CHAPTER 159

WAVE FORCE ON BREAKWATERS WITH CONCRETE BLOCK MOUND

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ABSTRACT

Analytical theory is derived for prediction of the wave force acting on a upright structure armored by concrete block mound of rectangular shape. The theoretical treatment starts from the linear long wave theory and the flow resistance in the mound is described by a linearized friction law. As a practical application of the theory, approximate method for the structure with sloping block mound is proposed. Experiments are conducted to verify the theory and approximate method.

1. INTRODUCTION

Mound-type structures consisting of rubble stones and concrete blocks have been attracted many attentions as an porous structure protecting effectively a harbor from the action of rough seas. In the past decades, analytical and experimental studies have been conducted to understand the reflection and transmission characteristics of the porous mound structure(Madsen and White, 1975; Massel and Mei, 1977). Excellent review of the previous studies up to 1972 is given by Sollitt and Cross(1975).

In addition, it has been well known among coastal engineers that mound-type structures backed by a caisson or vertical seawall reduce considerably wave forces acting on the backed-up structure as well as wave run-up and overtopping(Goda, 1985). In Japan, such the mound structures comprising randomly placed artificial concrete blocks are very common and have been constructed to reduce sever wave actions against the structures. Figure 1 shows an idealized typical cross-section of such a block mound breakwater of composite-type (or Japanese-type)(PIANC, 1988).

When the block mound is used for this purpose, it is especially important for coastal engineers to estimate the effectiveness of a given block mound on the wave force

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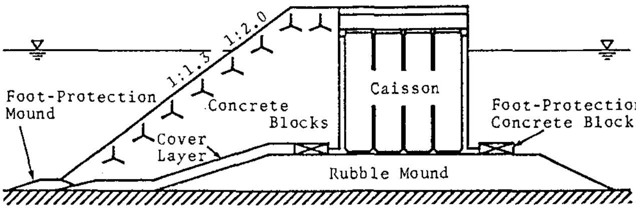

Fig. 1 Idealized typical section of a breakwater with concrete block mound.

reduction for a breakwater design. Incident wave energy is dissipated not only by wave breaking and frictional effect on the seaward slope of block mound, but also by resistance associated with oscillatory flows inside the mound. The interaction of incident waves with the block mound is a very intricate problem and relates various physical factors, such as wave properties, structural characteristics of the mound, and size and shape of concrete blocks.

Lack of adequate knowledge of these wave-structure interactions associated with the composite-type block mound breakwater has hindered from developing the analytical theory for predicting the wave force and reflection characteristics. As a result, laboratory model experiments have been recognized as one of possible and reliable approaches to the solution of such complicated problems. In Japan, more than 20 programs of laboratory experiments using regular and irregular waves have been made to deepen understanding on hydrodynamic behaviors of the composite-type structure with block mound , and to find a feasible standard design procedure for such structures. In spite of many intensive efforts, however, any empirical methods for predicting the wave force exerted on the backed-up structure have not been established yet with general confidence.

The principal aims of this study are as follows:

1. To develop an analytical theory for predicting the wave force acting on the caisson-type breakwater with rectangular block mound, and
2. To propose an approximate method for the estimate of wave force on the breakwater with sloping block mound.

An analytical solution for the present problem is derived under the following assumptions:

(1) Long waves are normally incident on the breakwater.
(2) The wave motion outside and inside the block mound can be described by linear long wave theory.
(3) The block mound structure with a rectangular cross section is homogeneous and isotropic.
(4) The flow resistance of the mound can be expressed by a linear relation in terms of the seepage velocity .

As Madsen and White discussed (1975), it is considered that wave conditions adopted in breakwater designs commonly falls in the long wave range of the spectrum. The linear long wave approximation, therefore, is used in the theoretical treatment.

An experimental program is carried out to confirm the validity of the theory and approximate method, and to determine empirical friction law.

2. THEORY

2.1 ANALYTICAL MODEL DERIVATION

The rectangular block mound with a width of l is located between $x = 0$ and $x = l$. The coordinate system is shown in Fig. 2. With the first and second assumptions, the governing equations for the wave motion outside the block mound ($x \leq 0$) are given as Eqs. (1) and (2).

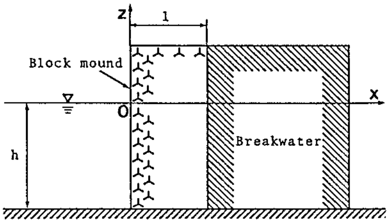

Fig. 2 Coordinate system.

$$(\partial \eta / \partial t) + h(\partial u / \partial x) = 0, \tag{1}$$

and

$$(\partial u / \partial t) + g(\partial \eta / \partial x) = 0, \tag{2}$$

in which η is the free surface elevation above the still water level, h is the constant water depth, u is the horizontal particle velocity, g is the gravitational acceleration, and t is the time.

Following to Kondo(1970) and Madsen and White(1975), the motion inside the block mound ($0 \leq x \leq l$) can be written as

$$(\partial \eta / \partial t) + (h/\epsilon)(\partial u / \partial t) = 0, \tag{3}$$

and

$$(S/\epsilon)(\partial u / \partial t) + g(\partial \eta / \partial x) + (f\omega/\epsilon)u = 0, \tag{4}$$

in which u is the seepage velocity, ϵ is the porosity of the mound, and ω is the angular frequency of the incident waves. The parameter S in the momentum equation (4) is given by Eq. (5) and represents effect of the unsteady fluid motion within the mound.

$$S = 1 + C_m(1 + \epsilon). \tag{5}$$

And f is the constant friction factor of the resistance law linearized using the Lorentz' Principle of equivalent work, and is written as

$$f\omega/\epsilon = \alpha + \beta|u|. \tag{6}$$

Two terms on the right hand of Eq. (6) represent laminar and turbulent flow resistance, respectively.

Since the governing equations are linear, we assume a periodic solution for the free surface elevation and horizontal particle velocity.

$$\eta(x,t) = \mathrm{Real}[\zeta(x)e^{i\omega t}], \tag{7}$$

and

$$u(x,t) = \mathrm{Real}[U(x)e^{i\omega t}], \tag{8}$$

in which $i = \sqrt{-1}$, and the amplitudes of η and u are functions of x only.

Outside the block mound, incident waves and partially reflected waves from the breakwater form a standing wave field. The flow inside the mound consists of two waves propagating in the positive and negative x-directions.

With the two matching conditions of the free surface elevation and particle velocity at the front surface of the block mound ($x=0$), and with the condition of no flow through the caisson($x=l$), the final expressions for the wave motions both outside and inside the block mound are obtained as follows:

For $x \leq 0$ (outside the block mound):

$$\eta(x,t) = (H_i/2)\{e^{-ik_0 x} + K_R e^{ik_0 x}\}e^{i\omega t}, \tag{9}$$

and

$$u(x,t) = U_i\{e^{-ik_0 x} - K_R e^{ik_0 x}\}e^{i\omega t} \tag{10}$$

For $0 \leq x \leq l$ (inside the mound):

$$\eta(x,t) = H_i \cosh\{ik(x-l)\}/\{\cosh(ikl) + \tau \sinh(ikl)\} \times e^{i\omega t}, \quad (11)$$

and

$$u(x,t) = -2U_i \tau \sinh\{ik(x-l)\}/\{\cosh(ikl) + \tau \sinh(ikl)\} \times e^{i\omega t}, \quad (12)$$

in which K_R is the reflection coefficient, as given by Eq.(13),

$$K_R = |H_r|/H_i = |\{1 - \tau \tanh(ikl)\}/\{1 + \tau \tanh(ikl)\}|. \quad (13)$$

The wave number k_0 and horizontal velocity amplitude of incident waves U_i are given by the long wave theory as

$$k_0 = 2\pi/L_i = \omega/\sqrt{gh}, \quad (14)$$

and

$$U_i = (H_i/2)\sqrt{g/h}. \quad (15)$$

k is the complex wave number of waves inside the mound, and is given by

$$k = k_0(S - if)^{1/2}, \quad (16)$$

and τ is the parameter for simplifying the theoretical expression.

$$\tau = \epsilon(S - if)^{-1/2}. \quad (17)$$

H_i(real) and H_r(complex) are the incident and reflected wave heights outside the block mound.

Since the wave number k is complex, the theory indicates that waves propagating to the positive x-direction through the block mound is attenuated exponentially due to the frictional effect of the mound. Thus, we presume that the wave is perfectly reflected at the vertical front surface of caisson, and the wave force is calculated by a linear wave theory.

With the linear long wave assumption, the maximum horizontal wave force per unit width of the caisson, F_{Hm}, is written by Eq. (18), a function of the maximum surface elevation at the caisson, η_{ml}.

$$F_{Hm} = (\rho g/2)[\, 2h\,\eta_{ml} + \eta_{ml}^2\,]. \quad (18)$$

By putting $x = l$ in Eq. (16), η_{ml} is written as

$$\eta_{ml} = \mathrm{Real}[H_i/\{\cosh(ikl) + \tau \sinh(ikl)\}]. \quad (19)$$

2.2 DISCUSSIONS ON THE THEORETICAL RESULTS

Practical calculations are usually made with the known mound geometry and porosity as well as the incident wave condition. Parameter S given by Eq. (5), depends not only on the shape and size of blocks, but also on the unsteady flow characteristics inside the mound. Measurements of the added mass coefficient C_m in Eq. (5) are so difficult that it is necessary to assign an appropriate value to the parameter S before discussing the theoretical results. Madsen and White

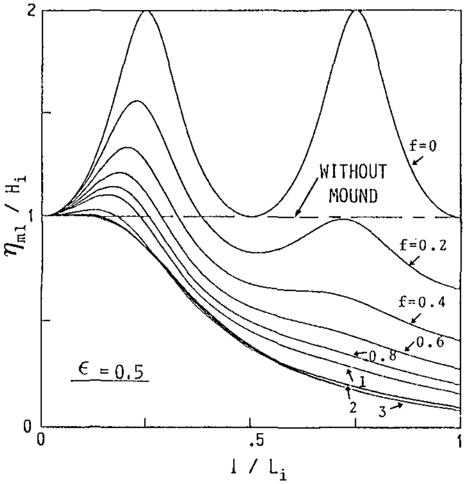

Fig. 3 Theoretical relation between η_{ml}/H_i and l/L_i.

(1975) discussed possible values of S and estimated to be $1 \leq S \leq 1.5$. They approximated it to be unity for practical circumstances. We shall, therefore, take S = 1.

The maximum wave force F_{Hm} is a function of η_{ml} only, so that dominant characteristics of the mound to the wave force reduction can be learned more clearly from the the general behavior of the solution of Eq. (19) for η_{ml}, in stead of Eq. (18) for F_{Hm}. An example of the numerical solution of Eq. (19) is presented in Fig. 3, showing the relative maximum surface elevation η_{ml}/H_i as a function of the relative mound width l/L_i for various values of the linearized friction factor f and for a mound porosity of $\epsilon = 0.5$. The broken line of $\eta_{ml}/H_i=1$ corresponds to the perfect reflection at a vertical breakwater without block mound.

When f = 0, it refers to as a fictitious mound, in which only the net flow cross section is reduced to a half of that seaside of the mound. From Eqs. (11) and (19), the maximum surface elevation for f=0 is given as

$$(\eta_{ml}/H_i)_{f=0} = [\cos^2 k_0 l + \epsilon^2 \sin^2 k_0 l]^{-1/2} \tag{20}$$

Cyclic change of η_{ml}/H_i with respect to l/L_i indicates that standing waves with a node and antinode at the common boundary of outside and inside the block mound are formed in conditions of l/L_i=0.25 and 0.50. With the increase of the friction factor f, the cyclic change is attenuated, and η_{ml}/H_i tends to approach constant values with increase of the relative mound width, when f>2. According to Eqs. (6) and (12), increase in the friction factor may be a result either of decreasing the mound porosity or of increasing the incident wave height (Sollitt and Cross, 1972). From Fig. 3, therefore, a very interest process is found that when small waves are incident on the mound breakwater, the block mound gives rise to stronger wave force than without the mound. This results from the two following facts: (1) decrease in the net flow cross section, and (2) the resonance of the standing wave systems between outside and inside the block mound.

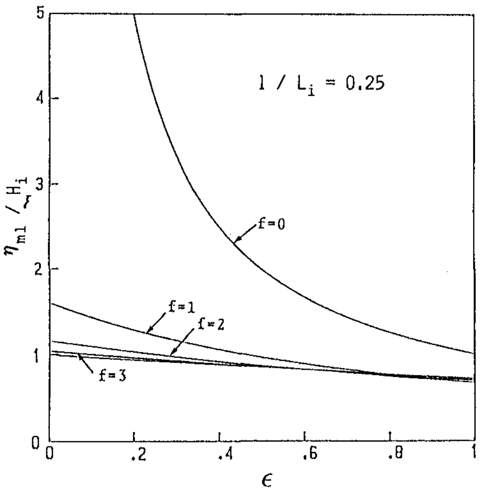

Fig. 4 Theoretical relation between η_{ml}/H_i and ϵ.

In a practical planning of coastal and harbor structures of block mound type, of a particular interest is the dependency of mound porosity on the wave force. Figure 4 is a numerical result showing the dependency of the mound porosity on the relative maximum free surface elevation η_{ml}/H_i for l/L_i=0.25. Since actual range of the mound porosity is estimated to be 40 % to 60 %, it is noticed that the amplitude of standing waves at the caisson or the wave force does not depend noticeably on the mound porosity for practical circumstances, in which the friction factor becomes usually larger than 2.

In addition to the wave force reduction, another important function of the block mound is the damping of reflected waves from the breakwater. Figure 5 is an numerical example of the relationship between the reflection coefficient K_R and relative mound width l/L_i for $\epsilon = 0.5$. As the relative mound width increases, the reflection coefficient tends to approach to a constant value for the case when the friction factor is larger than 2. Reduction in the wave reflection by the block mound can not be expected, if the relative width is smaller than 0.1.

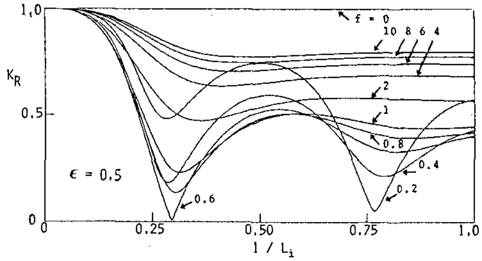

Fig. 5 Theoretical relation between K_R and l/L_i.

3. EXPERIMENTAL SETUP AND CONDITIONS

3.1 EXPERIMENTAL EQUIPMENT

Experiments were performed in a glass-walled wave flume of 1 m wide, 1 m deep and 50 m long. Figure 6 shows general arrangement of the experimental equipment. Waves were produced by a piston-type wave generator installed at one end of the wave flume. The wave flume was separated into two sections with a plywood bulkhead and equipped a steel beach of a slope of 1/50.

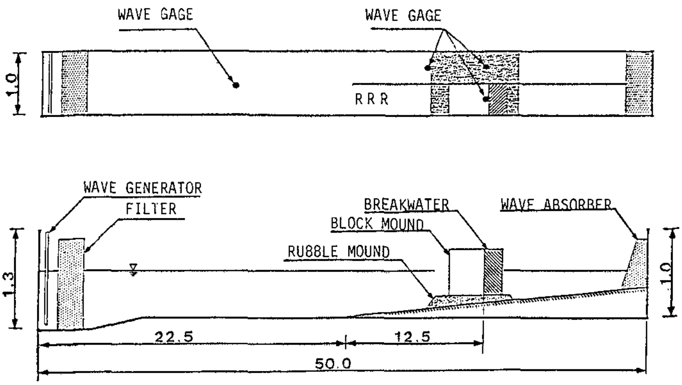

Fig. 6 General arrangement of experimental equipment. (Units:m)

In the test section with 0.50 m wide, model breakwaters comprising a steel rectangular parallelepiped as model caisson [0.466×m 0.20 m×0.40 m, weight in air: 20 kg] and rectangular or trapezoidal block mound was placed on a gravel mound of 5.0 cm high. The other section of the flume was used for measurements of incident waves as progressive waves at the breakwater location.

3.2 MODEL BLOCK MOUND

Rectangular model mounds with three different widths were comprised by model tetrapods of three different sizes, whose physical dimensions are described in Table 1. The block was contained in wire screen cribs, shaped as rectangular parallelepiped. In situ porosity was checked up by the total number of blocks used and volume of the crib.

Additional experiments were conducted to examine approximate method for predicting the wave force on the constructed with the same blocks used for the rectangular

Table 1 Dimensions of rectangular mounds(Tetrapods).

RELEVANT BLOCK SIZE*		B(cm)	5.0, 8.2, and 11.3
MOUND	WIDTH	l(cm)	22.5, 30.0, and 45.0
	HEIGHT	(cm)	40.0
IN SITU POROSITY		ϵ(%)	50.0

[*) The relevant block size is defined as $B = (abc)^{1/3}$, in which a, b, and c are long, intermediate and short sizes of a block]

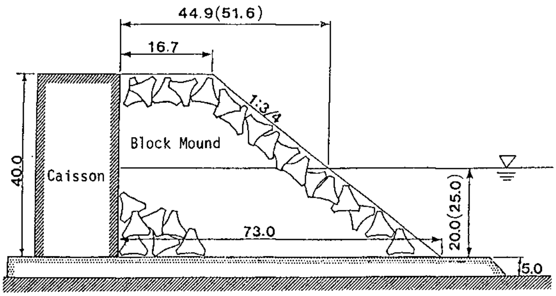

Fig. 7 Cross-section of sloping block mound(Units: cm).

mound. Figure 7 illustrates the dimensions of the trapezoidal block mound, constructed with the same blocks used for the rectangular model mound.

3.3 WAVE AND WAVE FORCE MEASUREMENT

Measurements of water free surface elevations at various locations, denoted in Fig. 6, were made using capacitance wave gages. In particular, simultaneous measurements of the free surface elevation at three offshore locations denoted by R in Fig. 6 were used for the computation of reflection coefficient of the breakwater. Resolution of incident and reflected waves was performed by means of the spectral component analysis with a simultaneous record of the free surface elevation at three adjacent locations of known relative distances (Goda, 1985).

Total horizontal wave forces acting on the model caisson were measured by using a specially designed force meter of cantilever-type, made of a steel square bar(40 mm×

70 mm × 1070 mm)(Hattori et al., 1979). The force meter was fixed on the backside of the caisson, and had a natural frequency of 13 Hz in a case when the caisson was placed in a water depth of 0.20 m. Outputs of the water surface elevation and wave force were recorded on a 7-channel analogue recorder. The data were digitized by an A-D converter at a sampling frequency of 100 Hz for extensive processing by computer.

Wave conditions are given in Table 2.

Table 2 Experimental conditions.

INCIDENT WAVE HEIGHT	H_i(cm)	1 to 17
PERIOD	T(s)	1.5 and 2.0
WATER DEPTH	h(cm)	20 and 25
BLOCK REYNOLDS NUMBER	$Re_B = BU_i/\nu$	$(1 \text{ to } 70)\times 10^3$

4. COMPARISONS OF THEORY AND EXPERIMENT

4.1 EMPIRICAL RELATIONSHIP FOR THE FRICTION FACTOR

The theoretical solutions indicate that the friction factor representing the hydraulic characteristics of the mound is a physically fundamental parameter for wave energy damping within the mound structures. Hence, the determination of the friction factor is a very important problem for the predictions of the wave force. To simplify the analytical treatment, the friction factor was assumed to be constant. In addition, the solutions were obtained by approximating the parameter S to be unity, because the added mass coefficient C_m of blocks could not be evaluated experimentally. It is, therefore, necessary to take into account of unsteady effect of the fluid motion in the determination of the friction factor.

In the previous studies, however, friction factor of rubble stones and concrete blocks were examined from experiments of steady and unidirectional flows in open channel or pipe (Shuto and Hashimoto, 1970; Sollitt and Cross, 1972). Based on some preliminary considerations and experiments, the friction factor was evaluated from measured maximum free surface elevation at the front of caisson breakwater, η_{m1}, with aids of Eqs. (11) and (16).

Since the friction factor is assumed to be independent of the time and space, we examine validity of the friction law of Eq. (6), in which the local seepage velocity u is replaced with the mean horizontal velocity inside the mound U_m, as given by Eq. (18).

$$U_m = (1/lT)\int_0^l\int_t^{t+T}|u|\,dx\,dt = (8/3\pi l)\int_0^l u_r\,dx, \qquad (18)$$

in which u_r is the real part of the seepage velocity given by Eq. (12). Substituting Eq. (9) into Eq. (18), U_m for a special case of f=0 is written as Eq. (19).

$$U_m = (2\epsilon U_i/k_0 l)[\cos^2 k_0 l + \epsilon^2 \sin^2 k_0 l]^{-1/2}\cdot[1-\cos k_0 l], \qquad (19)$$

Figure 8 shows relationships of $f\omega/\epsilon$ and U_m, and indicates that the linear friction law similar to Eq. (6) is valid for the mean seepage velocity. According to Eqs. (18) and (19), the mean seepage velocity U_m is proportional to the velocity amplitude of incident waves U_i and the mound

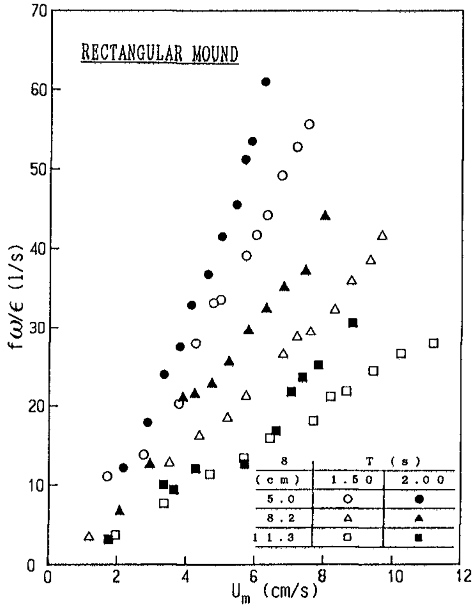

Fig. 8 Relationship between $f\omega/\epsilon$ and U_m.

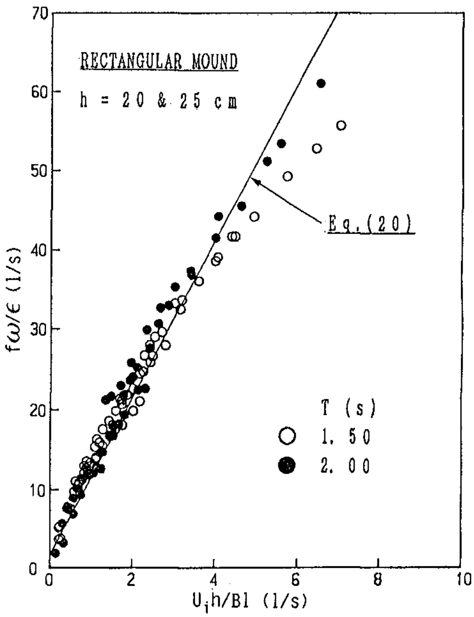

Fig. 9 Empirical friction law as a function of U_i.
[$f\omega/\epsilon$ and $U_i h/Bl$]

porosity ϵ , and is inversely proportional to the relative mound width l/L_i. Since U_m is generally a function of the friction factor, we can not explicitly calculate the friction factor from Eqs. (12) and (18) with a given incident wave condition and known mound properties.

From the standpoint of practical calculation of the friction factor, Fig. 9 is prepared for determining an empirical friction law as a function of the horizontal velocity amplitude of incident waves U_i, calculated from a linear wave theory. The best fitted line in Fig. 9 leads an empirical relationship for the friction factor of

$$f\omega/\epsilon = 2.0 + 10.4(U_i h/Bl), \qquad (20)$$

in which the factor (h/Bl) is introduced to describe the block mound characteristics.

4.2 REFLECTION COEFFICIENT

In order to examine applicability of the empirical relationship for the friction factor, experiments on the wave reflection were conducted. Figure 10 is an example of comparisons between the theoretical and experimental reflection coefficients K_R, with respect to the relative height of incident waves H_i/h for a relative mound width of l/L_i = 0.206. The friction factor is computed with the aid of Eq. (20). Although exhibiting some scatter of the data, the good agreement supports the determination of the friction factor adopted in this study, and validates the empirical equation (20).

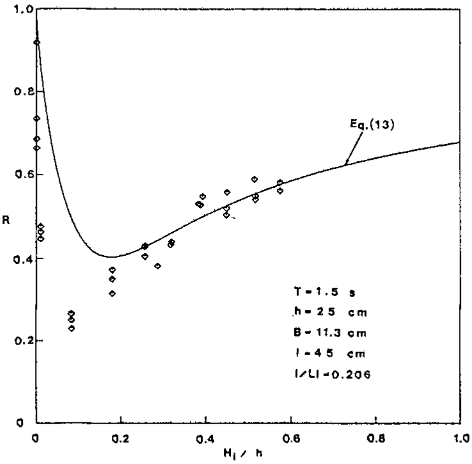

Fig. 10 Comparison between predicted and measured reflection coefficients.

4.3 MAXIMUM HORIZONTAL WAVE FORCE

As was mentioned previously, various factors influence on wave energy dissipation of the block mound. Hence, comparisons between experimental and predicted results of the maximum horizontal wave force on the upright caisson will be discussed by using the following dimensionless parameters, ϕ_m and K as given by Eqs. (21) and (22), which were derived from a dimensional analysis (Inagaki, Hattori, and Noguchi, 1986).

$$\phi_m = F_{mH} / \rho g H_i^2, \qquad (21)$$

and

$$K = [(1-\epsilon)/\epsilon]^{1/3}[(H_i l)/(hB)]\tanh k_0 h, \qquad (22)$$

where ϕ_m is the relative maximum wave force per unit width of breakwater, and K is the parameter combining the characteristics of an incident wave and block mound. For examples, the term of $[(1-\epsilon)/\epsilon]^{1/3}(l/B)$ on right hand of Eq. (22) represents the number of voids per unit width within

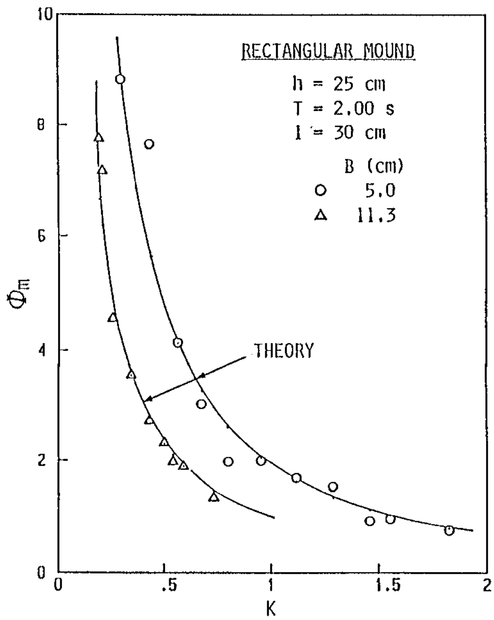

Fig. 11 Relationship between ϕ_m and K.

the block mound. And tanh k_0h is considered as a correction factor for the wave pressure distribution on the caisson.

Figure 11 is an experimental result on the relative maximum wave force ϕ_m as a function of K. For comparison the predictions by the present theory are also shown. And Figure 12 shows a comparison between measured and predicted maximum horizontal wave force for all the experiments. The agreement between the theory and experiments is fairy good and confirms the general validity of the present theory for the rectangular block mound breakwater.

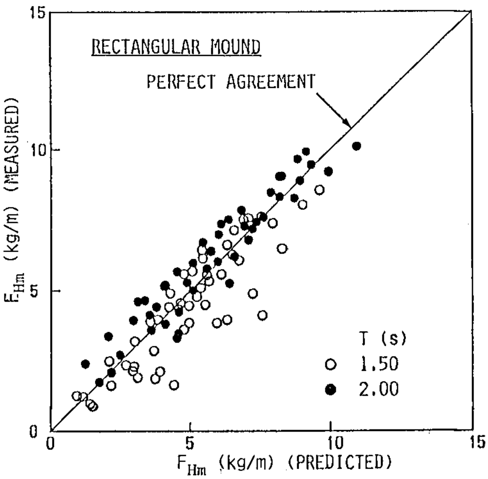

Fig. 12 Overall comparison between predicted and measured maximum wave forces.

5. APPROXIMATE METHOD FOR TRAPEZOIDAL BLOCK MOUND

5.1 PROPOSE OF APPROXIMATE METHOD

Wave motions at a block mound with steep front slope are very complicated and make the theoretical treatment more difficult than that for a rectangular block mound.

Sollitt and Cross (1972) proposed a concept of the equivalent rectangular breakwater which has the same submerged volume as that of the trapezoidal breakwater, and developed a semiempirical procedure for the prediction of reflection and transmission coefficients of permeable breakwater of trapezoidal shape. Hence, applicability of their approximate method to the present problem is examined by a comparison of all the measured maximum wave forces with the predictions based on the equivalent rectangular mound concept by Sollitt and Cross. In the predictions, energy dissipation due to the wave breaking is evaluated by the same procedure as by Sollitt and Cross. Figure 13 shows an overall comparison between predicted and measured maximum wave forces. We can clearly recognize a systematic trend that the predictions always underestimate the wave force and the deviation between the prediction and experiment becomes larger with increasing the maximum wave force, corresponds to the increase of incident wave height.

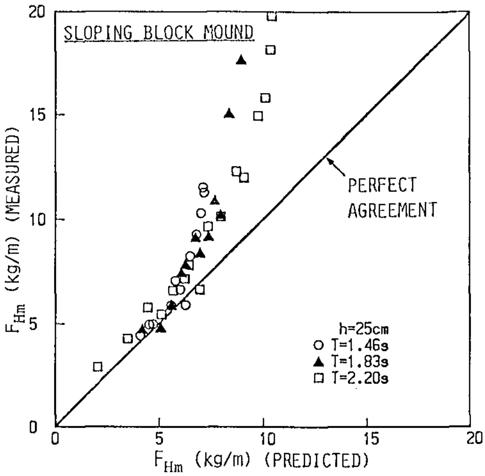

Fig. 13 Overall comparison between predicted and measured maximum wave forces.

According to experiments of trapezoidal block mounds, incident waves break and run-up on the sloping face of block mound. Water mass in the run-up wedge penetrates into the mound and flows through the upper portion of mound with narrower width than the submerged portion. As incident wave height increases, the water flow sometimes hit against the caisson and gives rise to additional wave force on the caisson. In addition, the water mass transport associated with the wave run-up on the sloping face rises the mean water level inside the mound above the still water level outside the mound. As a result, it spreads upwards the front surface area of caisson, to which wave forces are exerted.

Approximate method by Sollitt and Cross generally gives a wider mound width above the still water level than that of the actual mound. Thus, it does not reproduce reasonably well nonlinear effects of the wave motions observed both on the front slope and inside the block mound, and overestimates the energy dissipation due to flow resistance inside the mound. The experiments suggest that the following two effects of increasing the water depth around the block mound should be taken into account for the prediction of maximum wave force on a caisson backed by sloping block mound.

(1) Vertical asymmetry of the free surface elevation around the still water level associated with incident waves, and
(2) Wave set-up caused by the mass transport from the sloping face of the mound.

Consequently, we propose an approximate method that the sloping mound is replaced by an equivalent rectangular mound having the same width as that of the sloping mound at the mean water elevation including the wave set-up η_s and displacement of the mean water level from the still water level δ , as shown in Fig. 14. The mean water depth inside the block mound for determining the equivalent width of a sloping mound l_E is written as

$$h_E = h + \eta_s + \delta \quad , \tag{23}$$

and

$$\delta = \eta_c - H_i/2, \tag{24}$$

in which η_c is the crest elevation above the mean water level outside the mound and calculated by Cnoidal wave theory of first order(Isobe, 1985).

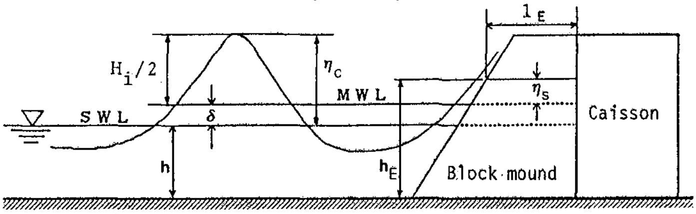

Fig. 14 Equivalent rectangular block mound.

5.2 WAVE SET-UP INSIDE BLOCK MOUND

Although many theoretical and experimental studies of the wave set-up on a natural beach have intensively been conducted, none of the published information has examined the wave set-up inside permeable structures. In order to examine the wave set-up inside the block mound, additional experiments both for the rectangular and sloping mound breakwaters were made. Deviation in the mean water level inside the mound from the still water level was evaluated from the time variation of free surface elevation measured at the front of the caisson.

Experimental results on the wave set-up inside the block mound are shown in Fig. 15, showing the relationship between the relative wave set-up η_s/H_i and the relative wave height H_i/h. Wave set-up inside the rectangular mound indicates a week dependency on incident wave height . On the contrary, as seen in Fig. 15, that inside the sloping

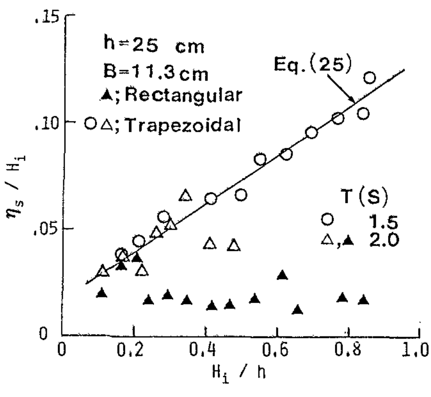

Fig. 15 Relation of η_s/H_i and H_i/h.

mound increases almost linearly with incident wave height, when $H_i/h > 0.1$. An empirical relationship between the wave set-up and incident wave height is obtained as

$$\eta_s/H_i = 0.02 + 0.11(H_i/h) \tag{25}$$

5.3 COMPARISONS BETWEEN PREDICTED AND MEASURED WAVE FORCE

Figure 16 shows an example of comparisons between predicted and measured maximum wave force, using the relationship between the nondimensional parameters, ϕ_m and K. From the fairly well agreements as seen in Fig. 16, it is concluded that the approximate procedure proposed here is applicable to the prediction of the wave force of a breakwater with sloping mound.

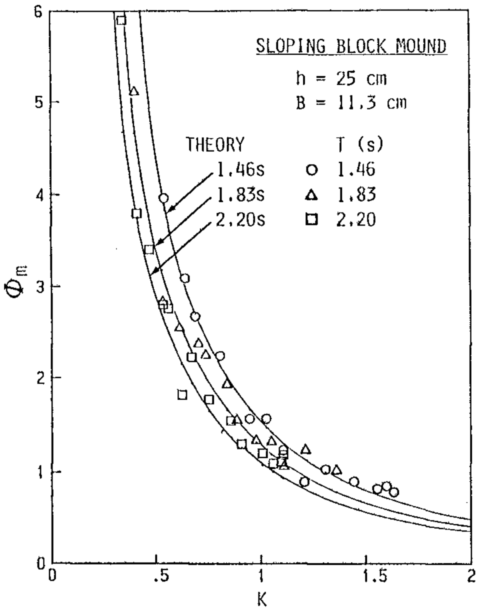

Fig. 16 Comparison between predicted and measured relative maximum wave forces ϕ_m.

To make both the analytical model and the approximate procedure generally applicable, the empirical equations for the friction law and for the wave set-up should be examined for various wave conditions and mound geometries.

6. CONCLUSIONS

In this paper, analytical theory are proposed for predicting the wave force acting on the upright caisson structure with a rectangular mound of concrete blocks of energy dissipation type. The theory is developed with approximation of the linear long wave. The wave damping inside the block mound is treated by the equivalent linearization technique after the Lorentz's Principle of equivalent work, and empirical formula is used. The validity of the analytical theory is confirmed from comparisons with laboratory experiments.

For practical applications, approximate procedure is developed for the prediction of the wave force on the caisson armored with sloping block mound. The sloping block mound problem is solved by introducing an equivalent rectangular mound which has the same mound width as that of the sloping mound at the mean water elevation taking account of two nonlinear effects, the vertical asymmetry of incident wave profile at the sloping face and wave set-up inside the mound.

Although a number of assumptions have been introduced in the approximate procedure, the good agreement between predicted and measured maximum wave forces indicates that the procedure may be valuable for practical applications.

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