Permanent Displacement of Composite Breakwaters Subject to Wave Impact

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Abstract: This paper is concerned with the permanent displacement of composite breakwaters under wave impact. It demonstrates that permanent displacement is a more rational criterion to design composite breakwater as well as to assess its performance when compared to merely a factor of safety. The horizontal wave force and uplift force acting on the caisson are determined from the Goda formulae and expressed as a fraction of the effective weight of caisson using the wave coefficients. A yield wave coefficient is defined for the horizontal wave force when the limiting condition against direct sliding is reached. For a storm that renders a wave coefficient larger than this yield value, a rigid-body motion is induced in the caisson. The equation of motion is double-integrated to obtain the magnitude of permanent displacement. The proposed procedure is illustrated with design example and compared with 35 case histories among which sliding was observed for 24 of them.

INTRODUCTION

Composite caissons are used effectively to protect shorelines and harbors by reflecting wave energy towards the open sea. They are widely constructed in Japan [e.g., Tanimoto and Takahashi (1994)] and European countries, such as Germany, Spain, and Italy (Franco 1994). Design standards of composite breakwaters are well documented [e.g., *Technical* (1989); Takahashi (1997)]. Fig. 1 gives a schematic sketch of a composite breakwater where a monolithic vertical concrete caisson rests on a rubble mound foundation. The caisson is filled with sand and capped with concrete.

The successful performance of composite breakwaters has been attributed to experience and continued research. The conventional rubble mound breakwaters are popular in some countries, such as the United States, as documented in the *Shore Protection Manual* (1984). However, economic benefits of composite breakwaters are realized as the water depth increases and also at the places where good quality rocks are not readily available. Composite breakwaters are environmentally more acceptable than the rubble mounds. The composite caissons had been constructed in water over 60 m deep at Kamaishi Port, Japan (Tanimoto and Takahashi 1994). Naturally, the design and construction of composite caissons are becoming extremely challenging when the water depth increases.

Oumeraci (1994) gave an account of reasons leading to failure of breakwaters. He concluded that failure could be attributed to the structure itself, inaccurate estimation of hydraulic and loading conditions, and foundation and seabed morphology. For example, the waves have been observed to break at a water depth in front of the caissons that is different from the case where caissons are not present. This mechanism of wave breaking is different from shoaling, and its inadequate understanding in design has led to failures of composite breakwaters.

The sliding of caissons due to wave impact is one of the major failure modes, especially for wide caissons. Earlier experience indicated that the engineers did not make a reasonable estimate of the design wave, such as its maximum height and period, which eventually led to catastrophic failure. In modern designs, a proper estimate of wave characteristics is becoming possible, yet failure still occurred because the pressures determined from available formulas are not accurate enough.

As another case of failure, the foundation may not have adequate bearing capacity to sustain the wave that generates a large eccentric load. The caisson may tilt and topple as a results of a lack of seabed stability. Seabed scour and erosion of rubble mound foundation are other factors leading to the failure of caissons. Issues related to bearing capacity and overturning have been examined by Terashi and Kitazume (1987), Kobayashi et al. (1987), Sekiguchi and Ohmaki (1992), and Sekiguchi and Kobayashi (1994), among others.

Most previous studies focused on the composite breakwater sliding did not take performance into account [e.g., Nagai (1963)]. The stability, as indicated by a factor of safety, is used where a value below unity implies failure. It is not correlated to any physical quantity, such as displacement. In reality, the performance of caissons, after a storm, is judged by the magnitude of permanent displacement. Fig. 2 shows a case of significant caisson displacement, following a storm, in the northern part of Japan. Under such a large displacement, the breakwaters may not function satisfactorily. In design, the information on the permanent displacement allows an appropriate dimensioning of caissons and rubble mounds such that excessive movement and catastrophic failure can be avoided. Thus an allowable magnitude of permanent displacement, instead of absolute stability, allows a rational design to be conducted without adversely affecting the performance.

This paper deals with the sliding of the composite breakwater caisson with a focus on its permanent displacement under wave impact. The Goda formulas, which form the basis of wave pressure, are used in this study. The proposed permanent-displacement design approach is formulated, compared with case histories, and discussed.

GODA WAVE PRESSURE FORMULAS

Waves acting on a caisson generate significantly large impact pressure that affects the overall stability and performance of the caisson. The issue related to the development of wave pressure theory is well summarized in Goda (1985). Prior to the design formulas of Goda (1973, 1974), the wave pressure has been estimated based on the works of Hiroi (1919), Sainflou (1928), and Minikin (1950). These earlier formulas are simple but do not account for a smooth transition of pressure

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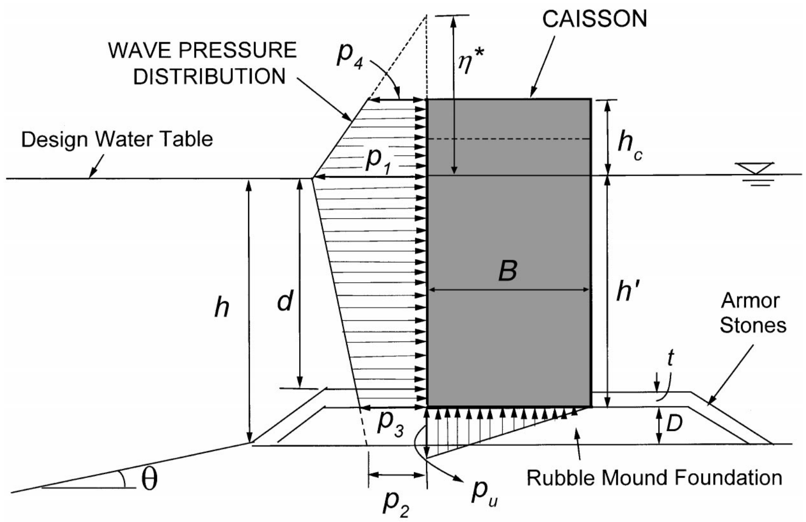

FIG. 1. Composite Breakwater and Wave Pressure Distribution

FIG. 2. Sliding of Caisson (Courtesy of S. Takahashi)

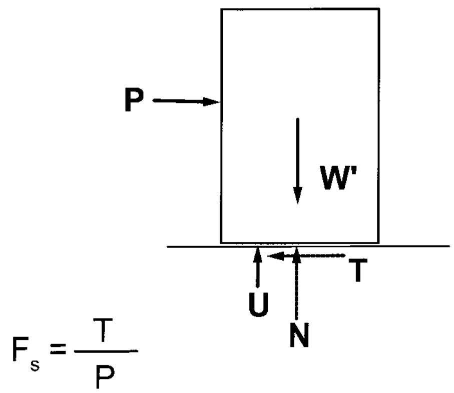

FIG. 3. External Forces Acting in Composite Breakwater under Wave Loading

between breaking and standing waves that can be crucial to the composite breakwater design. In this paper, the Goda formulas are used to determine the horizontal wave pressure and uplift pressure. The notation used follows that shown in Fig. 1: d, h, h', and h_c represent the depth of water measured from the surface to the top of armor block, the depth of water in front of breakwater, the depth of design water table to the bottom of caisson, and the crest elevation of caisson above the design water table, respectively. Only salient features of the Goda formulas are described herewith.

In the Goda formulas, the maximum wave height at the site $H_{\max}$ is used for wave pressure calculation. $H_{\max}$ is taken as the smaller value of $1.8H_{1/3}$ and H_b, where $H_{1/3}$ is the significant wave height and H_b is the weight height estimated at a distance $5H_{1/3}$ from the breakwater. The period of design wave $T_{\max}$ is commonly considered to be similar to the significant wave period $T_{1/3}$. Given the deep-water wavelength $L_o = gT^2/2\pi$, the design wave length L is calculated from the well-known relationship [e.g., Dean and Dalrymple (1991)]

$$L = \frac{gT^2}{2\pi} \tanh \frac{2\pi h}{L} \qquad (1)$$

For a caisson breakwater resting on a seafloor having a slope θ, and subject to a series of waves approaching at an angle β normal to the wall, Goda (1973, 1974) proposed that the horizontal wave pressure follows a trapezoidal distribution whereas the uplift pressure gives a triangular distribution (Fig. 1). These formulas were developed from laboratory test results as well as theoretical considerations. They are based partially on the nonlinear wave theory such that the breaking and gradually varying pressure components are included. The formulas were generalized by Tanimoto and Takahashi (1994) to include different types of structures, such as perforated wall breakwaters and inclined walls.

With reference to Fig. 1, the external forces acting on the caisson, due to the wave pressure, are determined as

$$P = \frac{1}{2}[(p_1 + p_3)h' + (p_1 + p_4)h_c^*] \qquad (2)$$

$$U = \frac{1}{2} p_u B \qquad (3)$$

where P and U = total horizontal wave force and total uplift force acting on the caisson, respectively [Fig. 3(a)]. The wave pressures (p_1, p_2, p_3, p_4, p_u) and related coefficients (α_1, α_2, α_3, α_4) are obtained through the following equations:

$$p_1 = \frac{1}{2}(1 + \cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_2\cos^2\beta)\gamma_o H_{\max} \qquad (4a)$$

$$p_2 = \frac{p_1}{\cosh(2\pi h/L)} \qquad (4b)$$

$$p_3 = \alpha_3 p_1 \qquad (4c)$$

$$p_4 = \alpha_4 p_1 \text{ for } \eta^* > h_c; \quad p_4 = 0 \text{ for } \eta^* \le h_c \qquad (4d)$$

$$p_u = \frac{1}{2}(1 + \cos\beta)\lambda_3\alpha_1\alpha_3\gamma_o H_{\max} \qquad (4e)$$

and

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi h/L}{\sinh(4\pi h/L)}\right]^2 \qquad (5a)$$

$$\alpha_2 = \min\left[\frac{h_b - d}{3h_b}\left(\frac{H_{max}}{d}\right)^2, \quad \frac{2d}{H_{max}}\right] \tag{5b}$$

$$\alpha_3 = 1 - \frac{h'}{h}\left[1 - \frac{1}{\cosh(2\pi h/L)}\right] \tag{5c}$$

$$\alpha_4 = 1 - \frac{h_c^*}{\eta^*} \tag{5d}$$

$$h_c^* = \min(\eta^*, h_c) \tag{5e}$$

$$\eta^* = 0.75(1 + \cos\beta)\lambda_1 H_{max} \tag{5f}$$

where γ_o (=10.1 kN/m^3) = unit weight of seawater; and η^* gives the elevation above water level where the pressure exerts. The factors λ_1, λ_2, and λ_3 are introduced to consider different types of structures (Takahashi 1997). In a vertical caisson, as is the case of this study, $\lambda_1 = \lambda_2 = \lambda_3 = 1$. The value of p_u used in design is normally smaller than p_3 (Goda 1973, 1974, 1985).

STABILITY AGAINST DIRECT SLIDING

The permanent-displacement approach proposed in this paper is composed of two components: stability and permanent displacement analysis. The stability of the caisson against direct sliding along the caisson–rubble mound interface is expressed using a factor of safety F_s, which is obtained as the ratio of total resisting force to disturbing force acting in the caisson [Fig. 3(a)]:

$$F_s = \mu\,\frac{W' - U}{P} \tag{6}$$

where W' = effective weight of the caisson, composed of the concrete cap, and portion of the fill (sand) below and above the water table. In calculating for effective weight of caisson W', the effective unit weight of fill material γ' is used in lieu of the total unit weight γ (i.e., $\gamma' = \gamma - \gamma_o$) for the portion of caisson submerged in water. μ is the coefficient of friction, which is related to the angle of friction δ between the caisson and rubble mound: $\mu = \tan\delta$. Although the coefficient of friction affects the results significantly, and is influenced by many factors, it is not easy to quantify under laboratory and field conditions. In design, μ is commonly taken as 0.6 (or $\delta = 31°$) and $F_s \geq 1.2$ [e.g., Goda (1985); Takahashi (1997)].

In this study, the horizontal wave force and uplift force are expressed as a fraction of the effective weight of caisson, i.e.,

$$P = C_h \cdot W' \tag{7}$$

$$U = C_u \cdot W' \tag{8}$$

where C_h and C_u are called the wave coefficients. Using these wave coefficients, the factor of safety [(6)] is rewritten as

$$F_s = \mu\,\frac{1 - C_u}{C_h} \tag{9}$$

In this calculation, the possible passive resistance offered by the armor stones is neglected, assuming it to be small relative to other external forces.

Figs. 4(a and b) show the required caisson width B under different water depths d for four commonly used design waves ($H_o' = 10$ m, $T_{1/3} = 13$ s; 7 m, 11 s; 5 m, 9 s; 3 m, 7 s) under a factor of safety equal to 1.0 and 1.2, respectively. The standard parameters used in calculations are $\beta = 0$; $\mu = 0.6$; height of mound $D = 3$ m; thickness of armor layer $t = 1.5$ m; $\tan\theta = 1{:}100$; water depth $d = 0 \sim 25$ m ($h = 4.5 \sim 29.5$ m); $h_c = 0.6\,H_{1/3}$; and tide level $WL = 0.6$ m. The effective unit weight of caisson below and above the water table, and the unit weight of concrete cap (3 m thick), are 11 kN/m^3, 21 kN/m^3, and 23 kN/m^3, respectively. Figs. 4(a and b) show two peak points for each design wave; the first point occurs at a smaller value of d or h, corresponding to the maximum value of α_2, and the other point is due to the equation used in estimating H_{max} [see Goda (1985) for details]. The required caisson height for different water depths and design waves $h_t = h' + h_c$, is shown in Fig. 5.

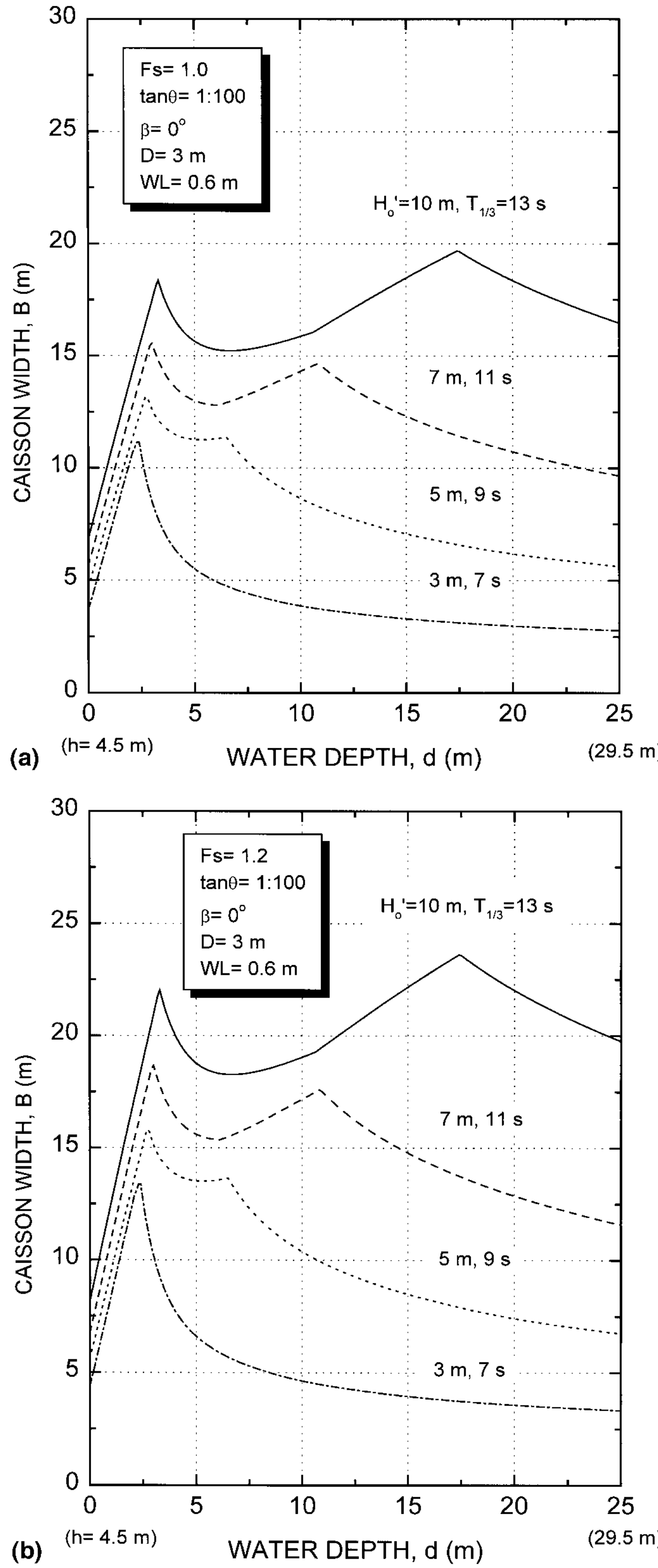

FIG. 4. Required Caisson Width for Different Design Waves and Water Depths: (a) $F_s = 1.0$; (b) $F_s = 1.2$

Figs. 6(a and b) present the coefficients C_h and C_u for these

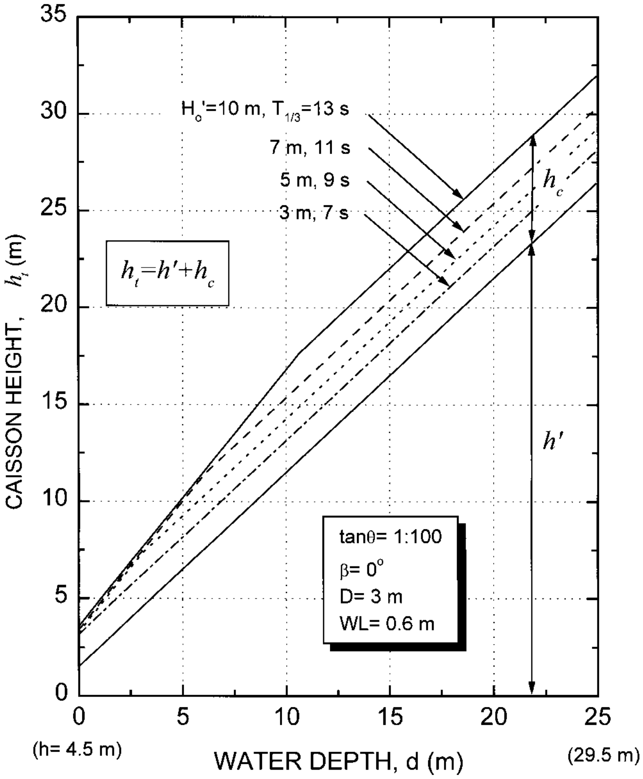

FIG. 5. Caisson Height for Different Design Waves and Water Depth

design waves. At F_s = 1.0, C_h = 0.4 ~ 0.6, and C_u = 0.01 ~ 0.4, depending on the design wave and water depth. At F_s = 1.2, a larger effective weight is involved such that the value of C_h becomes smaller than that at F_s = 1.0. C_u is apparently independent of B [see (3)] and so is F_s. C_h increases with the water depth but C_u reduces as the water depth increases. These figures would be useful while estimating for the yield wave coefficients of a caisson subject to wave loading, as demonstrated subsequently.

PERMANENT DISPLACEMENT OF CAISSON

There were a few attempts made in the past decades to determine the permanent displacement of a composite caisson under wave impact (Horikawa et al. 1972; Shimosako et al. 1994; Klammer et al. 1994). Horikawa et al. (1972) conducted a series of hydraulic model tests, using regular and irregular waves, to validate the sliding equation of Ito et al. (1966). It has to be noted that a reasonable equation for estimating wave pressure was not available at that time. Klammer et al. (1994) conducted experiments and investigated the relationships between wave characteristics, external forces, and permanent displacement. Shimosako et al. (1994) integrated the equation of motion to determine displacement. They also made a comparison between the calculated and measured results. Recently, the dynamic response of breakwater has also been idealized using a mass-spring system that simulates a coupled action of sliding and overturning (Goda 1994; Oumeraci and Kortenhaus 1994; Oumeraci et al. 1996). The drawbacks of a mass-spring system are that only elastic displacement is calculated and the selection of spring constants is not apparent.

A realistic displacement analysis of a composite breakwater may be conducted using a finite-element procedure (Zienkiewicz and Taylor 1991) where the wave loading conditions, material properties of the caisson and rubble mound, caisson-foundation interaction, and the fluid-caisson interaction are modeled realistically. Such a procedure, because of its complexity, is not considered a practical design tool. In this study,

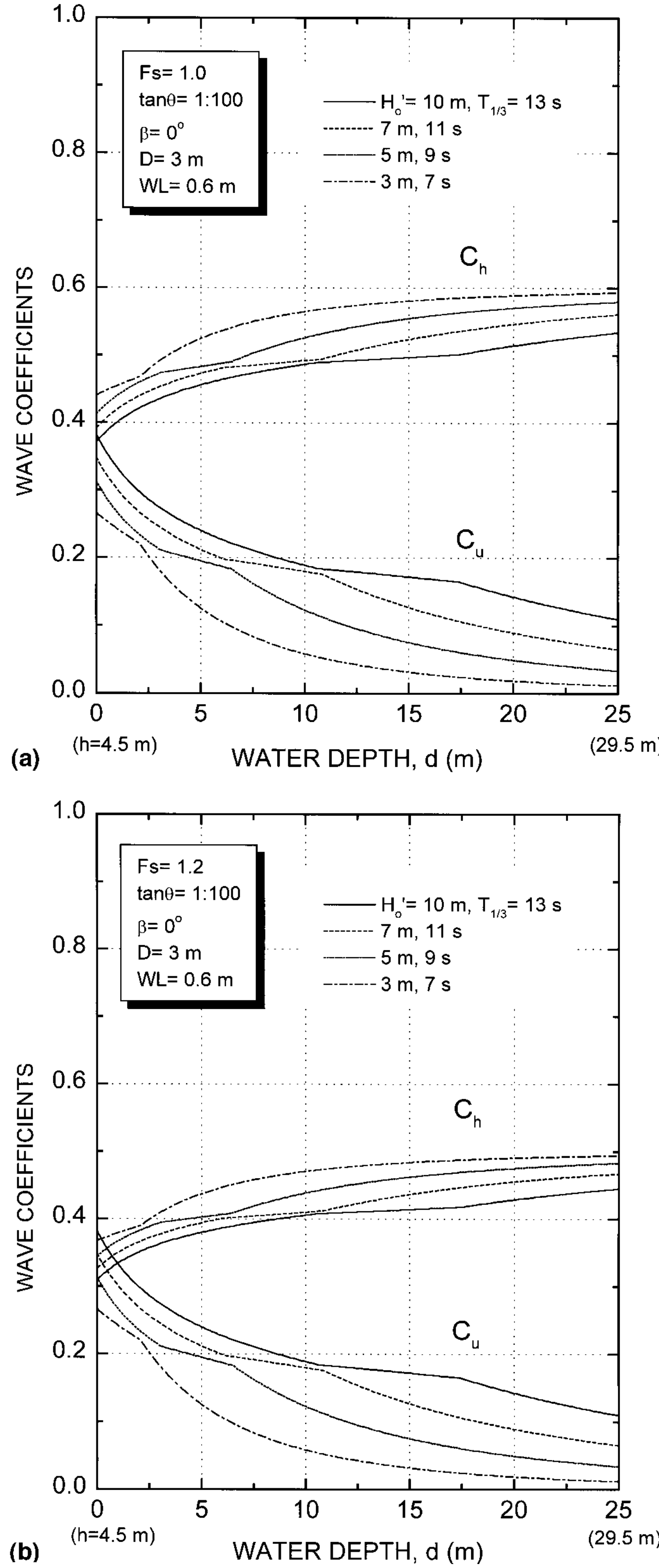

FIG. 6. Wave Pressure Coefficients for Different Design Waves and Water Depth: (a) F_s = 1.0; (b) F_s = 1.2

a simplistic procedure, based on the sliding block theory, is used to determine the permanent (plastic) displacement of a caisson subject to wave loading. The sliding block theory has been used by Newmark (1965) to determine the permanent displacement of earth dams under earthquake loading. This procedure has been extended to some other applications, such as retaining walls (Richards and Elms 1979), reinforced soil structures (Ling et al. 1997), waste containment system (Ling and Leshchinsky 1997), and rock block sliding (Ling and Cheng 1997).

In this procedure, the caisson is considered as a rigid block while its damping and stiffness properties are neglected. The sliding follows a rigid-plastic law, the Coulomb criterion. The caisson's equation of motion in the horizontal direction is written as

$$\frac{W + W_a}{g}\ddot{x} = P - \mu(W' - U) \tag{10}$$

where W, W_a, $\ddot{x}$, and g = total weight, added weight, horizontal acceleration of the caisson, and earth gravity, respectively. Goda (1994; personal communication, 1997) considered added mass by increasing the unit weight of the caisson fill from 21 kN/m^3 to 25 kN/m^3, giving about 20% additional weight. Oumeraci and Kortenhaus (1994) used a simple expression of the form $W_a = 0.543\gamma_o d^2$. In this paper, Goda's approach is adopted because of its simplicity.

Using the wave coefficients [(7) and (8)], the horizontal acceleration of the caisson is expressed as

$$\ddot{x} = \frac{W'}{W + W_a}[C_h - \mu(1 - C_u)]g \tag{11}$$

When the factor of safety is equal to unity, sliding is initiated in the caisson. At this instant of sliding, the wave coefficient calculated from the horizontal wave force is defined as the yield wave coefficient, C_{hy}. From (9), it is expressed as

$$C_{hy} = \mu(1 - C_u) \tag{12}$$

The wave coefficients may also be expressed using a constant ratio $r = C_u/C_h$ for the peak values of the horizontal wave force and uplift force. Thus

$$C_{hy} = \frac{\mu}{1 + r\cdot\mu} \tag{13}$$

Comparison of C_h and C_{hy} gives a quick indication if a caisson slides. From (11) and (12), the acceleration of caisson is simplified as

$$\ddot{x} = \frac{W'}{W + W_a}(C_h - C_{hy})g \tag{14}$$

For the four design waves and water depths shown in Fig. 4, the ratio $W'/(W + W_a)$ ranges from 0.464 to 0.678.

The wave force in excess of yield value triggers movement in the caisson leading to a permanent (plastic) displacement. During the duration of wave loading, the acceleration is double integrated to yield permanent displacement whenever this yield value is exceeded:

$$x = \frac{W'}{W + W_a}\int\int(C_h - C_{hy})g\cdot dt \tag{15}$$

To produce conservative results, it is recommended that the motion of the caisson and thus the permanent displacement in the reverse direction (seaward) be neglected. In this paper, integration of (15) is done numerically such that irregular wave may also be incorporated. The numerical procedure of calculation of permanent displacement follows that presented in Ling et al. (1997).

ILLUSTRATIVE EXAMPLE

The permanent-displacement concept of designing a composite caisson is presented herein. Consider a composite caisson designed to resist a wave of properties $H_o' = 7$ m and $T_{1/3}$ = 11 s, at d = 7.5 m and WL = 0 m. Under F_s = 1.2, the required caisson width B and height h_t are determined as 15.92 and 12.86 m, respectively [which be determined graphically from Figs. 4(b) and 5 by considering d = 6.9 m and WL = 0.6 m]. Therefore, it is assumed that the construction should be conducted using B = 16 m and h_t = 13 m, respectively. These design values correspond to C_h = 0.400 and C_u = 0.189. The yield wave coefficient of this caisson, calculated from (13), is C_{hy} = 0.468. If the caisson will be subject to a storm of H_o' = 8.3 m, $T_{1/3}$ = 12 s, at d = 8.5 m, the wave coefficients are calculated as 0.474 and 0.230 (denoted as C_{ho} and C_{uo}). Since C_{ho} is larger than C_{hy}, the caisson slides.

The variation of wave coefficient (or wave force) with time is assumed sinusoidal, with a peak value corresponding to calculated C_{ho} and C_{uo} (Fig. 7), and they are in phase with each other. Given $\omega = 2\pi/T_{1/3}$, the horizontal and vertical wave coefficients are expressed as

$$C_h = C_{ho}\cdot\sin(\omega\cdot t);\quad C_u = C_{uo}\cdot\sin(\omega\cdot t) \tag{16a,b}$$

The yield coefficient obtained from (12), which is dependent on C_u, is plotted as a dotted line in Fig. 7. For this particular wave cycle, it exhibits a short duration, for less than 1 s, where the yield coefficient is exceeded. When the horizontal coefficient is larger than the yield coefficient, the motion is induced in the caisson. The horizontal wave force increases to a peak value, then decreases to the critical value, where the velocity of the caisson starts to reduce and stops when it becomes zero. The permanent displacement per wave cycle is calculated as 2.1 cm for this particular example. If 20 wave cycles of sinusoidal waves are used for a storm, the accumulated permanent displacement would be close to 40 cm.

Note that if the caisson is designed against the absolute stability for this wave (H_o' = 8.3 m, $T_{1/3}$ = 12 s, and d = 8.5 m), i.e., without allowing for any permanent displacement, the required caisson dimension is B = 17.2 m and h_t = 14.6 m at F_s = 1.2. Thus, a larger volume of caisson per unit length is needed. In a real storm, where the wave height varies randomly, it is expected that only a limited number of wave cycles are actually responsible for permanent displacement (Goda, personal communication, 1997). Therefore, in practice,

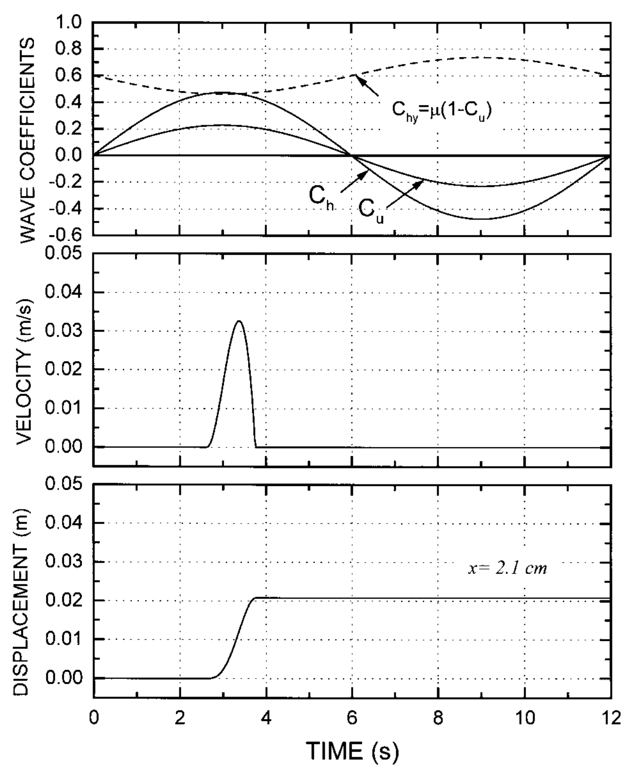

FIG. 7. Illustrative Example—Response of Caisson Subject to Sinusoidal Wave

the real storm records are recommended for the permanent-displacement design.

CASE HISTORY COMPARISON

Goda (1973, 1974) verified his pressure formulas with 34 case histories of caisson performance in Japan, including 21 unstable and 13 stable cases. The results were expressed in terms of factor of safety and compared with the formulas proposed by Sainflou (1928), Hiroi (1919), and Minikin (1950). The comparison indicated that Goda formulas are able to predict satisfactorily the stability of composite breakwaters whereas inconsistencies arose in other formulas.

These 34 case histories of caisson performance are revisited. The geometry of caisson and additional site conditions are found in Goda (1973, 1974). The equivalent deep-water height was obtained as $H_o' = H_{1/3}/K_s$, where K_s is the shoaling coefficient [see Goda (1985) for details]. Table 1 shows the calculated factor of safety, wave coefficients, and yield wave coefficient for these case histories. The relationships between F_s and $C_{ho} - C_{hy}$ is given in Fig. 8, which shows that F_s reduces following an increase in $C_{ho} - C_{hy}$. It is seen that for the sliding cases, the factor of safety is less than unity. In other words, the wave coefficient for an estimated storm (C_{ho}) is larger than the yield coefficient of the caisson (C_{hy}).

The permanent displacement per one sinusoidal wave cycle is calculated and summarized in Table 1. The total displacement measured at the sites (Goda 1973) are compared. Table 1 shows that for certain cases, the calculated permanent displacement per wave cycle is much larger than the measured displacement. For other cases, the total number of wave cycles leading to the measured displacement ranges from 2 to 38, but most of them correspond to less than 20 wave cycles. A number, say 20, could be reasonable in conducting for a permanent-displacement design. Although not included in Table 1, the caisson's factor of safety against overturning was calculated to be larger than that of direct sliding for these case histories.

The comparison as previously described indicated that a reasonable estimation of permanent sliding can be made if the real wave height versus time records of the storm are available. The predicted displacements are mostly on the conservative side when compared to the measured values. Note that the accuracy of comparison could be affected by (a) the irregular wave characteristics during a storm; (b) the fact that added mass was approximated and the movement in the reversed seaward direction was ignored in this study gave a larger calculated permanent displacement; and (c) the possible out-of-phase relationship between C_h and C_u.

The test results of Shimosako et al. (1994, Figs. 6 and 8) and Klammer et al. (1994, Figs. 4 and 6), which are based on hydraulic model tests, did not show any phase lag between the horizontal wave force and uplift force. This, however, may not be the case of a full-size caisson sitting in the sea at a much greater water depth. The possible effects of this phase difference are investigated by introducing a phase angle ϕ into (16*b*):

$$C_u = C_{uo} \cdot \sin(\omega \cdot t + \phi) \tag{17}$$

The Abashiri case is calculated using two different values of ϕ (45° and 90°). Fig. 9 shows the relationships between the wave coefficients, velocity, and permanent displacement.

TABLE 1. Peak and Yield Wave Coefficients and Permanent Displacements

Port (1)	Number (2)	H_{max} (m) (3)	$T_{1/3}$ (s) (4)	d (m) (5)	F_s (6)	C_{ho} (7)	C_{uo} (8)	C_{hy} (9)	Measured total displacement (m) (10)	Calculated displacement per wave cycle (m) (11)
1. Abashiri	A	8.7	9.0	5.0	0.843	0.560	0.213	0.489	4	0.50
2. Monbetsu	B-1	7.8	8.0	6.0	0.672	0.687	0.231	0.499	3	1.92
3. Monbetsu	B-2	6.7	8.0	6.0	1.467	0.358	0.126	0.495	—	—
4. Rumoi	C-1	8.5	9.0	5.4	0.683	0.656	0.253	0.487	0.5	2.20
5. Rumoi	C-2	9.5	9.0	13.9	0.942	0.478	0.249	0.457	—	0.05
6. Iwanai	D	8.1	9.0	7.0	0.780	0.571	0.258	0.472	0.3	1.00
7. Todohokke	E-1	5.5	9.7	2.1	0.669	0.725	0.191	0.518	1.5	3.49
8. Todohokke	E-2	5.3	10.0	4.0	1.118	0.435	0.190	0.475	—	—
9. Himekawa	F-1	11.3	11.1	4.5	0.466	0.895	0.305	0.498	5.4	15.07
10. Himekawa	F-2	10.8	11.0	8.5	1.068	0.462	0.178	0.488	—	—
11. Kanazawa	G-1	6.7	9.0	3.0	0.654	0.571	0.378	0.429	0.4	2.35
12. Kanazawa	G-2	7.7	11.4	5.0	1.225	0.389	0.206	0.456	—	—
13. Hachinohe	H-1	6.0	10.0	2.5	0.750	0.573	0.284	0.462	3.7	1.46
14. Hachinohe	H-2	7.6	11.5	4.5	1.111	0.448	0.170	0.489	1.4	—
15. Onahama	I-1	9.9	10.0	7.0	1.134	0.401	0.242	0.441	0.9	—
16. Onahama	I-2	9.8	11.0	6.0	1.004	0.470	0.215	0.471	0.6	0.06
17. Onahama	I-3	9.4	10.0	6.0	1.025	0.480	0.180	0.490	—	—
18. Onahama	I-4	10.8	11.0	7.0	0.868	0.526	0.240	0.471	—	0.39
19. Kashima	J-1	11.7	14.0	8.5	0.953	0.507	0.194	0.488	1.0	1.11
20. Kashima	J-2	11.7	14.0	12.5	0.839	0.543	0.240	0.474	0.5	0.09
21. Kashima	J-3	11.7	14.0	12.5	0.953	0.503	0.202	0.483	1.4	0.07
22. Kashima	J-4	11.5	14.0	8.5	0.953	0.500	0.206	0.481	—	0.08
23. Kashima	J-5	11.7	14.0	10.5	1.039	0.474	0.180	0.489	—	—
24. Kashima	J-6	11.7	14.0	12.5	1.170	0.441	0.140	0.504	—	—
25. Yokohama	K-1	5.8	6.0	6.5	0.961	0.501	0.198	0.485	1.0	0.96
26. Yokohama	K-2	6.3	7.3	5.2	0.947	0.553	0.126	0.528	—	0.04
27. Kurihama	L	5.7	8.5	4.5	0.710	0.649	0.232	0.494	5.0	0.22
28. Kaizuka	M	4.7	5.5	1.5	0.882	0.512	0.243	0.465	8.0	0.21
29. Kobe	N	5.9	6.0	4.9	0.847	0.607	0.143	0.526	0.05	0.17
30. Mega	P-1	6.5	6.8	5.2	0.868	0.575	0.169	0.510	0.4	0.61
31. Mega	P-2	6.5	6.8	5.2	1.012	0.464	0.217	0.469	—	—
32. Wakayama	Q-1	9.2	11.0	6.1	0.790	0.546	0.281	0.459	0.4	1.18
33. Wakayama	Q-2	8.6	12.0	5.0	0.854	0.552	0.215	0.487	0.6	0.75
34. Niigata	R	10.1	12.5	8.0	0.787	0.625	0.180	0.511	—	1.91

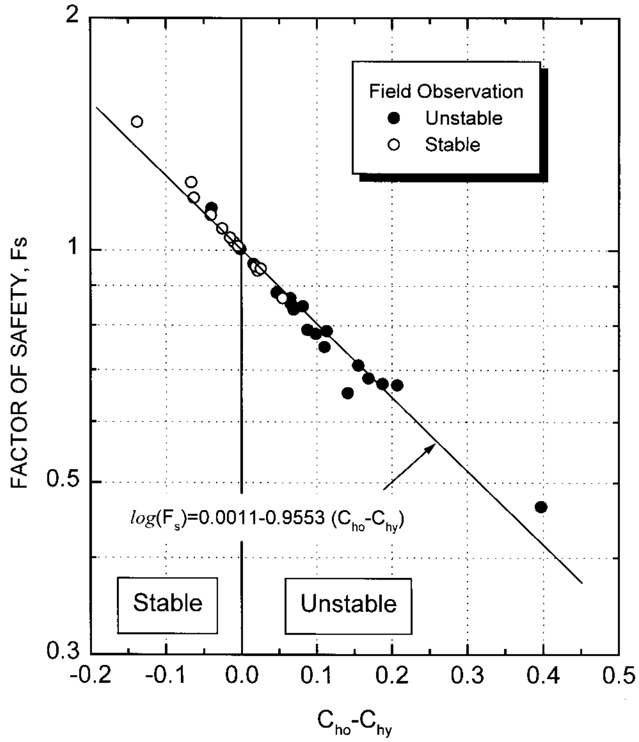

FIG. 8. Relationships between Factor of Safety and Wave Coefficients

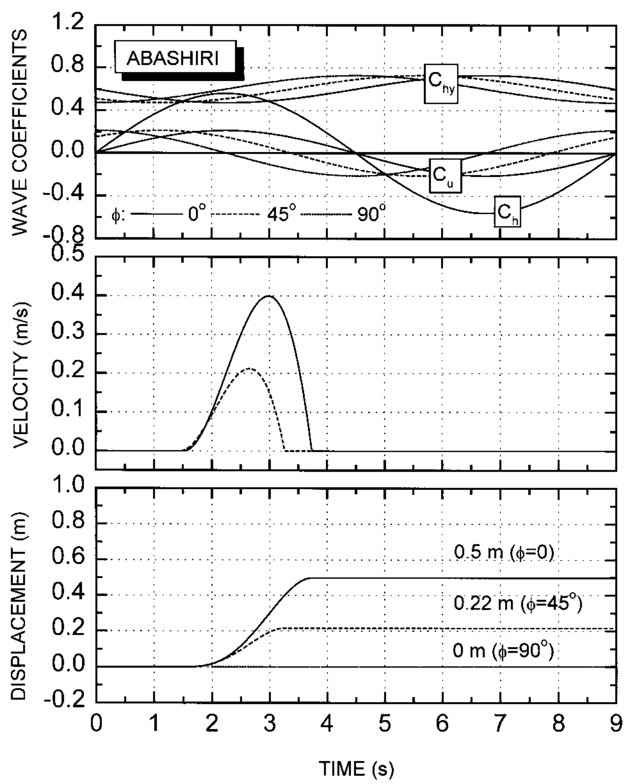

FIG. 9. Effect of Phase Angle on Caisson Response: Abashiri Case History

When ϕ = 45°, the yield coefficient increases and thus a smaller permanent displacement compares to that of ϕ = 0 results. When ϕ = 90°, C_{hy} is greater than C_{ho} and no permanent displacement is anticipated. Table 2 summarizes the results of all reported sliding cases where the permanent displacement reduces significantly as ϕ changes from 0 to 45° and to 90°. Thus, possible effects of phase angle between the horizontal wave force and uplift force warrant further study; in particular, field monitoring is recommended. Note, however, ϕ = 0 results in a conservative design.

TABLE 2. Phase Angle between C_h and C_u and Permanent Displacement Per Wave Cycle (m)

Number (1)	ϕ = 0 (2)	ϕ = 45° (3)	ϕ = 90° (4)
A	0.50	0.22	—
B-1	1.92	1.43	0.43
C-1	2.20	1.55	0.32
D	1.00	0.52	—
E-1	3.49	2.99	0.12
F-1	15.07	12.80	7.46
G-1	2.35	1.40	0.02
H-1	1.46	0.72	—
J-1	0.06	—	—
J-2	1.11	0.42	—
J-3	0.09	—	—
K-1	0.07	—	—
L	0.96	0.67	0.12
M	0.22	0.01	—
M	0.21	0.12	0.01
P-1	0.17	0.07	—
Q-1	1.18	0.54	—
Q-2	0.75	0.29	—
R	1.91	0.37	0.04

SUMMARY AND CONCLUSIONS

This paper suggested a design procedure for a caisson breakwater using a permanent displacement in addition to sliding stability. The wave coefficients were introduced to express rationally the total horizontal wave force and uplift force in a composite caisson subjects to wave impact. The yield wave coefficient was obtained at the limiting condition when the factor of safety against direct sliding equal to unity. The acceleration was then double-integrated to give the permanent displacement of caisson whenever this yield value is exceeded. An estimation of permanent displacement allows a reasonable assessment of performance and rational design of caisson when compared to simply a factor of safety. This simplified procedure was verified against 34 case histories and its practical implications were discussed.

The proposed procedure includes several assumptions leading to a conservative design. It is recommended that additional case histories be verified to support this design procedure. The field monitoring and real wave records would improve the understanding of the permanent-displacement analysis of composite breakwaters.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

B = width of caisson;
C_h, C_u = wave coefficients;
C_{ho}, C_{uo}, C_{hy} = peak wave coefficients and yield wave coefficient;
D = height of rubble mound;
d = depth of water to the top of armor block;
F_s = factor of safety to resist direct sliding under static conditions;
g = earth gravity (9.81 m/s^2);
H_b, H_o = wave height at distance $5H_{1/3}$ from breakwater, and deepwater wave height;
$H_{\max}$, $H_{1/3}$ = maximum and significant wave heights;
h, h', h_b = depth of water in front of breakwater, depth of water to bottom of caisson, and depth of water at distance $5H_{1/3}$ from breakwater;
h_c, h_t = crest elevation of caisson above water level, and total height of caisson;
K_s = shoaling coefficient;
L_o, L = deep-water wavelength and design wave length;
P = horizontal wave force;
p_1, p_2, p_3, p_4, p_u = wave pressures;
$r = C_u/C_h$ = ratio of wave coefficients;
$T_{\max}$, $T_{1/3}$ = design wave period and significant wave period;
t = time and thickness of armor layer;
U = uplift force;
W, W_a, W' = weight, added weight, and effective weight of caisson;
WL = tide level;
$\ddot{x}$, x = acceleration and permanent displacement of caisson;
α_1, α_2, α_3, α_4 = wave pressure coefficients;
β = angle of wave incidence normal to breakwater;
γ_o = unit weight of sea water (10.1 kN/m^3);
δ = angle of friction between caisson and rubble mound foundation;
η^* = elevation above water level where wave pressure acts;
θ = sea floor gradient;
λ_1, λ_2, λ_3 = coefficients related to structure type of caisson;
μ = coefficient of friction between caisson and rubble mound foundation;
ϕ = phase angle between horizontal wave force and uplift force; and
ω = angular frequency.