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PART 3. COASTAL STRUCTURES AND RELATED PROBLEMS

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CHAPTER 88

RESEARCHES ON SEA-WALLS

S Nagai
Professor of Hydraulics,
Dean of Faculty of Engineering,
Osaka City University, Osaka, Japan

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 place 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 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, and
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

$$q_o = \int_0^{T/2}\int_{-h_1}^{0} u \, dzdt$$

$$= \int_0^{T/2}\int_{-h_1}^{0} \frac{H}{2}\frac{2\pi}{T}\frac{\cosh\frac{2\pi}{L}(h_1+Z)}{\sinh\frac{2\pi}{L}h_1}\sin\left(\frac{2\pi}{L}x - \frac{2\pi}{T}t\right) dzdt$$

$$= \frac{HL}{2\pi}$$

If q/q_o is used as a dimensionless parameter of wave overtopping, it is a function of the following dimensionless parameters

$$q/q_o = f\,(\,h_1/H,\ h_1/L,\ H_c/H,\ H_r/H,\ B/H,\ i,\ V/\sqrt{gH},\ \tan\alpha\,) \cdots\cdots (1)$$

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 1 5 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 $\tan\alpha$ was kept 1 1 5 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 $V/\sqrt{gH}$ 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

$$q/q_o = f\ (\ h_1/H,\ H/L,\ H_c/H,\ H_r/H,\ i\ ), \cdots\cdots\cdots\cdots (2)$$

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, h_1, 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 h_1 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 h_1 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/q_o

TABLE 1 CLASSIFICATION OF BEHAVIORS OF OVERTOPPING

Classification	Behavior of overtopping	q/q_o	Propriety
I	Only spray overtops (very well absorption of wave)	0 to 10^{-4}	Adequate for a sea-wall
II	Lumps of water overtop (higher limit applicable to a sea-wall)	10^{-4} to 5×10^{-3}	
III	A substantial part of wave overtops (imperfect absorption of wave)	5×10^{-3} to 10^{-2}	Inadequate for a sea-wall
IV	Large volume of wave overflows (poor absorption of wave)	10^{-2} to 10^{-1}	

EXPERIMENTAL EQUIPMENT AND PROCEDURES

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 summerized in Table 2 The 10 m-length of the bottom in front of the sea-wall has a slope of i, and the remaining part of the bottom is flat

TABLE 2 CONDITIONS USED IN EXPERIMENTS

Group of experiment	Water depth		Wave characteristics				Slope of bottom	Wind velocity	
	model	proto-type	model		proto-type			model	proto-type
	$(h_1)_m$ cm	$(h_1)_p$ m	Hm cm	Tm sec	Hp m	Tp sec	i	Vm cm/sec	Vp m/sec
Shal-lower waters	2 to 33	0 4 to 6 6	3 to 22	1 4 to 3 0	0 6 to 4 4	6 3 to 13 4	1/10 and 1/40	4 5	20
Deeper waters	40 to 70	8 0 to 14 0	10 to 25	1 12 to 2 8	2 0 to 5 0	5 0 to 13 0	1/100	4 5	20

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 1 5 as shown in Fig 1 The characteristics of the armor blocks used in the experiments are summerized in Table 3

TABLE 3 CHARACTERISTICS OF ARMOR BLOCKS USED IN EXPERIMENTS

Armor block	Weight in ton	Placing	NO of blocks per 100 m^2	Void ratio	K_Δ for no damage condition	
					Reg placing	Pell-mell
Hollow square	2	2 layers	80	49	20	13
Hollow tetrahedron	6	2 layers	42	66	11	7 6
Hollow N_1	6	2 layers	45	53	20	-
Hollow N_3	6	2 layers	49	61	20	-

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_0 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_0 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 $h_b = 1\ 28\ H_b$ 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 $h_1/H \geqq 1\ 7$, 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_0 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_0 less than 5×10^{-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_0 for the scope of $1\ 0 \leqq H_c/H < 1\ 3$, and also the effect of the bottom slope on q/q_0 It may be understood in Figs 9 and 10 that in order to be q/q_0 less than 5×10^{-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_0, 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_0 is negligibly smaller than the other parameters, and in the scope of the "Breaking on Slope" the following relationship was found

$$\frac{H_c}{H} + \frac{H_r}{H} = C \quad \cdots\cdots\cdots\cdots \quad (3)$$

in which C is a constant for a value of q/q_o 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_o and the two kinds of armor block such as N_1- and N_2- blocks Tables 4 and 5 show the values of C

TABLE 4 VALUES OF C FOR N_1-BLOCK

q/q_o	10^{-4}	10^{-3}	5×10^{-3}	10^{-2}
C	2 5	2 0	1 75	1 6

TABLE 5 VALUES OF C FOR N_3-BLOCK

q/q_o	10^{-4}	10^{-3}	5×10^{-3}	10^{-2}
C	2 3	1 8	1 55	1 4

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 ratioes of 53 per cent for N_1, 55 per cent for N_2, and 61 per cent for N_3, 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

$$m \frac{d^2x}{dt^2} = - P \qquad (4)$$

$$m \frac{d^2z}{dt^2} = - mg + P_u \qquad (5)$$

in which P denotes the horizontal component of the wind pressure acting from the offshore side, P_u is the vertical component of the wind pressure, and g defines the acceleration of gravity P is given by

$$P = \zeta \cdot w_a \cdot A \cdot \frac{V^2}{2g} \qquad (6)$$

in which V denotes the wind velocity, A is the area of the water mass exerted by P, $w_a = \rho_a \cdot g$ = unit weight of air, and ζ 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

$$R_e = \frac{Vd}{\nu} = 1\ 33 \times 10^{-4} \text{ to } 1\ 33 \times 10^{-5}$$

The drag coefficient is nearly constant for the Reynolds number, i e , ζ = 0 4 to 0 5

Taking ζ = 0 5, and $m = (4/3)\rho\pi r^3$ in which ρ is the density of water, and r is the radius of the water sphere,

$$\frac{d^2x}{dt^2} = - \frac{3}{16} \cdot \frac{\rho_a}{\rho} \cdot \frac{V^2}{r} \qquad (7)$$

Let the velocity of the water mass be v_o at t = 0 and x = 0, and the angle between the direction and the x-axis be θ_o,

$$\frac{dx}{dt} = - \frac{3}{16} \frac{\rho_a}{\rho} \cdot \frac{V^2}{r} t + v_o \cos \theta_o \qquad (8)$$

and

$$x = - \frac{3}{32} \cdot \frac{\rho_a}{\rho} \cdot \frac{V^2}{r} t^2 + v_o t \cos \theta_o \qquad (9)$$

The time, t_o, 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

$$t_o = \frac{v_o \cos \theta_o}{\frac{3}{32} \cdot \frac{\rho a}{\rho} \cdot \frac{V^2}{r}} \quad \cdots\cdots \quad (10)$$

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

$$P_u = \zeta \cdot \rho_a \cdot g \cdot A \cdot \frac{V_u^2}{2g} = \frac{1}{4} \rho_a \cdot \pi r^2 \cdot V_u^2 \quad \cdots\cdots \quad (11)$$

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

$$P_u = \frac{1}{36} \rho_a \cdot \pi r^2 \cdot V^2 \quad \cdots\cdots \quad (12)$$

Substituting Eq 12 into Eq 5,

$$\frac{d^2 z}{dt^2} = - g + \frac{1}{48} \frac{\rho_a}{\rho} \frac{V^2}{r} \quad \cdots\cdots \quad (13)$$

as

$$\frac{dz}{dt} = v_o \sin \theta_o, \text{ for } t = 0$$

Integrating

$$z = v_o t \sin \theta_o + \frac{1}{2} \left(- g + \frac{1}{48} \cdot \frac{\rho_a}{\rho} \cdot \frac{V^2}{r} \right) t^2 \quad \cdots\cdots \quad (14)$$

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,

$$z = v_o t \sin \theta_o - \frac{1}{2} g t^2 \quad \cdots\cdots \quad (15)$$

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,

$$t_z = \frac{2 v_o \sin \theta_o}{g} \quad \cdots\cdots \quad (16)$$

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,

$$t_z \geqq t > t_o \quad \cdots\cdots \quad (17)$$

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 θ_o = 50° and 60°

TABLE 6 VALUES OF t_o, t_z, and $\frac{1}{2}\ t_o v_o \cos \theta_o$ $(\rho_a/\rho = 1/827)$

V (m/sec)	θ_o	r = 0 5 cm			r = 1 cm			r = 5 cm		
		t_o (sec)	t_z (sec)	$\frac{1}{2}t_o v_o \times\cos\theta_o$ (m)	t_o (sec)	t_z (sec)	$\frac{1}{2}t_o v_o \times\cos\theta_o$ (m)	t_o (sec)	t_o (sec)	$\frac{1}{2}t_o v_o \times\cos\theta_o$ (m)
10	50°	1 14	0 63	1 47	2 27	0 63	2 93	11 4	0 63	14 7
	60	0 88	0 71	0 88	1 76	0 71	1 76	8 80	0 71	8 80
15	50	0 50	0 63	0 65	1 01	0 63	1 30	5 04	0 63	6 48
	60	0 39	0 71	0 39	0 78	0 71	0 78	3 92	0 71	3 92
20	50	0 28	0 63	0 36	0 57	0 63	0 72	2 85	0 63	3 68
	60	0 22	0 71	0 22	0 44	0 71	0 44	2 20	0.71	2 20

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 $\leqq$ 1 cm will jump into the land over the sea-wall when V $\geqq$ 15 m/sec, the water mass with d $\leqq$ 2 cm will jump into the land when V $\geqq$ 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 ζ = 2 for Reynolds numbers Re = 5×10^3 to 10^6, the horizontal component of the wind pressure per unit area is

$$P = \rho_a \cdot V^2 \quad \cdots\cdots\cdots\cdots \quad (18)$$

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

$$\frac{d^2x}{dt^2} = - \frac{\rho_a}{\rho} \cdot \frac{V^2}{b} \quad \cdots\cdots\cdots\cdots \quad (19)$$

Using the boundary conditions $dx/dt = v_o \cos\theta_o$ and $x = 0$ for $t = 0$,

$$x = -\frac{1}{2} \cdot \frac{\rho_a}{\rho} \cdot \frac{V^2}{b}\, t^2 + v_o t \cos\theta_o \quad \cdots\cdots \quad (20)$$

$t = t_o$ for $x = 0$ is given by

$$t_o = \frac{v_o \cos\theta_o}{\frac{1}{2} \cdot \frac{\rho_a}{\rho} \cdot \frac{V^2}{b}} \quad \cdots\cdots \quad (21)$$

If P_u is neglected, t_z is given by Eq 16 As previously mentioned, only when the condition $t_z \geqq t > t_o$ 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 (1/2) $t_o v_o \cos\theta_o$

TABLE 7 VALUES OF t_o, t_z, and $\frac{1}{2} t_o v_o \cos\theta_o$

V (m/sec)	θ_o	b = 1 cm			b = 5 cm			b = 10 cm		
		t_o (sec)	t_z (sec)	$\frac{1}{2}t_o v_o \times\cos\theta_o$ (m)	t_o (sec)	t_z (sec)	$\frac{1}{2}t_o v_o \times\cos\theta_o$ (m)	t_o (sec)	t_z (sec)	$\frac{1}{2}t_o v_o \times\cos\theta_o$ (m)
10	50° 60	0 43 0 33	0 63 0 71	0 55 0 33	2 13 1 65	0 63 0 71	2 88 1 65	4 25 3 31	0 63 0 71	5 46 3 31
15	50 60	0 19 0 15	0 63 0 71	0 24 0 15	0 95 0 74	0 63 0 71	1 22 0 74	1 89 1 47	0 63 0 71	2 43 1 47
20	50 60	0 11 0 08	0 63 0 71	0 14 0 08	0 53 0 41	0 63 0 71	0 68 0 41	1 06 0 83	0 63 0 71	1 36 0 83

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 thicknessof 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 $_pRe$ for a wind velocity in prototype of V_p = 20 m/sec is

$$_pRe = \frac{V_p \cdot d_p}{\nu} = 4\ 0 \times 10^4 \text{ to } 1\ 33 \times 10^5$$

In the model

$$_mRe = \frac{V_m \cdot d_m}{\nu} = 4\ 5 \times 10^2 \text{ to } 1\ 47 \times 10^3$$

Drag coefficients for the spheres are

$$\zeta_p = \sim 0\ 45 \text{ to } 0\ 5 \text{ in prototype,}$$

and

$$\zeta_m = \sim 0\ 60 \text{ to } 0\ 45 \text{ in model,}$$

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 cm, the motion of the water mass exerted by a wind of V_p = 20 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 ne approximately similar to the results of the nature

However, if the diameter of the water mass, d_p, is smaller than about 3 cm, $\zeta_p = \sim 0\ 4$ to 0 5 for $_pRe = \sim 10^4$, as against $\zeta_m = \sim 0\ 6$ to 0 9 for $_mRe = \sim 10^2$ 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, ζ, 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 ungone 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 8 million m^2 reclaimed at the northern part of the Wakayama North Harbor

(a) When the height and period of the design wave are taken H_p = 5 0 m and T_p = 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 = ∿ 1 0 and H_r/H = 3 76/5 = 0 75, we obtain $q/q_o = 5 \times 10^{-3}$ from Fig 11

(b) When H_p = 5 5 m and T_p = 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_r/H = 3 76/5 5 = 0 68, we obtain $q/q_o = 5 \times 10^{-3}$ for "Breaking at Toe", and from Fig 11 $q/q_o = 8 \times 10^{-3}$ for "Breaking on Slope", which necessitate a large drainage channel as shown in Fig 20

<u>Decision of the Width and Depth of the Drainage Channel</u>

The design conditions of the drainage channel

Δz = height from the bottom of the drainage channel to the x-axis
= (D L + 6 70 m) - (D L + 4 00 m) = 2 70 m

θ_o = angle of the parapet wall to the x-axis = 70°,

V = wind velocity = 20 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 + 2 24 m The velocity of a water mass of sphere with a diameter of d at the origin of the co-ordinates, O, is obtained by $v_o = \sqrt{2 \times 9\ 8\ (6\ 0 - 4\ 46)} = 5\ 5$ 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 O is given by

$$t_z = \frac{v_o \sin\theta_o + \sqrt{v_o^2 \sin^2\theta_o + 2g\,\Delta z}}{g} \quad \cdots\cdots\cdots\cdots\cdots\cdots (22)$$

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 O onto the channel bottom is obtained by

$$l_x = \frac{3}{32} \cdot \frac{\rho a}{\rho} \cdot \frac{V^2}{r}\, t_z - v_o t \cos\theta_o \quad \cdots\cdots\cdots\cdots (23)$$

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 O, 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_{max}, was

$$q = 8 \times 10^{-3} q_o = 8 \times 10^{-3} \times 5 \times 88/2\pi = 0\ 56\ \mathrm{m^3/9sec/m}$$

$$Q_{max} = 2q \times 350 = 393\ \mathrm{m^3/9sec}$$

$$= 43\ 7\ \mathrm{m^3/sec} = \sim 45\ \mathrm{m^3/sec}$$

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_{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 - 2 50 m to 3 00 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 $H_{1/3}$ = 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 D L + 3 80 m Since h_1/H = 5 80/2 5 = 2 3 > 1 7, the case is "Breaking on Slope" Using H_c/H = 3 95/2 50 = 1 58 and H_r/H = 1 75/2 5 = 0 70, we obtain from Figs 11 and 12 , $q/q_o = 5 \times 10^{-4}$ and 2×10^{-4}, respectively

Since the tetrahedron blocks of two tons were used as the armor block of the rubble-mound, q/q_o is estimated approximately 3×10^{-4} from Fig 14 According to the experiments in which two high tides of D L + 3 80 m and D 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 D L + 3 80 m and only a few spray was observed for a tide of D 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 D L + 4 00 m and the average highest high tide for two hours was estimated approximately D L + 3 60 m The maximum wave height was assumed 2 50 m or 3 0m, 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

1 "Wave Run-up and Overtopping, Levee Sections, Lake Okeechobee, Florida", Corps of Engrs , Waterways Experiment Sta , Tech Report NO 2-449, January, 1959

2 Saville, Thorndike, "Wave Run-up on Shore Structures", Proc ASCE, Vol 82, NO WW2, April, 1956

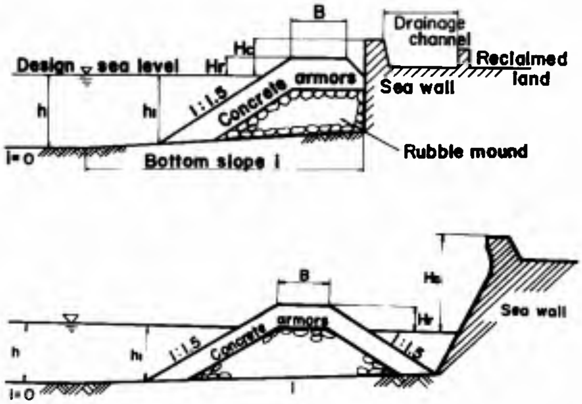

FIG. 1. - CROSS-SECTIONS TYPICAL TYPES OF SEA-WALL IN JAPAN
UPPER SECTION USED IN DEEPER WATERS
LOWER SECTION USED IN SHALLOW BEACH

FIG. 2. - WAVE CHANNEL WITH A WIND BLOWER

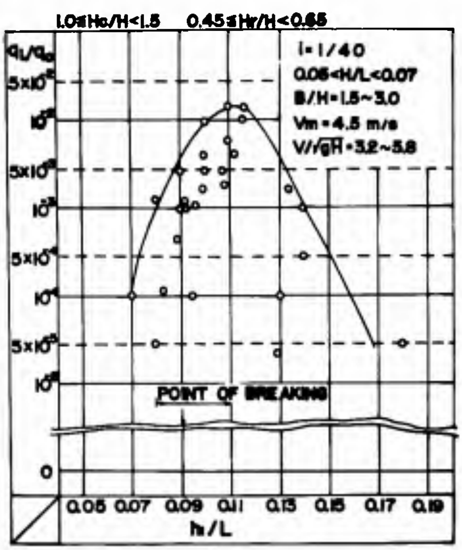

FIG. 3. - RELATIONSHIP BETWEEN RELATIVE OVERTOPPING AND h_1/L

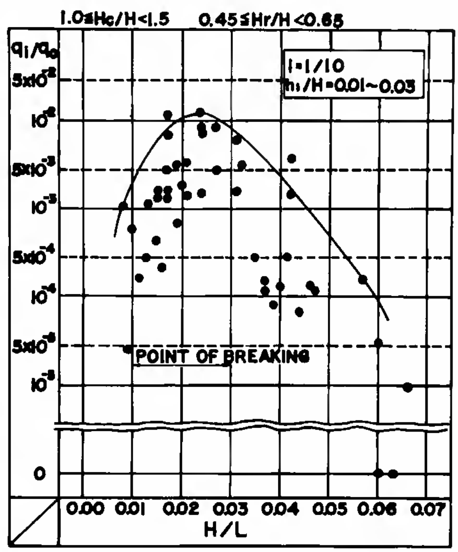

FIG. 4. - RELATIONSHIP BETWEEN q/q_0 AND H/L

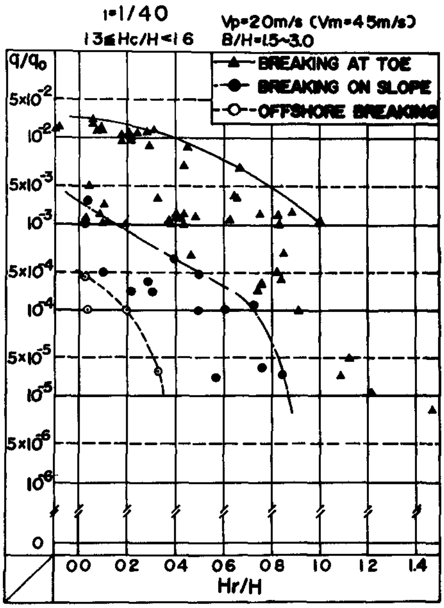

FIG 5 - RELATIONSHIP BETWEEN q/q_o AND H_r/H, $i = 1/40$

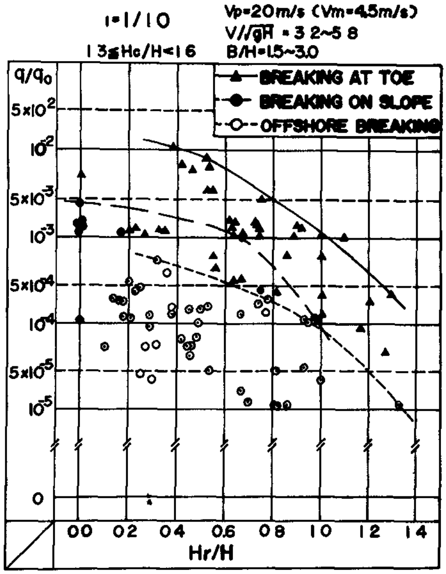

FIG 6 - RELATIONSHIP BETWEEN q/q_o AND H_r/H, $i = 1/10$

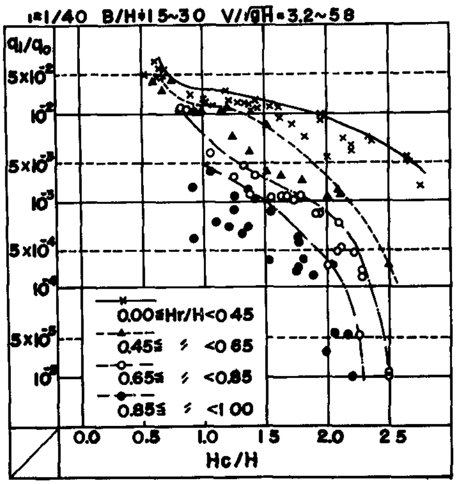

FIG 7 - RELATIONSHIP AMONG q/q_o, H_c/H AND H_r/H

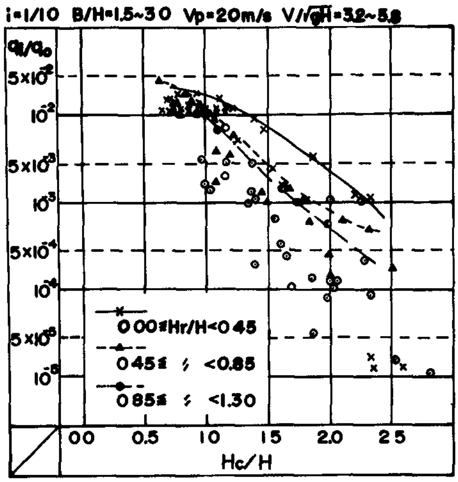

FIG 8 - RELATIONSHIP AMONG q/q_o, H_cH AND H_r/H

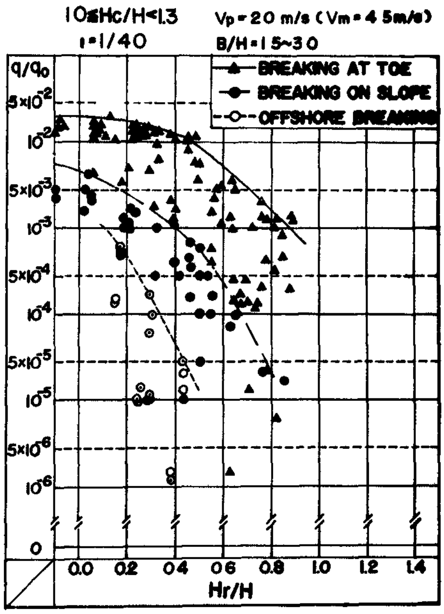

FIG 9 - RELATIONSHIP BETWEEN q/q_o AND H_r/H, $i = 1/40$

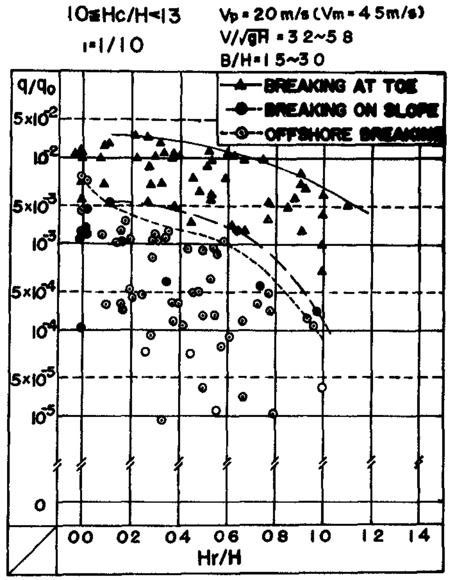

FIG 10 - RELATIONSHIP BETWEEN q/q_o AND H_r/H, $i = 1/10$

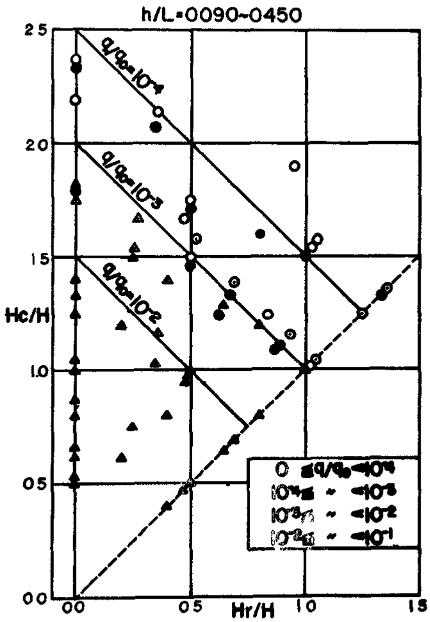

FIG 11 - RELATIONSHIP AMONG q/q_o, H_c/H, AND H_r/H FOR N_1-BLOCK

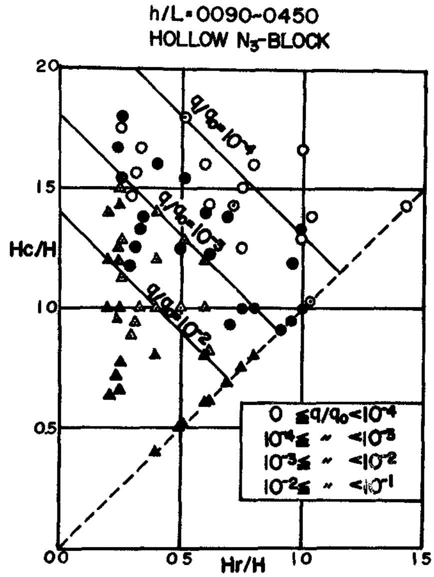

FIG 12 - RELATIONSHIP AMONG q/q_o, H_c/H, AND H_r/H FOR N_3-BLOCK

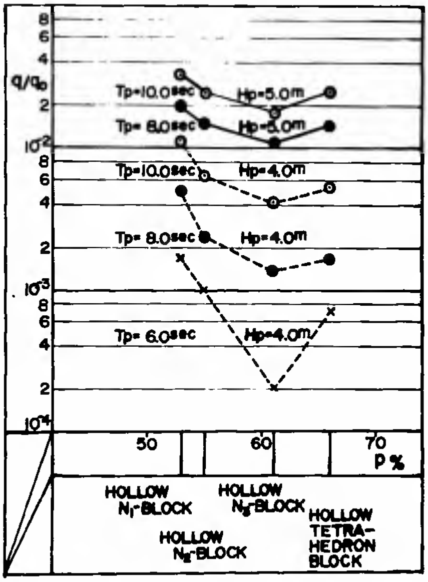

FIG. 14. - RELATIONSHIPS BETWEEN q/q_0 AND THE VOID RATIO AND SHAPE OF VOID OF ARMOR LAYORS

FIG. 13. - HOLLOW N-BLOCK

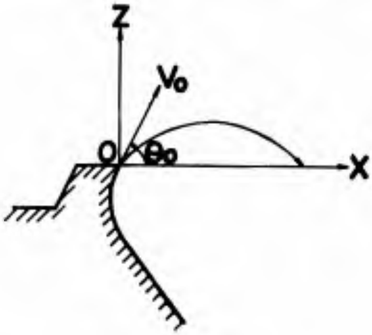

FIG. 15

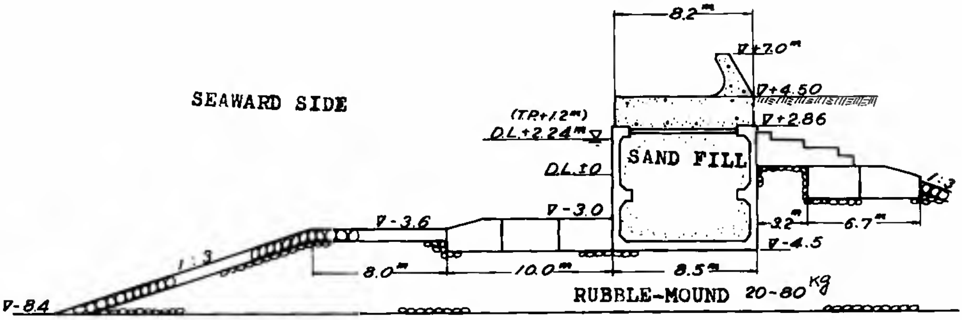

FIG. 16. - CROSS-SECTION OF THE SEA-WALL IN WAKAYAMA HARBOR BEFORE 1961

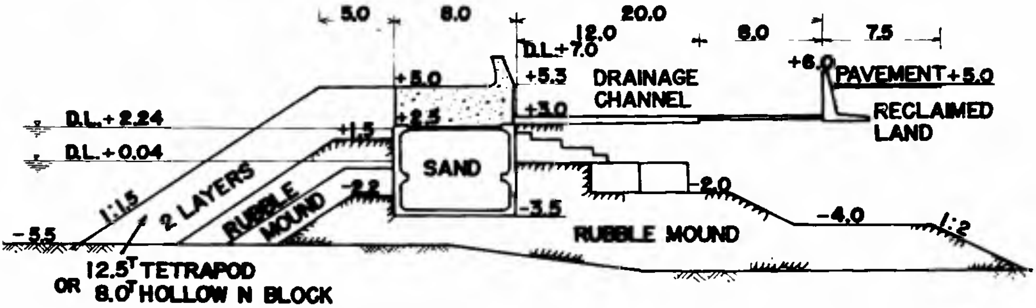

FIG. 19. - CROSS-SECTION OF THE SEA-WALL IMPROVED AFTER THE EXPERIMENTS

FIG. 17. - LARGE WAVE OVERTOPPINGS AT SEA-WALLS OF WAKAYAMA HARBOR

FIG. 18.

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

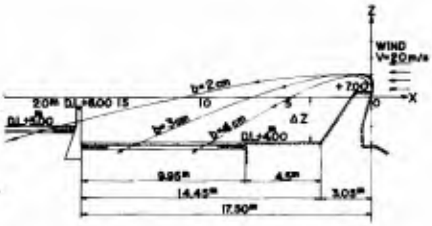

FIG. 21. - LOCI OF THE WALLS OF WATER OVERTOPPING THE SEA-WALL IN FIG. 20

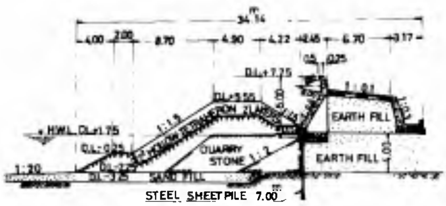

FIG. 22. - CROSS-SECTION OF THE SEA-WALL OF THE KANSAI ELECTRIC POWER CO.LTD

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