THE DESIGN OF UPRIGHT BREAKWATERS

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ABSTRACT

The historical development of upright breakwaters in Japan is briefly reviewed as an introduction. Various wave pressure formulas for vertical walls are discussed, and then the design formulas currently employed in Japan are presented with an example of calculation. Several design factors are also discussed.

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1. INTRODUCTION

An upright breakwater is defined here as a structure having an upright section rested upon a foundation. It is often called a vertical breakwater or composite breakwater. The former is sometimes referred to a structure directly built on the rock foundation without layers of rubble stones. The latter on the other hand means a breakwater functioning as a sloping-type structure when the tide level is low but as a vertical-wall structure when the tide level is high. Because the terminology may vary from person to person, the definition above is given here in order to avoid further confusion.

Upright breakwaters are of quite old structural type. Old ports in the Roman Empire or ports in even older periods had been provided with breakwaters with upright structures. The upright breakwaters of recent construction have the origin in the 19th century. Italian ports have many upright breakwaters as discussed in the following lecture by Dr. L. Franco. British ports also have a tradition of upright breakwater construction as exemplified in Dover Port. The British tradition can be observed in old breakwaters of Indian ports such as Karachi, Bombay, and Madras. Japanese ports owes this tradition of upright breakwaters to British ports, because the modern breakwater construction began at Yokohama Port in 1890 under supervision of British army engineer, retired Major General H. S. Palmer. Since then Japan has built a large number of upright breakwaters along her long coastline extending over 34,000 km. The total length of upright breakwaters in Japan would exceed several hundred kilometers, as the total extension of breakwaters is more than 1,000 km.

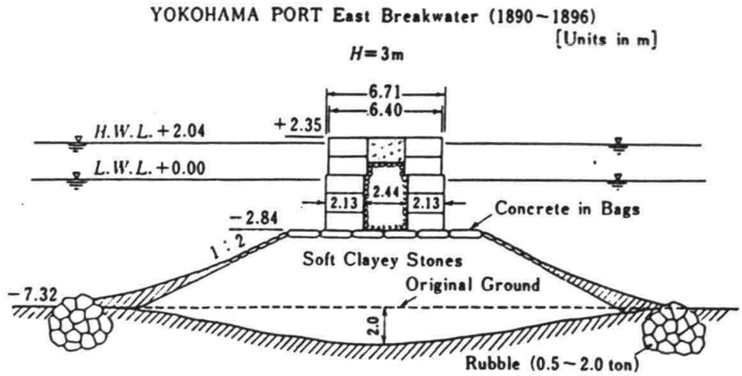
The present note is intended to introduce the engineering practice of upright breakwater design to coastal and harbor engineers in the world, based on the experience of Japanese engineers.

2. HISTORICAL DEVELOPMENT OF UPRIGHT BREAKWATERS IN JAPAN

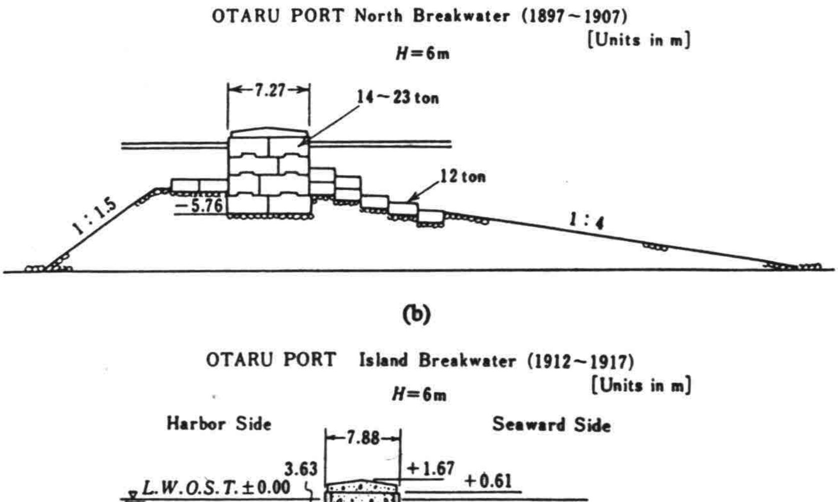
2.1 Examples of Upright Breakwaters in Modern History of Japanese Ports

Figure 1 illustrates typical cross sections of upright breakwaters in Japan in time sequences, which is taken from Goda [1985]. The east breakwater of Yokohama Port in Fig. 1 (a) utilized the local material of soft clayey stones for rubble foundation and minimized the use of concrete blocks in the upright section. The stone-filled middle section was replaced by concrete blocks fully during reconstruction after the storm damage in 1902. The wave condition in Yokohama was not severe with the design height of 3 m.

The structural type of upright breakwaters was adopted at a more exposed location of Otaru Port as shown in Fig. 1 (b) by I. Hiroi in 1897, who was the chief engineer of regional government, later became a professor of the Tokyo Imperial University, and established the framework of Japanese harbor engineering. The first reinforced-concrete caisson breakwater in Japan was built at Kobe in 1911, based on the successful construction of caisson-type quaywall



(a)



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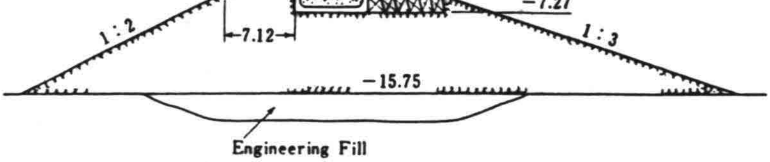
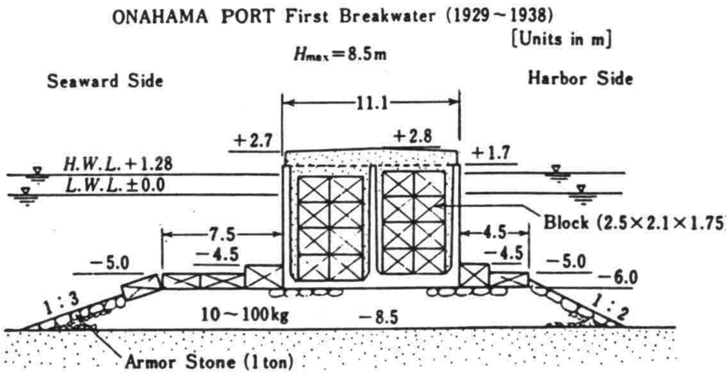


(c)

Fig. 1 (a-c) Historical development of upright breakwater in Japan after Goda [1985].



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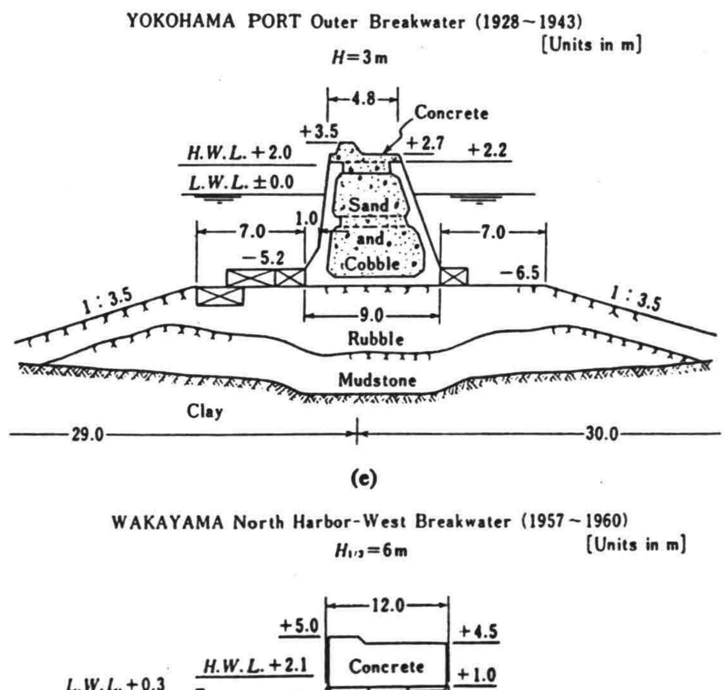

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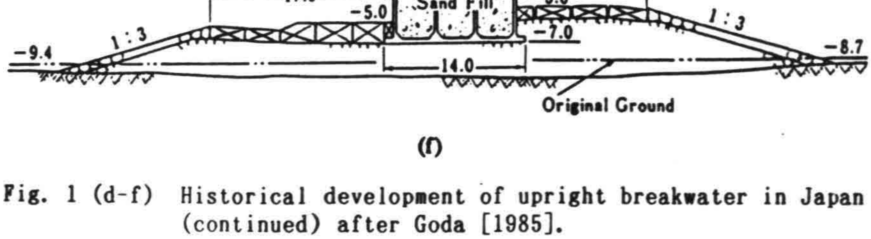
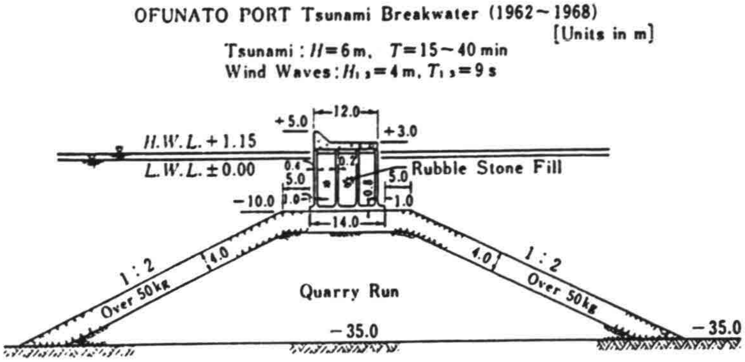


(f)

Fig. 1 (d-f) Historical development of upright breakwater in Japan (continued) after Goda [1985].



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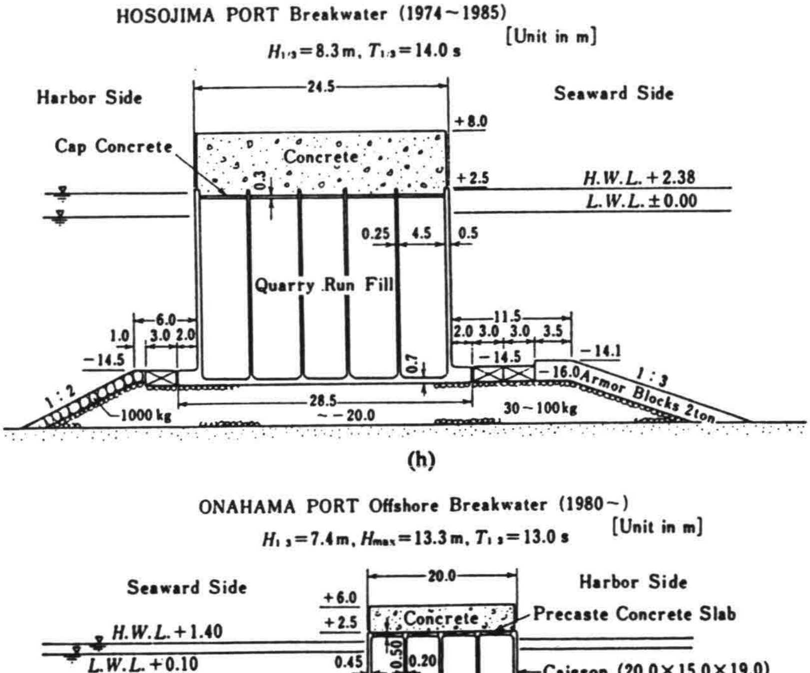

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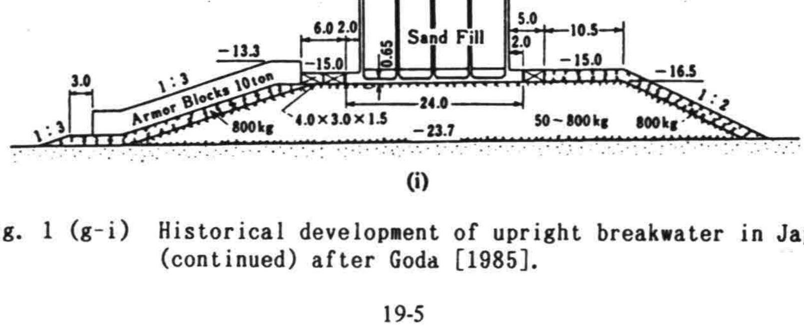

(i)

Fig. 1 (g-i) Historical development of upright breakwater in Japan (continued) after Goda [1985].

at Rotterdam in 1905. Then Hiroi, immediately seeing the bright future of caisson breakwaters, employed the concept to an island breakwater of Otaru Port shown in Fig. 1 (c), where the design wave was 6 m high. He carried out various field measurements, including wave pressures on a vertical wall, for his finalization of breakwater design. Through these efforts, he came to propose the wave pressure formula for breakwater design, which is to be discussed in the next section.

Hiroi's breakwater caissons were filled with concrete for durability and stability. The work time for concrete placement was sometimes saved by the use of precast blocks as in the example of Onahama Port in Fig. 1 (d). Concrete filling of breakwater caisson had been a tradition before the end of World War II, but a pioneering construction of reinforced concrete caisson breakwater with sand filling was carried out in Yokohama Port during the period of 1928 to 1943: Fig. 1 (e) shows its cross section. After World War II the use of sand as the filler material of caisson cells gradually became a common practice in Japan.

The breakwater of Wakayama Port shown in Fig. 1 (f) was built upon a quite soft ground so that it was provided with a wide foundation for the purpose of counter-balancing the weight of upright section. The breakwater of Ofunato Port in Fig. 1 (g) was built to reduce the inflow of tsunami waves into the bay. The water depth of 35 m below the datum level was the deepest one at the time of construction in 1962, but the present record of the deepest breakwater in Japan is held at Kamaishi Port with the depth of 60 m. Some design features and wave pressures on this breakwater have been discussed by Tanimoto and Goda [1991b]. One of the widest breakwaters is that of Hosojima Port shown in Fig. 1 (h): the widest at present is found at Hedono Port in a remote island with the width 38m (see Tanimoto and Goda 1991a). The breakwater of Onahama Port shown in Fig. 1 (i) is of recent design using Goda's wave pressure formulas to be discussed later.

2.2 Some Features of Japanese Upright Breakwaters

As seen in these examples, Japanese breakwaters of upright type have a few common features. One is the relatively low crest elevation above the high water level. Presently, the recommendation for ordinary breakwaters is the crest height of 0.6 $H_{1/3}$ above the high water level for the design condition. For the design storm condition, this elevation is certainly insufficient to prevent wave agitations by the overtopped waves. However, it is a way of thinking of harbor engineers in Japan that the design waves are accompanied by strong gale and storm winds in any case and safe mooring of large vessels within a limited area of harbor basin cannot be guaranteed even if wave agitations are reduced minimum. As the storm waves with the return period of one year or less are much lower than the design wave, the above crest elevation is thought to be sufficient for maintaining a harbor basin calm at the ordinary stormy conditions.

Another feature of Japanese upright breakwaters is a relatively wide berm of rubble foundation and provision of two to three rows of large foot (toe) protection blocks. There is no fixed rule for selection of the berm width and engineers always consult with the examples of existing breakwaters in the neighborhood or those at the location of similar wave conditions. It is somewhat proportional to the size of concrete caisson itself, but the final decision must await good judgment of the engineer in charge. The foot protection concrete blocks have the size ranging from 2 to 4 m in one direction and the height of 1.5 to 2 m, weighing 15 to 50 tf. Though these blocks used to be solid ones, recent blocks are provided with several vertical holes to reduce the uplift force and thus to increase the stability against wave action.

A new development in upright breakwaters of Japan is the employment of various modifications to the shape of concrete caissons, such as perforated walls, vertical slits, curved slits with circular arc members, dual cylindrical walls and others (see Tanimoto and Goda 1991a). These new caisson shapes have been developed to actively dissipate wave energy and thus to reduce wave reflection and wave pressures. A number of these breakwaters have been built and functioning as expected.

3. REVIEW OF WAVE PRESSURE FORMULAS FOR VERTICAL WALL

3.1 Hiroi's Formula

Prof. Hiroi published the wave pressure formula for breakwater design in 1919. It is a quite simple formula with the uniform pressure distribution of the following intensity:

$$p = 1.5\ w_0\ H \qquad (1)$$

where w_0 denotes the specific weight of sea water and H the incident wave height. This pressure distribution extends to the elevation of $1.25\ H$ above the design water level or the crest of breakwater if the latter is lower, as shown in Fig. 2.

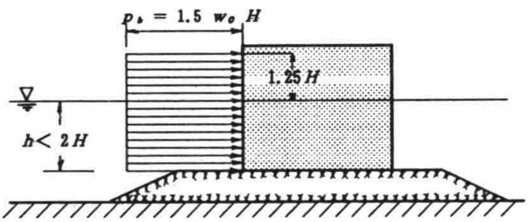

Fig. 2 Wave pressure distribution by Hiroi's formula.

Prof. Hiroi explained the phenomenon of wave pressure exerted upon a vertical wall as the momentum force of impinging jet flow of breaking waves and gave the reasoning for its quantitative evaluation. However, he must have had some good judgment on the magnitude of wave pressure from his long experience of harbor construction and several efforts of pressure measurements in situ. He states that he obtained the records of wave pressure exceeding 50 tf/m² by the pressure gauges set at a concrete wall in water of several meters deep. Nevertheless, he did not incorporate such high pressures into the formula of breakwater design, by saying that the high wave pressure must have lasted for only a short duration and are ineffective to cause appreciable damage to breakwaters.

Hiroi's wave pressure formula was intended for use in relatively shallow water where breaking waves are the governing factor. He also recommended to assume the wave height being 90% of water depth if no reliable information is available on the design wave condition. Hiroi's wave pressure formula was soon accepted by harbor engineers in Japan, and almost all breakwaters in Japan had been designed by this formula till the mid-1980s.

The reliability of Hiroi's formula had been challenged thrice at least. The first challenge was the introduction of Sainflou's formula in 1928 for standing wave pressures. Differentiation of two formulas was made, by referring to the recommendation of PIANC in 1935, in such a way that Hiroi's formula was for the case of the water depth above the rubble foundation being less than twice the incident wave height, while Sainflou's formula was for the water depth equal to or greater than twice the wave height. The second challenge was raised when the concept of significant wave was introduced in early 1950s. Which one of H_{max} , $H_{1/10}$, or $H_{1/3}$ is to be used in Hiroi's formula was the question. A consensus was soon formed as the recommendation for the use of $H_{1/3}$ based on the examination of existing breakwater designs and wave conditions. The third challenge was made by Goda [1973] against the insensitivity of the estimated pressure intensity to the variations in wave period and other factors. Hiroi's formula could not meet this challenge and is not used presently for the design of major breakwaters.

Though the pressure formula by Hiroi was so simple, the total wave force thus estimated was quite reliable on the average. Thanks to this characteristic, Japanese breakwaters had rarely experienced catastrophic damage despite the very long extension around the country.

3.2 Sainflou's Formula

As well known, Saiflou published a theory of trochoidal waves in front of a vertical wall in 1928 and presented a simplified formula for pressure estimation. The pressure distribution is sketched as in Fig. 3, and the pressure intensities and the quantity of water level rise δ_0 are given as

$$\left.\begin{aligned} p_1 &= (p_2 + w_0 h)\ (H + \delta_0) / (h + H + \delta_0) \\ p_2 &= w_0 H / \cosh kh \\ \delta_0 &= (\pi H^2 / L) \coth kh \end{aligned}\right\} \qquad (2)$$

where L is the wavelength and k is the wavenumber of $2\pi/L$.

Sainflou [1928] presented the above formula for standing wave pressures of nonbreaking type and the formula has been so utilized. The formula was derived for the purpose of practical application from the standpoint of a civil engineer and it has served its objective quite well. Just like the case of Hiroi's formula, it was born when the concept of wave irregularity was unknown. There seems to exist no established rule for the choice of representative wave height to be used with Sainflou's formula. Some advocates the use of $H_{1/3}$, some favors $H_{1/10}$, and the other prefers the selection of $H_{1\%}$.

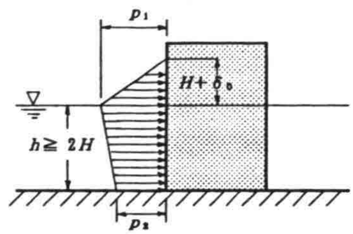

Fig. 3 Wave pressure distribution by Sainflou's formula.

It was customarily in Japan to use $H_{1/3}$ with Sainflou's formula but in a modified form. Through examinations of several minor damage of breakwaters, it had been revealed that a simple application of Sainflou's formula had yielded underestimation of wave pressures under storm conditions. For the zone extending $\pm H/2$ around the design water level, the wave pressure by Sainflou's foumula was replaced with that by Hiroi's formula. The modified formula was sometimes called the partial breaking wave pressure formula in Japan, because it was aimed to introduce the effect of partial wave breaking in relatively deep water. The dual system of Hiroi's wave pressure formula for breaking waves and of modified Sainflou's formula for standing waves had been the recommended engineering practice of breakwater design in Japan for the period from around 1940 to the early 1980s.

3.3 Minikin's Formula and Others

Although Hiroi's formula had been regarded as the most dependable formula for breaking wave pressures in Japan, it remained unknown in Europe and America. As the field measurement at Dieppe revealed the existence of very high pressures caused by impinging breaking waves and the phenomenon was confirmed by laboratory experiments by Bagnold [1939], harbor engineers in western countries began to worry about the impact breaking wave pressures. Then in 1950, Minikin proposed the following formula for breaking wave pressures, which consisted of the dynamic pressure p_m and the hydrostatic pressure p_s as sketched in Fig. 4:

Dynamic pressure :

$$\left.\begin{aligned} p_m &= p_{max}\,(1 - 2|z|/H)^2 \quad &: |z| \leqq H/2 \\ p_{max} &= 101\, w_0\, d\,(1+d/h)\, H/L & \end{aligned}\right\} \qquad (3)$$

Hydrostatic pressure :

$$p_s = \begin{cases} 0.5\, w_0\, H\,(1 - 2z/H) & : 0 \leqq z < H/2 \\ 0.5\, w_0\, H & : z < 0 \end{cases} \qquad (4)$$

Because it was the first descriptive formula for breaking wave pressures, it was immediately accredited as the design formula and listed in many textbook and engineering manuals. Even in present days, technical papers based on Minikin's formula are published in professional journals from time to time.

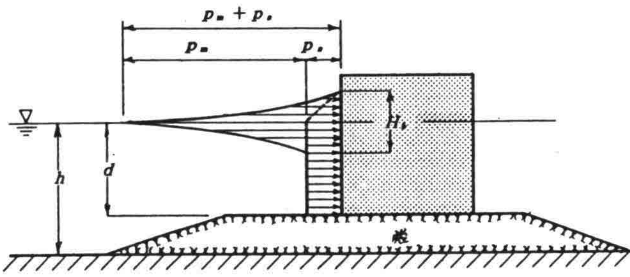

Fig. 4 Wave pressure distribution by Minikin's formula.

Minikin [1950] did not give any explanation how he derived the above formulation except for citing the experiments of Bagnold. In the light of present knowledge on the nature of impact breaking wave pressures, the formula has several contradictory characteristics. First, the maximum intensity of wave pressure increases as the wave steepness increases, but the laboratory data indicates that waves with long periodicity tends to generate well developed plunging breakers and produce the impact pressure of high intensity. In fact, Bagnold carried out his experiments using a solitary wave.

Second, Eq. 3 yields the highest p_{max} when d is equal to h or when no rubble foundation is present. It is harbor engineers' experience that a breakwater with a high rubble mound has a larger possibility of being hitten by strong breaking wave pressures than a breakwater with a low rubble mound.

Third, Minikin's formula yields excessively large wave force against which no rational upright breakwater could be designed. To the author's knowledge, no prototype breakwater has ever been constructed with the wave pressures estimated by Minikin's formula. Reanalysis of the stability of prototype breakwaters in Japan which experienced storm waves of high intensity, some undamaged and others having been displaced over a few meters, has shown that the safety factor against sliding widely varies in the range between 0.31 and 2.06 [Goda 1973b and 1974]. The safety factors of undamaged and displaced breakwaters were totally mixed together and no separation was possible. Thus the applicability of Minikin's formula on prototype breakwater design has been denied definitely.

There has been several proposals of wave pressure formulas for breakwater design. Among them, those by Nagai [1968, 1969]] and Nagai and Otusbo [1968] are most exhaustive. Nagai classified the various patterns of wave pressures according to the wave conditions and the geometry of breakwater, and presented several sets of design formuilas based on many laboratory data. However, his system of wave pressure formulas was quite complicated and these formulas gave different prediction of wave pressures at the boundaries between the zones of their applications. Another problem in the use of Nagai's method is the lack of specification for represtantaive wave height for irregular waves. There was only a few cases of verification of the applicability of his method for breakwater design using the performance data of prototype breakwaters. Because of these reasons, the method is not used in Japan presently.

The Miche-Rundgren formula for standing wave pressure [CERC 1984] represents an effort to improve the accuracy of Sainflou's formula for engineering application. Certainly, the formula would give better agreement with the laboratory data than Sainflou's one. However, it has not been verified with any field data and its applicability for brekwater design is not confirmed yet.

4. DESIGN FORMULAS OF WAVE PRESSURES FOR UPRIGHT BREAKWATERS

4.1 Proposal of Universal Wave Pressure Formulas

It is a traditional approach in wave pressure calculation to treat the phenomena of the standing wave pressures and those by breaking waves separately. Casual observations of wave forms in front of a vertical wall could lead to a belief that breaking wave pressures are much more intensive than nonbreaking wave pressures and they should be calculated differently. The previous practice of wave pressure calculation with the dual formulas of Hiroi's and Sainflou's in Japan was based on such belief. The popularity of Minikin's formula prevailing in western countries seems to be owing to the concept of separation of breaking and nonbreaking wave pressures.

The difference between the magnitudes of breaking and nonbreaking wave pressures is a misleading one. The absolute magnitude of breaking wave pressures is certainly much larger than that of nonbreaking one. The height of waves which break in front of a vertical wall, however, is also greater than that of nonbreaking waves. The dimensionless pressure intensity, $p / w_0 H$, therefore, increases only gradually with the increase of incident wave height beyond the wave breaking limit, as demonstrated in the extensive laboratory data by Goda [1972].

A practical inconvenience in breakwater design with the dual pressure formula system is evident when a breakwater is extended offshoreward over a long distance from the shoreline. While the site of construction is in shallow water, the wave pressures are evaluated with the breaking wave pressure formula. In the deeper portion, the breakwater would be subject to nonbreaking waves. Somewhere in between, the wave pressure formula must be switched from that of breaking to nonbreaking one. At the switching section, the estimated wave pressures jump from one level to another. With the Japanese system of the combined formulas of Hiroi's and modified Sainflou's, the jump was about 30%. To be exact with the pressure calculation, the width of upright section must be changed also. However, it is against the intuition of harbor engineers who believe in smooth variation of the design section. The location of switching section is also variable, dependent on the design wave height. If the design wave height is modified by a review of storm wave conditions after an experience of some damage on the breakwater, then an appreciable length of breakwater section would have to be redesigned and reconstructed.

The first proposal of universal wave pressure formula for upright breakwater was made by Ito et al. [1966] based on the sliding test of a model section of breakwaters under irregular wave actions. Then Goda [1973b, 1974] presented another set of formulas based on extensive laboratory data and being supported by verification with 21 cases of breakwater displacement and 13 cases of no damage under severe storm conditions. The proposed formulas were critically reviewed by the corps of engineers in charge of port and harbor construction in Japan, and they were finally adopted as the recommended formulas for upright breakwater design in Japan in 1980, instead of the previous dual formulas of Hiroi's and modified Sainflou's.

4.2 Design Wave

The upright breakwater should be designed against the greatest force of single wave expected during its service life. The greatest force would be exerted by the highest wave among a train of random waves corresponding to the design condition on the average. Thus the wave pressure formulas presented herein are to be used together with the highest wave to be discussed below.

(1) Wave height

$$H_{max} = \begin{cases} 1.8\, H_{1/3} & : h/L_0 \geqq 0.2 \\ \min\{(\beta_0^* H_0' + \beta_1^* h),\ \beta_{max}^* H_0',\ 1.8\, H_{1/3}\} & : h/L_0 < 0.2 \end{cases} \tag{5}$$

$$H_{1/3} = \begin{cases} K_s H_0' & : h/L_0 \geqq 0.2 \\ \min\{(\beta_0 H_0' + \beta_1 h),\ \beta_{max} H_0',\ K_s H_0'\} & : h/L_0 < 0.2 \end{cases} \tag{6}$$

in which the symbol $\min\{a, b, c\}$ stands for the minimum value among a, b and c, and H_0' denotes the equivalent deepwater significant height. The coefficients β_0 and others have empirically been formulated from the numerical calculation data of random wave breaking in shallow water as follows, after Goda [1975]:

$$\left.\begin{aligned} \beta_0 &= 0.028\,(H_0'/L_0)^{-0.38} \exp[20\tan^{1.5}\theta] \\ \beta_1 &= 0.52 \exp[4.2\tan\theta] \\ \beta_{max} &= \max\{0.92,\ 0.32\,(H_0'/L_0)^{-0.29} \exp[2.4\tan\theta]\} \end{aligned}\right\} \tag{7}$$

$$\left.\begin{aligned} \beta_0^* &= 0.052\,(H_0'/L_0)^{-0.38} \exp[20\tan^{1.5}\theta] \\ \beta_1^* &= 0.63 \exp[3.8\tan\theta] \\ \beta_{max}^* &= \max\{1.65,\ 0.53\,(H_0'/L_0)^{-0.29} \exp[2.4\tan\theta]\} \end{aligned}\right\} \tag{8}$$

in which the symbol $\max\{a, b\}$ stands for the larger of a or b, and $\tan\theta$ denotes the inclination of sea bottom.

The shoaling coefficient K_s is evaluated by taking the finite amplitude effect into consideration. Figure 5 has been prepared for this purpose based on the theory of Shuto [1974].

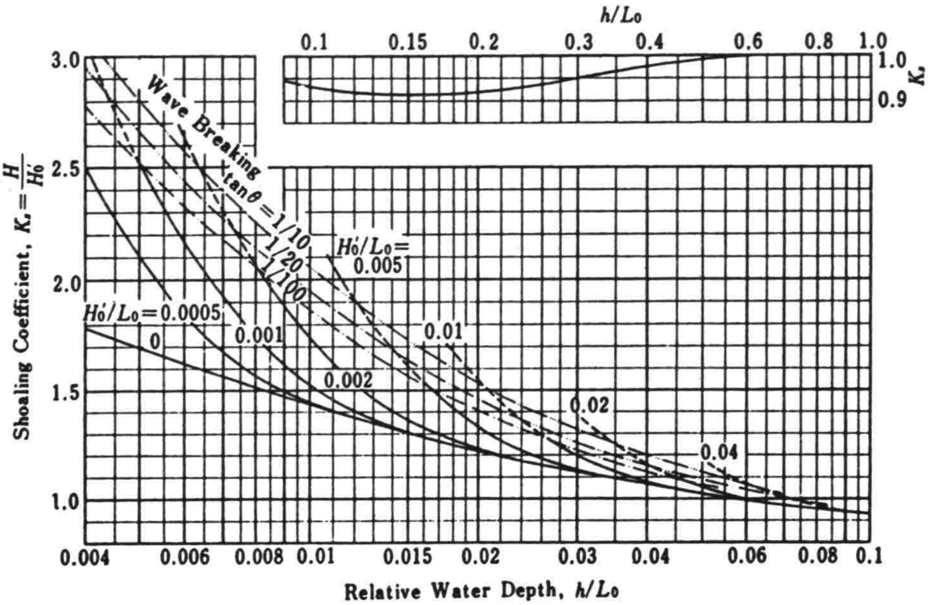

Fig. 5 Diagram of nonlinear wave shoaling coefficient K_s.

The selection of the fixed relation $H_{max} = 1.8\ H_{1/3}$ outside the surf zone was based on three factors of reasonining. First, the fixed ratio was preferred to an introduction of duration-dependent relation based on the Rayleigh distribution of wave heights, because such variability in the design wave height would cause some confusion in design procedures. Second, the examination of prototype breakwater performance under severe storm wave actions yielded reasonable results of safety factor against sliding by using the above fixed relation. Third, a possible deviation of the ratio $H_{max} / H_{1/3}$ from 1.8 to 2.0, say, corresponds to an increase of 11% and it can be covered within the margin of safety factor which is customarily taken at 1.2. However, it is a recommendation and an engineer in charge of breakwater design can use other criterion by his own judgment.

For evaluation of H_{max} by the second part of Eq. 7 or within the surf zone, the water depth at a distance 5 $H_{1/3}$ seaward of the breakwater should be employed. This adjustment of water depth has been introduced to simulate the nature of breaking wave force which becomes the greatest at some distance shoreward of the breaking point. For a breakwater to be built at the site of steep sea bottom, the location shift for wave height evaluation by the distance 5 $H_{1/3}$ produces an appreciable increase in the magnitude of wave force and the resultant widening of upright section.

(2) Wave Period

The period of the highest wave is taken as the same with the significant wave period of design wave, *i.e.*,

$$T_{max} = T_{1/3} \tag{9}$$

The relation of Eq. 9 is valid as the ensemble mean of irregular waves. Though individual wave records exhibit quite large deviations from this relation, the use of Eq. 9 is recommended for breakwater design for the sake of simplicity.

(3) Angle of Wave Incidence to Breakwater

Waves of oblique incidence to a breakwater exert the wave pressure smaller than that by waves of normal incidence, especially when waves are breaking. The incidence angle β is measured as that between the direction of wave approach and a line normal to the breakwater. It is recommended to rotate the wave direction by an amount of up to 15° toward the line normal to the breakwater from the principal wave direction. The recommendation was originally given by Prof. Hiroi together with his wave pressure formula, in consideration of the uncertainty in the estimation of wave direction, which is essentially based on the 16 points-bearing of wind direction.

4.3 Wave Pressure, Buoyancy, and Uplift Pressure

(1) Elevation to which the the wave pressure is exerted

The exact elevation of wave crest along a vertical wall is difficult to assess because it varies considerably from $1.0H$ to more than $2.0H$, depending on the wave steepness and the relative water depth. In order to provide a consistency in wave pressure calculation, however, it was set as in the following simple formula:

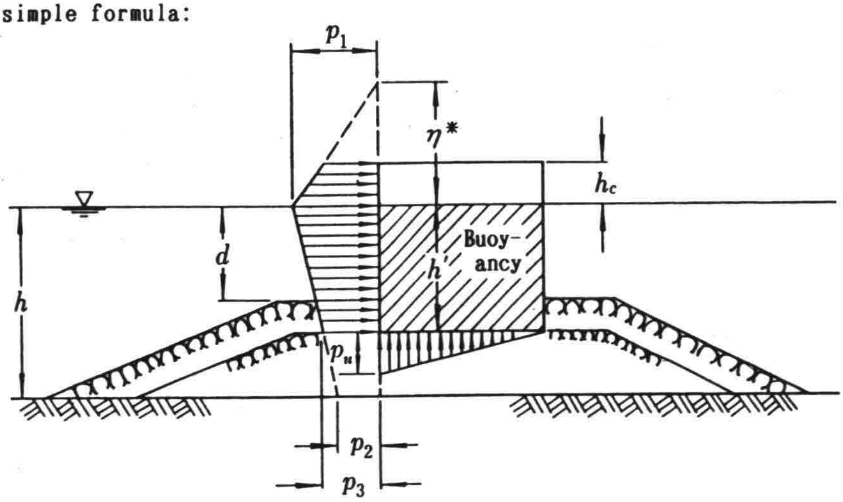

Fig. 6 Wave pressure distribution by Goda's formulas.

$$\eta^* = 0.75\,(1 + \cos\beta)\,H_{max} \tag{10}$$

For waves of normal incidence, Eq. 10 gives the elevation of $\eta^* = 1.5\,H_{max}$.

(2) Wave pressure exerted upon the front face of a vertical wall

The distribution of wave pressure on an upright section is sketched in Fig. 6. The wave pressure takes the largest intensity p_1 at the design water level and decreases linearly towards the elevation η^* and the sea bottom, at which the wave pressure intensity is designated as p_2.

The intensities of wave pressures are calculated by the following:

$$\left.\begin{aligned} p_1 &= 0.5\,(1 + \cos\beta)\,(\alpha_1 + \alpha_2\cos^2\beta)\,w_0\,H_{max} \\ p_2 &= p_1/\cosh kh \\ p_3 &= \alpha_3\,p_1 \end{aligned}\right\} \tag{11}$$

in which

$$\left.\begin{aligned} \alpha_1 &= 0.6 + 0.5\,[2kh/\sinh 2kh]^2 \\ \alpha_2 &= \min\{[(h_b - d)/3h_b](H_{max}/d)^2,\ 2d/H_{max}\} \\ \alpha_3 &= 1 - (h'/h)\,[1 - 1/\cosh kh] \end{aligned}\right\} \tag{12}$$

where h_b denotes the water depth at the location at a distance $5H_{1/3}$ seaward of the breakwater.

The coefficient α_1 takes the minimum value 0.6 for deepwater waves and the maximum value 1.1 for waves in very shallow water. It represents the effect of wave period on wave pressure intensities. The coefficient α_2 is introduced to express an increase of wave pressure intensities by the presence of rubble mound foundation. Both coefficients α_1 and α_2 have empirically been formulated, based on the data of laboratory experiments on wave pressures. The coefficient α_3 is derived by the relation of linear pressure distribution. The above pressure intensities are assumed to remain the same even if wave overtopping takes place.

The effect of the incident wave angle on wave pressures is incorporated in η^* and p_1 with the factor of $0.5\,(1 + \cos\beta)$ and a modification to the term of α_2 with the factor of $\cos^2\beta$.

(3) Buoyancy and uplift pressure

The upright section is subject to the buoyancy corresponding to its displacement volume in still water below the design water level. The uplift pressure acts at the bottom of the upright section, and its distribution is assumed to have a triangular distribution with the toe pressure p_u given by Eq. 13.

$$p_u = 0.5\,(1 + \cos\beta)\,\alpha_1\,\alpha_3\,w_0\,H_{max} \tag{13}$$

The toe pressure p_u is set smaller than the wave pressure p_3 at the lowest point of the front wall. This artifice has been introduced to improve the accuracy of the prediction of breakwater stability, because the verification with the data of prototype breakwater performance indicated some overestimation of wave force if p_u were taken the same with p_3.

When the crest elevation of breakwater h_c is lower than η^*, waves are regarded to overtop the breakwater. Both the buoyancy and the uplift pressure, however, are assumed to be unaffected by wave overtopping.

4.4 Stability Analysis

The stability of an upright breakwater against wave action is examined for the three modes of failure: *i.e.*, sliding, overturning, and collapse of foundation. For the first two modes, the calculation of safety factor is a common practice of examination. The safety factors against sliding and overturning are defined by the following:

$$\text{Against sliding :} \quad S.F. = \mu \, (W - U) \, / P \tag{14}$$

$$\text{Against overturning :} \quad S.F. = (Wt - M_U) \, / M_P \tag{15}$$

The notations in the above equations are defined as follows:

- M_P : moment of total wave pressure around the heel of upright section
- M_U : moment of total uplift pressure around the heel of upright section
- P : total thrust of wave pressure per unit extension of upright section
- t : horizontal distance between the center of gravity and the heel of upright section
- U : total uplift pressure per unit extension of upright section
- W : weight of upright section per unit extension in still water
- μ : coefficient of friction between the upright section and the rubble mound

The safety factors against sliding and overturning are dictated to be equal to or greater than 1.2 in Japan. The friction coefficient between concrete and rubble stones is usually taken as 0.6. The coefficient seems to have a smaller value in the initial phase of breakwater installment, but it gradually rises to the value around 0.6 through consolidation of the rubble mound by the oscillations of the upright section under wave actions. The fact that most of breakwater displacements by storm waves occur during the construction period or within a few years after construction supports the above conjecture.

The bearing capacity of the rubble mound and the sea bottom foundation was used to be examined with the bearing pressures at the heel of upright section and at the interface between the rubble mound and the foundation. However, a recent practice in Japan is to make analysis of circular slips passing through the rubble mound and the foundation, by utilizing the simplified Bishop method (see Kobayashi et al. 1987). For the rubble mound, the apparent cohesion of $c = 2$ tf/m^2 and the angle of internal friction of $\phi = 35°$ are recommended.

4.5 Example of Wave Pressure Calculation

An example of calculation is given here in order to facilitate the understanding of the breakwater design procedure. The design wave and site conditions are set as in the following:

Waves:	$H_0' = 7.0$ m,	$T_{1/3} = 11$ s,	$\beta = 10°$	
Depth etc.:	$h = 18$ m,	$d = 10$ m,	$h' = 11.5$ m,	$h_c = 4.5$ m
Bottom slope:	$\tan\theta = 1/50$			

The incident wave angle is the value after rotation by the amount up to 15°. The geometry of upright breakwater is illustrated in Fig. 7.

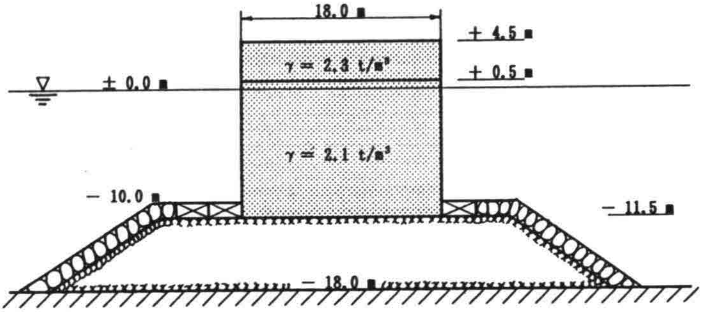

Fig.7 Sketch of upright breakwater for stability analysis.

i) Design wave height H_{max} and the maximum elevation of wave pressure η^*

The coefficients for wave height calculation are evaluated as

$$L_0 = 188.8 \text{ m}, \quad H_0'/L_0 = 0.0371, \quad h/L_0 = 0.0953, \quad K_s = 0.94$$
$$\beta_0 = 0.1036, \quad \beta_1 = 0.566, \quad \beta_{max} = \min\{0.92, 0.84\} = 0.92$$
$$\beta_0^* = 0.1924, \quad \beta_1^* = 0.680, \quad \beta_{max}^* = \min\{1.65, 1.39\} = 1.65$$

Then, the wave heights and the maximum elevation are obtained as

$$H_{1/3} = \min\{10.91, 6.44, 6.58\} = 6.44 \text{ m}$$
$$h_b = 18.0 + 5 \times 6.44/50 = 18.64 \text{ m}$$
$$H_{max} = \min\{14.02, 11.55, 11.84\} = 11.55 \text{ m}$$
$$\eta^* = 0.75 \times (1 + \cos 10°) \times 11.55 = 17.19 \text{ m}$$

ii) Pressure components

The wavelength at the depth 18 m is $L = 131.5$ m. The coefficients for wave pressure are evaluated as

$$kh \doteq 2\pi \times 18/131.5 = 0.860$$
$$\alpha_1 = 0.6 + 0.5 \times [2 \times 0.860/\sinh(2 \times 0.860)]^2 = 0.802$$

$$\alpha_2 = \min\{[(18.64-10.0)/(3\times 18.64)]\times(11.55/10)^2,\ 2\times 10/11.55\}$$
$$= \min\{0.206,\ 1.732\} = 0.206$$
$$\alpha_3 = 1-11.5/18.0\times[1-1/\cosh(0.860)] = 0.820$$

Then, the intensities of wave pressure and uplift pressure are calculated as

$$p_1 = 0.5\times(1+0.9848)\times[0.802+0.206\times(0.9848)^2]\times 1.03\times 11.55$$
$$= 11.83\ \mathrm{tf/m^2}$$
$$p_2 = 11.83/\cosh(0.860) = 8.49\ \mathrm{tf/m^2}$$
$$p_3 = 0.820\times 11.83 = 9.70\ \mathrm{tf/m^2}$$
$$p_4 = 11.83\times(1-4.5/17.19) = 8.73\ \mathrm{tf/m^2}$$
$$p_u = 0.5\times(1+0.9848)\times 0.802\times 0.820\times 1.03\times 11.55 = 7.76\ \mathrm{tf/m^2}$$

The symbol p_4 denotes the pressure intensity at the top of upright section.

iii) Total pressure and uplift, and their moments

$$P = 0.5\times(11.83+9.70)\times 11.5+0.5\times(11.83+7.76)\times 4.5 = 167.9\ \mathrm{tf/m}$$
$$M_P = 1366.2\ \mathrm{tf\text{-}m/m}$$

$$U = 0.5\times 18.0\times 7.76 = 69.8\ \mathrm{tf/m}$$
$$M_U = (2/3)\times 69.8\times 18 = 837.6\ \mathrm{tf\text{-}m/m}$$

iv) Stability of upright section against wave action

The specific weight of upright section is assumed as in the following:

The portion above the elevation +0.5 m : $\gamma_c = 2.3\ \mathrm{tf/m^3}$
The portion below the elevation +0.5 m : $\gamma_c' = 2.1\ \mathrm{tf/m^3}$

The difference in the specific weight reflects a current practice of sand filling in the cells of concrete caisson. The weight of upright section is calculated for the dry and in situ conditions, respectively, as

$$W_a = 2.1\times(11.5+0.5)\times 18.0+2.3\times(4.5-0.5)\times 18.0 = 619.2\ \mathrm{tf/m}$$
$$W = 619.2-1.03\times 11.5\times 18.0 = 406.0\ \mathrm{tf/m}$$

The safety factors against sliding and overturning of the upright section are calculated as in the following:

Against sliding: $S.F. = 0.6\times(406.0-69.8)/167.9 = 1.20$
Against overturning: $S.F. = (406.0\times 9.0-837.6)/1366.2 = 2.06$

Therefore, the upright breakwater with the uniform width of $B = 18.0$ m sketched in Fig. 7 is considered stable against the design wave of $H_0' = 7.0$ m and $T_{1/3} = 11.0$ s.

5. DISCUSSION OF SEVERAL DESIGN FACTORS

5.1 Precautions against Impulsive Breaking Wave Pressure

The universal wave pressure formulas described hereinbefore do not address to the problem of impulsive breaking wave pressure in a direct manner. The coefficient α_2, however, has the characteristic of rapid increase with the decrease of the ratio d/H_{max}. This increase roughly reflects the generation of impulsive breaking wave pressure.

Though the impact pressure of breaking waves exerted upon a vertical wall is much feared by coastal and harbor engineers, it occurs under the limited conditions only. If waves are obliquely incident to a breakwater, the possibility of impact pressure generation is slim. If a rubble mound is low, the sea bottom should be steep and waves be of swell type for the impact pressure to be generated. A most probable situation under which the impact pressure is exerted upon an upright breakwater is the case with a high rubble mound with an appreciable berm width (see Tanimoto et al. 1987). Most of breakwater failures attributed to the action of the impulsive breaking wave pressure are due to the wave forces of normal magnitude, which could be estimated by the universal wave pressure formulas described in the present lecture note.

The impact pressure of breaking waves last for a very short time duration, which is inversely proportional to the peak pressure intensity. In other words, the impulse of impact pressure is finite and equal to the forward momentum of advancing wave crest which is lost by the contact with the vertical wall. The author has given an estimate of the average value of the impact pressure effective in causing sliding of an upright section, by taking into account the elastic nature of a rubble mound and foundation [Goda 1973a]. Because the major part of impact is absorbed by the horizontal oscillations and rotational motion of the upright section, the impact pressure effective for sliding is evaluated as $(2\sim3)\ w_0 H_{max}$.

Nevertheless, the above order of pressure intensity is too great to be taken into the design of upright breakwaters: the mean intensity of wave pressure employed for the stability analysis of the breakwater sketched in Fig. 7 is only $0.91\ w_0 H_{max}$. Engineers in charge of breakwater design should arrange the layout and the cross section of breakwater in such way to avoid the danger of impact pressure generation. If the exertion of impulsive breaking wave pressure on the upright section seems inevitable, a change in the type of breakwater structure, such as a sloping-type breakwater or a vertical breakwater protected by a mound of concrete blocks, should be considered.

5.2 Structural Aspects of Reinforced Concrete Caisson

The upright section of vertical breakwater is nowadays made by reinforced concrete caisson. The width is determined by the stability condition against wave action. The height of caisson or the base elevation is so chosen to yield the minimum sum of the construction cost of rubble mound and upright section.

The length of caisson is governed by the capacity of manufacturing yard. In March 1992, Kochi Port facing the Pacific in Shikoku, Japan, set a breakwater caisson with the length 100 m in position. It is of hybrid structure with steel frames and prestressed concrete.

A concrete caisson is divided into a number of inner cells. The size of inner cells is limited to 5 m or less in ordinary design. The outer wall is 40 to 50 cm thick, the partition wall 20 to 25 cm thick, and the bottom slab 50 to 70 cm thick. These dimensions are subject to the stress analysis of reinforced concrete. As the upright breakwater withstands the wave force mainly with its own weight, the use of prestressed concrete for breakwater caisson is not advantageous in the ordinary situations. For the caisson of special shapes for enhancing wave dissipation such as the caisson with circular arc members, prestressed concrete is utilized.

5.3 Armor Units for Rubble Mound

The berm and slope of a rubble mound needs to be protected with armor units against the scouring by wave action. Foot-protection blocks weighing from 15 to 50 tf are placed in front of an upright section. The rest of the berm and slope are covered by heavy stones and/or specially-shaped concrete blocks. The selection of armor units is left to the judgment of engineers, with the aid of hydraulic model tests if necessary.

A formula for the weight of armor stones on the berm of rubble mound has been proposed by Tanimoto et al. [1982] as the results of systematic model tests with irregular waves. The minimum weight of armor stones can be calculated by a formula of the Hudson type:

$$W = \gamma_r H_{1/3}^{3} / [N_s^{3} (S_r - 1)^{3}] \qquad (16)$$

in which W is the weight of armor stones, γ_r the specific weight of armor stones, S_r the ratio of γ_r to the specific weight of seawater, and N_s the stability number, the value of which depends on the wave conditions and mound dimensions.

For waves of normal incidence, Tanimoto et al. [1982] gave the following function for armor stones:

$$N_s = \max \{1.8, \ [1.3 \frac{1-\kappa}{\kappa^{1/3}} \frac{h'}{H_{1/3}} + 1.8 \exp [-1.5 \frac{(1-\kappa)^2}{\kappa^{1/3}} \frac{h'}{H_{1/3}}]] \} \qquad (17)$$

in which the parameter κ is calculated by

$$\kappa = [2kh'/\sinh 2kh'] \sin^2 (2\pi B_M / L') \qquad (18)$$

and where h' denotes the water depth at which armor stones are placed, L' the wavelength at the depth h', and B_M the berm width.

Though the stability number for concrete blocks has not been formulated, a similar approach to the data of hydraulic model tests on concrete blocks will enable the formulation of the stability number for respective types of concrete blocks.

6. CONCLUDING REMARKS

The design and construction of upright breakwaters is a well established, engineering practice, at least in Japan, Korea, and Taiwan. A large number of these breakwaters have been built and will be built to protect ports and harbors. In these countries, the problem of impulsive breaking wave pressure is rather lightly dealt with. The tradition owes to Prof. Hiroi, who established the most reliable wave pressure formula in shallow water and showed the upright breakwaters could be successfully constructed against breaking waves.

This is not to say that no breakwaters have failed by the attack of storm waves. Whenever a big storm hits the coastal area, several reports of breakwater damage are heard. However, the number of damaged caissons is very small compared with the total number of breakwater caissons installed along the whole coastline. Probably the average rate per year would be less than 1%, though no exact statistic is available. Most cases of breakwater damage are attributed to the underestimation of the storm wave condition when they were designed.

In the past, the majority of breakwaters were constructed in relatively shallow water with the depth up to 15 m, for example, because the vessels calling ports were relatively small. In such shallow water, the storm wave height is controlled by the breaking limit of the water depth. One reason for the low rate of breakwater failure in the past could be this wave height limitation at the locations of breakwaters.

The site of breakwater construction is moving into the deeper water in these days. Reliable evaluation of the extreme wave condition is becoming the most important task in harbor engineering, probably much more than the improvement of the accuracy of wave pressure prediction.

REFERENCES

Bagnold, R.A. [1939]: Interim report on wave-pressure research, *J. Inst. Civil Engrs.*, Vol.12, pp.202-226.

CERC (Coastal Engineering Research Center, U.S. Army Corps of Engrs.) [1984]: *Shore Protection Manual*, U.S. Government Printing Office, pp.7-161～173.

Goda, Y. [1972]: Experiments on the transition from nonbreaking to postbreaking wave pressures, *Coastal Engineering in Japan*, Vol.15, pp.81-90.

Goda, Y. [1973a]: Motion of composite breakwater on elastic foundation under the action of impulsive breaking wave pressure, *Rept. Port and Harbor Res. Inst.*, Vol.12, No.3, pp.3-29 (*in Japanese*).

Goda, Y. [1973b]: A new method of wave pressure calculation for the design of composite breakwater, *Rept. Port and Harbor Res. Inst.*, Vol. 12, No. 3, pp.31-70 (*in Japanese*).

Goda, Y. [1974]: New wave pressure formulae for composite breakwater, *Proc. 14th Int. Conf. Coastal Eng.*, pp.1702-1720.

Goda, Y. [1975]: Irregular wave deformation in the surf zone, *Coastal Engineering in Japan*, Vol.18, pp.13-26.

Goda, Y. [1985]: *Random Seas and Design of Maritime Structures* , University of Tokyo Press., pp.108-110.

Hiroi, I. [1919]: On a method of estimating the force of waves, *Memoirs of Engg. Faculty, Imperial University of Tokyo*, Vol. X, No.1, p.19.

Ito, Y., Fujishima, M., and Kitatani, T. [1966]: On the stability of breakwaters, *Rept. Port and Harbor Res. Inst.*, Vol. 5, No. 14, pp.1-134 (*in Japanese*).

Kobayashi, M., Terashi, M., and Takahashi, K. [1987]: Bearing capacity of a rubble mound supporting a gravity structure, *Rept. Port and Harbor Res. Inst.*, Vol.26, No.5, pp.215-252.

Minikin, R. R. [1950]: *Winds, waves and Maritime Structures,* Griffin, London, pp.38-39.

Nagai, S. [1968]: Pressures of partial standing waves, *J. Waterways and Harbors Div., Proc. ASCE,* Vol.94, No.WW3, pp.273-284.

Nagai, S. [1969]: Pressures of standing waves on a vertical wall, *J. Waterways and Harbors Div., Proc. ASCE,* Vol.95, No.WW1, pp.53-76.

Nagai, S. and Otsubo, T. [1968]: Pressure by breaking waves on composite breakwaters, *Proc. 11th Int. Conf. Coastal Engg,* pp.920-933.

Sainflou, G. [1928]: Essai sur les digues maritimes, verticales, *Annales Ponts et Chaussées,* Vol.98, No.4.

Shuto, N. [1974]: Nonlinear long waves in a channel of variable section, *Coastal Engineering in Japan,* Vol.17, pp.1-14.

Tanimoto, K., Yagyu, T., and Goda, Y. [1982]: Irregular wave tests for composite breakwater foundation, *Proc. 18th Int. Conf. Coastal Engg.*, pp.2144-2163.

Tanimoto, K., Takahashi, S., and Kimura, K. [1987]: Structures and hydraulic characteristics of breakwaters — The state of arts of breakwater design in Japan, Rept. Port and Harbor Res. Inst., Vol.26, No.5, pp.11-55..

Tanimoto, K. and Goda, Y. [1991a]: Historical development of breakwater structures in the world, *Proc. Conf. on Coastal Structures and Breakwaters, Inst. Civil Engrs.*, pp.153-166.

Tanimoto, K. and Goda, Y. [1991b]: Stability of deepwater caisson breakwater against random waves, *Proc. Conf. on Coastal Structures and Breakwaters, Inst. Civil Engrs.*, pp.181-193.