}t_o v_o \times \cos \theta_o$ (m)</td>
<td>$t_o$ (sec)</td>
</tr>
<tr>
<td>10</td>
<td>50°</td>
<td>14</td>
<td>0.63</td>
<td>1.47</td>
</tr>
<tr>
<td>15</td>
<td>50°</td>
<td>50</td>
<td>0.63</td>
<td>0.65</td>
</tr>
<tr>
<td>20</td>
<td>50°</td>
<td>28</td>
<td>0.63</td>
<td>0.36</td>
</tr>
</tbody>
</table>

Since $t_o$ is larger than $t_z$ in the scope surrounded by a thick line, the water mass does not jump into the land over the sea-wall. This means that if the water mass is assumed a sphere with a diameter of $d$, the water mass with $d \leq 1$ cm will jump into the land over the sea-wall when $V \geq 15$ m/sec, the water mass with $d \leq 2$ cm will jump into the land when $V \geq 20$ m/sec.

(2) A wall of water

Let us consider that the water spray over the top of the sea-wall is a wall of water with a thickness of $b$. Since the drag coefficient of the water wall is taken $\zeta = 2$ for Reynolds numbers $Re = 5 \times 10^3$ to $10^6$, the horizontal component of the wind pressure per unit area is

$$p = \rho_a \cdot v^2$$

Since $m = \rho \cdot b$, Eq 4 may be written

$$\frac{d^2x}{dt^2} = - \frac{\rho_a \cdot v}{\rho} \cdot \frac{v^2}{b}$$

(18)
Using the boundary conditions \( \frac{dx}{dt} = v_0 \cos \theta_0 \) and \( x = 0 \) for \( t = 0 \),

\[
x = -\frac{1}{2} \cdot \frac{\partial a}{\rho} \cdot \frac{v^2}{b} t^2 + v_0 t \cos \theta_0 \quad \cdots \quad \cdots \quad \cdots \quad \cdots \quad (20)
\]

\( t = t_o \) for \( x = 0 \) is given by

\[
t_o = \frac{v_0 \cos \theta_0}{\frac{1}{2} \cdot \frac{\partial a}{\rho} \cdot \frac{v^2}{b}} \quad \cdots \quad \cdots \quad \cdots \quad \cdots \quad (21)
\]

If \( P_a \) is neglected, \( t_z \) is given by Eq. 16. As previously mentioned, only when the condition \( t_z > t > t_o \) is satisfied, the water wall can fall into the land over the sea-wall. Table 7 is shown the values of \( t_o \), \( t_z \), and \( \frac{1}{2} t_o v_0 \cos \theta_0 \).

**TABLE 7** VALUES OF \( t_o \), \( t_z \), and \( \frac{1}{2} t_o v_0 \cos \theta_0 \)

<table>
<thead>
<tr>
<th>( V ) (m/sec)</th>
<th>( \theta_0 )</th>
<th>( b = 1 ) cm</th>
<th>( b = 5 ) cm</th>
<th>( b = 10 ) cm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( t_o ) (sec)</td>
<td>( t_z ) (sec)</td>
<td>( \frac{1}{2} \cdot \frac{v_0 \cos \theta_0}{\rho} ) (m)</td>
<td>( t_o ) (sec)</td>
</tr>
<tr>
<td>10</td>
<td>0.43</td>
<td>0.63</td>
<td>0.55</td>
<td>2.13</td>
</tr>
<tr>
<td>15</td>
<td>0.33</td>
<td>0.71</td>
<td>0.33</td>
<td>1.65</td>
</tr>
<tr>
<td>20</td>
<td>0.19</td>
<td>0.24</td>
<td>0.15</td>
<td>0.95</td>
</tr>
<tr>
<td>10</td>
<td>0.11</td>
<td>0.14</td>
<td>0.08</td>
<td>0.53</td>
</tr>
<tr>
<td>15</td>
<td>0.08</td>
<td>0.08</td>
<td>0.08</td>
<td>0.41</td>
</tr>
</tbody>
</table>

In Table 7 the scope enclosed by a thick line shows the cases of \( t_o > t_z \). According to Table 7, it may be known that a wall of overtopping with a thickness of one cm is blown down within one second into the land over the sea-wall by winds with velocities equal to or larger than 10 m/sec, and a wall of overtopping with a thickness of 5 cm by winds with velocities equal to or larger than 20 m/sec.
Similarity on Overtopping

(1) A water mass of sphere

Let us consider of a water mass of sphere with a diameter \( d \) overtopping a sea-wall. Assuming \( d_p \), the diameter in prototype, is 3 cm to 10 cm, \( d_m \), the diameter in model, the scale of which is 1:20 to prototype, is 1.5 to 5 mm.

Reynolds number \( p \text{Re} \) for a wind velocity in prototype of \( V_p = 20 \text{ m/sec} \) is

\[
p \text{Re} = \frac{V_p \cdot d_p}{v} = 4 \times 10^4 \text{ to } 1 \times 10^5
\]

In the model

\[
m \text{Re} = \frac{V_m \cdot d_m}{v} = 4.5 \times 10^2 \text{ to } 1.47 \times 10^3
\]

Drag coefficients for the spheres are

\[
\zeta_p \simeq 0.45 \text{ to } 0.5 \text{ in prototype,}
\]

and

\[
\zeta_m \simeq 0.60 \text{ to } 0.45 \text{ in model,}
\]

thus, it may be assumed approximately \( \zeta_p = \zeta_m \).

This means that if a mass of water overtopping a sea-wall is a sphere with a diameter \( d \geq 3 \text{ cm} \), the motion of the water mass exerted by a wind of \( V_p = 20 \text{ m/sec} \) may be stated to be approximately followed by Froude law of similarity, i.e., the results of the model experiment may be stated to be approximately similar to the results of the nature.

However, if the diameter of the water mass, \( d_p \), is smaller than about 3 cm, \( \zeta_p \simeq 0.4 \text{ to } 0.5 \) for \( p \text{Re} = \sim 10^4 \), as against \( \zeta_m \simeq 0.6 \text{ to } 0.9 \) for \( m \text{Re} = \sim 10^2 \). Therefore we cannot expect a good similitude between model and prototype. But from a practical point of view on the water quantity of wave overtopping, the volume of such spray of water would be considered to be negligible small.

(2) A wall of water

Let us consider the overtopping as a wall of water. This may be the case when large overtopping is seen in prototype and model, as seen in Figs. 17, 18, and 19, therefore, this case would be the most important in wave overtopping over sea-walls.

If the overtopping of wave is assumed as a wall of water, the drag coefficient of the wall, \( \zeta \), is constantly two for all Reynolds numbers larger than \( 10^2 \). It, therefore, may be stated that the results of experiments conducted by Froude law are similar to the results in the nature. Field observations of overtopping at sea-walls during typhoons have proved that this assumption is correct.

It may be concluded from the theoretical considerations described above that the recurvature of a sea-wall would have little effect on wave overtopping when wind velocity exceeds about 15 m/sec.
APPLICATION TO DESIGN OF SEA-WALLS AND VERIFICATION BY TYPHOONS

The results studied in our laboratory have been applied to the design of sea-walls in Japan since 1960 and those sea-walls have been tested by severe typhoons. All of the sea-walls which have undergone the natural tests have proved that the designs were satisfactory and there were generally a fairly good agreement between the experiments and prototypes. Some examples are presented herein.

(1) Sea-wall in the Port of Wakayama

This sea-wall was constructed in 1958 at a water depth of 7 m to 8 m below the Datum Line offshore a long sandy beach exposed to an open sea in the North Harbor of the Port of Wakayama, Wakayama Prefecture, which is one of the biggest industrial harbors for steel firms in Japan. This was the first big sea-wall that harbor engineers in Japan constructed to protect a reclaimed industrial land located offshore from wave overtopping. Fig 16 shows a cross-section of the sea-wall.

After completion it was severely hit three times consecutively in September of 1959, 1960 and 1961. Figs 17 and 18 show huge overtoppings of waves which were taken when the typhoon was located still far offshore from the harbor.

After calculations and experiments, it was recommended that the sea-wall should have a rubble-mound covered with precast concrete armors in front of the wall and a large drainage channel with a width of 20 m at the land-side of the sea-wall, as shown in Fig 19.

(2) Another Sea-Wall in the Port of Wakayama

The sea-wall, the cross-section of which is shown in Fig 20, was constructed in 1965 and 1966 to protect the industrial area of 1 million m² reclaimed at the northern part of the Wakayama North Harbor.

(a) When the height and period of the design wave are taken \( H_D = 5.0 \) m and \( T_D = 9.0 \) sec

Since \( h_1/H = 8.74/5 = 1.75 > 1.7 \), this case is "Breaking on Slope".

For \( H_C/H = 4.76/5 = 0.95 \approx 1.0 \) and \( H_T/H = 3.76/5 = 0.75 \), we obtain \( q/q_0 = 5 \times 10^{-3} \) from Fig 11.

(b) When \( H_D = 5.5 \) m and \( T_D = 13.0 \) sec

Since \( H/L = 0.041 \) and \( h_1/L = 0.065 \), this case is on the critical condition between "Breaking at Toe" and "Breaking on Slope".

Using \( H_C/H = 0.87 \) and \( H_T/H = 3.76/5 = 0.68 \), we obtain \( q/q_0 = 5 \times 10^{-3} \) for "Breaking at Toe", and from Fig 11 \( q/q_0 = 8 \times 10^{-3} \) for "Breaking on Slope", which necessitate a large drainage channel as shown in Fig 20.

Decision of the Width and Depth of the Drainage Channel

The design conditions of the drainage channel.

\[ \Delta z = \text{height from the bottom of the drainage channel to the x-axis} \]

\[ = (D_L + 6.70 \text{ m}) - (D_L + 4.00 \text{ m}) = 2.70 \text{ m} \]

\[ \theta_0 = \text{angle of the parapet wall to the x-axis} = 70^\circ \]

\[ V = \text{wind velocity} = 20 \text{ m/sec} \]

The wave run-ups, \( R_u \), for the design waves with heights of 5.0 m and 5.5 m and periods of 9 sec and 13 sec, respectively, were approximately 1.2 H according to the experiments carried out by using the model of the sea-wall shown in Fig 21.
Let us consider about the design wave with a height of 5 m and a period of 9 sec, and take the design sea level \( D_L + 24 \) m. The velocity of a water mass of sphere with a diameter of \( d \) at the origin of the co-ordinates, 0, is obtained by \( v_0 = \sqrt{2 \times 9.8 (60 - 4.46)} = 5.5 \) m/sec.

\( t_z \), which denotes the time that the water mass spends until it falls down on the bottom of the drainage channel over the sea-wall after leaving the origin 0 is given by

\[
t_z = \frac{v_0 \sin \theta_0 + \sqrt{v_0^2 \sin^2 \theta_0 + 2g \Delta z}}{g}
\]

By substitution of Eq 22 into Eq 9, the horizontal distance \( l_x \) which the water mass reaches at the time \( t_z \) from the origin 0 onto the channel bottom is obtained by

\[
l_x = \frac{3}{32} \cdot \frac{\rho a}{\rho} \cdot \frac{v^2}{x} t_z - v_0 t \cos \theta_0
\]

From Eqs 22 and 23 \( t_z = 1.44 \) sec and \( l_x = 16 \) 0 m for \( d = 0.01 \) m, \( l_x = 6.7 \) m for \( d = 0.02 \) m.

Considering a wall of water with a thickness of \( b \), the horizontal distance \( l_x \) is obtained by substitution of Eq 22 into Eq 20, \( l_x = 47.4 \) m for \( b = 0.01 \) m, \( l_x = 22.4 \) m for \( b = 0.02 \) m, \( l_x = 14.0 \) m for \( b = 0.03 \) m, and \( l_x = 9.8 \) m for \( b = 0.04 \) m.

The loci of those walls of water are shown in Fig 21. According to Fig 21, if the side wall with a height of 2 m from the bottom of the channel is constructed at a distance of \( l_x = 17.5 \) m from the origin 0, the walls of water with thicknesses of more than \( b = 2 \) cm and the spheres of water with diameters more than one cm could be taken into the drainage channel.

The length of the sea-wall over which the design wave overtops simultaneously was decided about 350 m by the experiments, and the factor of safety for the maximum relative overtopping was taken two, therefore the maximum discharge of water for the drainage channel, \( Q_{\text{max}} \), was

\[
Q = 8 \times 10^3 \times 5 \times 88/2\pi = 0.56 \text{ m}^3/9\text{sec/m}
\]

\[
Q_{\text{max}} = 2q \times 3.50 = 393 \text{ m}^3/9\text{sec}
\]

\[
= 43.7 \text{ m}^3/\text{sec} \approx 45 \text{ m}^3/\text{sec}
\]

If the slope of the channel bottom is taken 1/1000, the width 20 m, and the depth 2 m, the channel can discharge \( Q_{\text{max}} \) safely.

After completion of the drainage channel, the sea-wall has undergone severe typhoons several times, and it has been reported due to the visual observations during the typhoons that the behaviors of the overtopping were quite similar to those of the experiments and almost all overtoppings were assembled into the channels and flowed down safely into the harbor basin.

(3) Sea-Wall of the Kansai Electric Power Co Ltd

The sea-wall was constructed in an open sea with a water depth of \( D_L = 250 \) m to 300 m to protect a reclaimed land of about 38 acres which was used for an electric power plant of the Kansai Electric Power Co Ltd. The sea-wall was requested to be designed no-overtopping even
during the heaviest typhoon ever experienced there. After the calculations shown here and model experiments carried out on a scale of 1:20 in the wave channel shown in Fig. 2, the sea-wall shown in Figs. 22 and 23 was designed.

**Estimation of the Overtopping during the Heaviest Typhoon**

The significant wave height and period of the design wave were taken as $H_{1/3} = 2.50$ m, and $T_{1/3} = 6.3$ sec. The highest high tide averaged for two hours including the highest tide ever recorded in the harbor was estimated as $H_L + 3.80$ m. Since $h/L = 5.80/2.5 = 2.3 > 1.7$, the case is "Breaking on Slope." Using $H_c/H = 3.95/2.5 = 1.58$ and $H_r/H = 1.75/2.5 = 0.70$, we obtain from Figs. 11 and 12, $q/q_o = 5 \times 10^{-1}$ and $2 \times 10^{-1}$, respectively.

Since the tetrahedron blocks of two tons were used as the armor block of the rubble-mound, $q/q_o$ is estimated approximately $3 \times 10^{-1}$ from Fig. 14. According to the experiments in which two high tides of $H_L + 3.80$ m and $H_L + 3.30$ m were used and the wind velocity used was always $20$ m/sec, a few overtopping was seen for a tide of $H_L + 3.80$ m and only a few spray was observed for a tide of $H_L + 3.30$ m.

Shortly after the completion of the sea-wall, it was hit by one of the most severe typhoons ever observed in the harbor, the Second Muroto typhoon, which passed near over the location of the sea-wall on September 16, 1961. The highest tide during the typhoon was $H_L + 4.00$ m and the average highest high tide for two hours was estimated approximately $H_L + 3.60$ m. The maximum wave height was assumed $2.50$ m or $3.0$ m, and strong winds of from $20$ m/sec to $35$ m/sec blew from offshore for about $3$ hr.

Pictures taken with the 8-mm movie camera near the top of the sea-wall during the typhoon showed that comparatively small volume of wave overtopping occurred sometimes over the top of the rubble-mound covered with the two layers of the hollow tetrahedron armors. However, there was no overtopping over the top of the sea-wall, as seen in Fig. 23. It was proved that there was a good agreement between the experiment and prototype.

**APPENDIX - REFERENCES**

2. Saville, Thorndike, "Wave Run-up on Shore Structures," Proc ASCE, Vol 82, NO WW2, April, 1956
FIG. 1. - CROSS-SECTIONS TYPICAL TYPES OF SEA-WALL IN JAPAN
UPPER SECTION USED IN DEEPER WATERS
LOWER SECTION USED IN SHALLOW BEACH

FIG. 2. - WAVE CHANNEL WITH A WIND BLOWER

FIG. 3. - RELATIONSHIP BETWEEN RELATIVE OVERTOPPING AND $h_1/L$

FIG. 4. - RELATIONSHIP BETWEEN $q/q_0$ AND $H/L$
FIG 5 - RELATIONSHIP BETWEEN $q/q_0$ AND $H_r/H$, $l = 1/40$

FIG 6 - RELATIONSHIP BETWEEN $q/q_0$ AND $H_r/H$, $l = 1/10$

FIG 7 - RELATIONSHIP AMONG $q/q_0$, $H_c/H$ AND $H_r/H$

FIG 8 - RELATIONSHIP AMONG $q/q_0$, $H_c/H$ AND $H_r/H$
FIG 9 - RELATIONSHIP BETWEEN $q/q_0$ AND $H_r/H$, $r = 1/40$

FIG 10 - RELATIONSHIP BETWEEN $q/q_0$ AND $H_r/H$, $r = 1/10$

FIG 11 - RELATIONSHIP AMONG $q/q_0$, $H_c/H$, AND $H_r/H$ FOR N1-BLOCK

FIG 12 - RELATIONSHIP AMONG $q/q_0$, $H_c/H$, AND $H_r/H$ FOR N3-BLOCK
FIG. 14. - RELATIONSHIPS BETWEEN $q/q_0$ AND THE VOID RATIO AND SHAPE OF VOID OF ARMOR LAYERS

FIG. 13. - HOLLOW N-BLOCK

FIG. 15

FIG. 16. - CROSS-SECTION OF THE SEA-WALL IN WAKAYAMA HARBOR BEFORE 1961

FIG. 19. - CROSS-SECTION OF THE SEA-WALL IMPROVED AFTER THE EXPERIMENTS
FIG. 17. - LARGE WAVE OVERTOPPINGS AT SEA-WALLS OF WAKAYAMA HARBOR

FIG. 18.

FIG. 20. - CROSS-SECTION OF THE SEA-WALL IN WAKAYAMA HARBOR

FIG. 21. - LOCI OF THE WALLS OF WATER OVERTOPPING THE SEA-WALL IN FIG. 20

FIG. 22. - CROSS-SECTION OF THE SEA-WALL OF THE KANSAI ELECTRIC POWER CO.LTD

FIG. 23. - SEA-WALL OF THE KANSAI E. P. CO.LTD. DURING A TYphoon, 1961