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EVALUATION OF VERTICAL SETTLEMENT OF A CAISSON BREAKWATER PROTECTED BY FAILED ARMOUR

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There are a number of methods available to estimate the damage that a breakwater could take from a given storm, such as the procedure of Shimosako and Takahashi (1999) as modified by Esteban et al. (2007) for caisson breakwaters. However, for the case of an armoured caisson breakwater, it is not clear what effect the failure of the armour will have on the final movement of the caisson. In Japan, caissons that are located at a transition area between an armoured section and a non-armoured section usually suffer more damage than caissons located at other sections. This can be a serious problem, and one that has received comparatively little attention in the past, to the authors' knowledge. The aim of the present study is thus to clarify the failure mechanism of the caissons and to estimate the probability distribution functions of vertical movement at the heel of the caisson.

Key Words : *caisson breakwater, reliability design, armour, failure probability*

1.INTRODUCTION

Recently there has been a move from traditional deterministic to probabilistic methods to design caisson breakwaters. Many of these methodologies started with the work done by Shimosako and Takahashi (1999), who proposed a Level 3 design method for caisson breakwaters referred to as the "deformation-based reliability design". This model uses the Goda formula (1974) as modified by Takahashi et al. (1994) in order to obtain the wave pressures at the face of the caisson. This modification simplifies the time history of wave pressure on the caisson into a triangular "church-roof" shape (impulsive wave force) and a sinusoidal part (standing wave force). In this approach the expected sliding distance of the caisson is a statistical average of the sliding distance over the service lifetime of the structure as computed by a Monte-Carlo type simulation.

More recently research by Kim and Takayama (2003) and Takagi and Shibayama (2006) have proposed different improvements to the basic model of Shimosako and Takahashi (1999). However, in all these models the displacement caused by a certain wave pressure is assumed to stay constant throughout the caisson's life. Kim and Takayama (2004) modified this model to take into account the effect of caisson tilting on the computation of sliding distance, though they rely on assumptions about the final tilting angle. By using simple soil mechanics consolidation theory Esteban et al. (2007) calculated the amount of settlement at the heel of the caisson, thus allowing for the calculation of the tilt in the breakwater.

However, all these authors dealt with the rather simple case of non-armoured caisson breakwaters. To understand the effect that armour has on the forces acting on the face of the caisson, Esteban et al. (2009) carried out experiments using different configurations of armour layers in front of the caisson. This is quite important for the case of breakwaters where one part of the breakwaters is formed of armoured caissons and the other of caissons only, as the transitional area between these two sections usually suffers the greatest damage. This effect is also important for damaged armour layers, as although the erosion of the armour can be calculated using the Van der Meer formula(1988), the effect that this damage has on the computation of the forces on the caisson (by using the Goda (1974) formula for example) is still poorly understood. Based on their laboratory experiments Esteban et al. (2009) proposed certain modifications to the Goda formula. However, the effect that these modifications would have on the final vertical movement at the back of the caisson has still not been quantified. The present paper will thus modify the methodology of Esteban et al. (2007) by using the effect of different configurations layers in front of a caisson proposed by Esteban et al. (2009) to determine the probability distribution functions of vertical movement at the shoreside heel of a caisson breakwater.

2.METHODOLOGY

The model of Esteban et al. (2007) relies on the Goda formula (1974) as modified by Takahashi et al. (1994) to determine the pressure of the wave on the face of the caisson breakwater. However, this formula was not designed for an armour protected caisson breakwater. Hence, to correctly evaluate the failure of a caisson breakwater protected by a partially constructed damaged armour layer Esteban et al. (2009) modified the Goda formula by including an extra parameter that takes into account this magnifying effect.

The model of Esteban et al. (2007) attempts to reproduce the vertical and horizontal movements of a caisson over the duration of one or several storms. To calculate the force exerted by each wave on the caisson, the procedure proposed by Tanimoto et al. (1996) is used. The time history model is made of the superposition of an impulsive "church-roof" shaped wave force $P_2(t)$, and a slowly varying standing one $P_1(t)$, as given by:

$$P(t) = \max \{ P_1(t), P_2(t) \} \qquad (1)$$

To calculate $P_1(t)$ the Goda formula is used considering only a parameter α_1, and to calculate $P_2(t)$ it is necessary to evaluate the pressure exerted by an impulsive (breaking) wave. In the formula of Goda (1974), as modified by Takahashi et al. (1994), the impulsive pressure component of the wave is given by a parameter α^*, which replaces the factor α_2 in the original Goda formula. This factor is defined as follows:

$$\alpha^* = \max(\alpha_2, \alpha_I) \qquad (2)$$

where α_2 denotes a coefficient indicating the effect of the impulsive pressure in the original Goda formula, and α_I gives an impulsive pressure coefficient introduced by Takahashi et al. (1994).

It is then necessary to establish how this force acting on the face of the breakwater transmits itself onto the foundations. Goda (1985) indicates how for sites where the seabed consists of a dense sand layer or soil of good bearing capacity a simplified technique of examining the magnitude of the heel pressure can be used. In this case, it is assumed that a trapezoidal or triangular distribution of bearing pressure exists beneath the bottom of the upright section, and the largest bearing pressure at the heel p_e can be calculated by using:

$$p_e = \frac{2W_e}{3t_e} \qquad : t_e \le \frac{1}{3}B$$

$$p_e = \frac{2W_e}{B}(2 - 3\frac{t_e}{B}) \qquad : t_e > \frac{1}{3}B \qquad (3)$$

in which

$$t_e = \frac{M_e}{W_e}, \qquad M_e = W't - M_U - M_P, \qquad W_e = W' - U \qquad (4)$$

where W' is the weight of the caisson per unit extension in still water, t the horizontal distance between the centre of gravity and the heel of the upright section, U the total uplift pressure, M_u the momentum around the heel of the caisson due to this uplift, M_p the moment around the bottom of an upright section due to the pressure at the face of the breakwater and B the width of caisson.

Once p_e is determined then the movement of the breakwater can be estimated by using Newton's Law of Motion (see Shimosako and Takahashi, 1999 and Esteban et al, 2007). For the case of the horizontal direction, the movement of the caisson is mostly resisted by the friction between the gravel particles and the bottom of the caisson. However, for the case of the the vertical displacement, Esteban et al. (2007) indicate how it is the stiffness of the gravel (represented by its bearing capacity, q_U) which resists this motion. This bearing capacity changes according to the density of the gravel, which Esteban et al. (2007) suggests increases due to the compaction effect of the waves, slowing the caisson movement given by the equation:

$$\left(\frac{W}{g} + M_a\right)\ddot{x}_E = \left(\frac{2 \cdot P_{foundation} + W'}{B} - q_U\right)s \qquad (5)$$

where $\ddot{x}_G$ is the acceleration at the centre of gravity of the caisson, M_a the added mass, W the caisson weight in air, g the acceleration of gravity, $P_{foundation}$ is the total pressure applied to the foundation by one wave, q_U is the bearing capacity of the foundation, and s is the area of maximum pressure at the heel of the caisson.

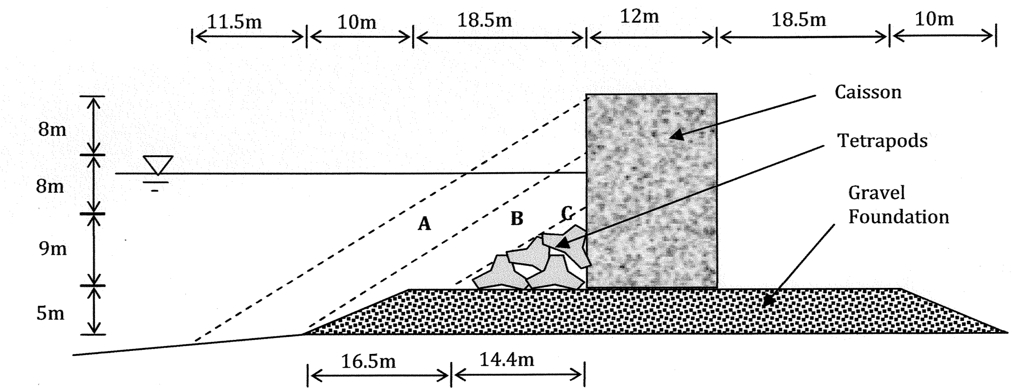

Fig. 1. Basic geometry of the various armour configurations

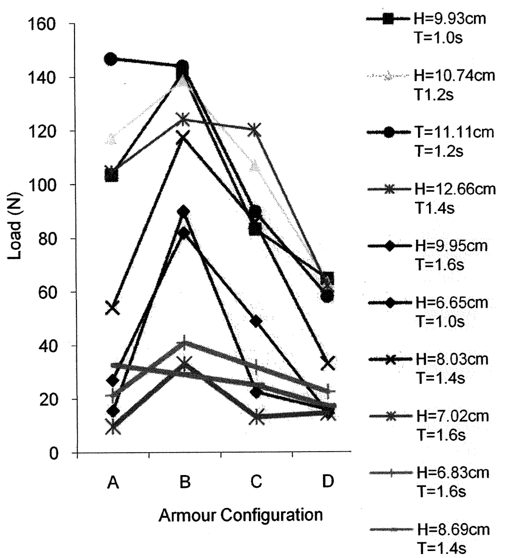

Fig. 2. Maximum loads for breaking and overtopping waves

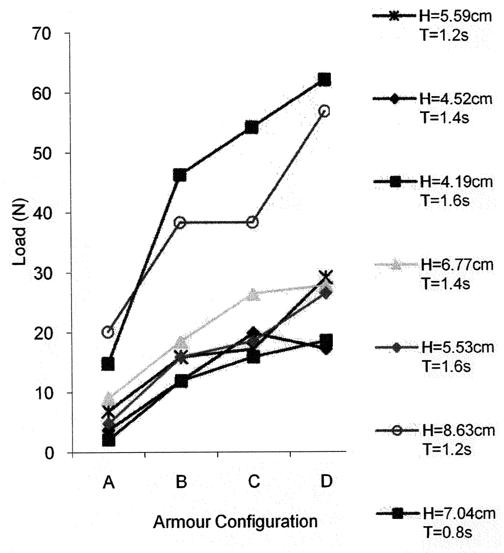

Fig. 3. Maximum loads for non-breaking waves

However, to determine the effect that partially failed armour has on the pressure at the face of the breakwater, Esteban et al. (2009) carried out experiments on overtopping, breaking and non-breaking waves for a variety of armour layer configurations, as shown in Fig. 1 (going from a full layer in configuration A to no armour being present in configuration D). Each type of wave and armour layer configuration was shown to have different effect on the forces exerted on the caisson, as shown in Figs. 2 and 3. Hence, Esteban et al. (2009) proposed a new parameter, α_a, which describes the influence of the different armour configurations and wave types on the load applied to the foundations. Essentially, Esteban et al. (2009) calculated how much larger each of the loads for configurations A to C were with regards to D, and presenter their parameter map as shown in Table 1 were derived by calculating how much larger were each of the loads for configurations A to C with respect to D. Thus, for each of the armour configurations shown in Fig. 1 α_a would take a different value, depending on the type of wave. This parameter map shows how for the case of overtopping and breaking waves, armour can increase the forces exerted by the caisson onto the foundation. However, for the case of non-breaking waves most of the energy of the wave is dissipated by the armour layers.

Table 1. α_a parameter map

	A	B	C	D
Overtopping Waves	2.0	2.2	1.7	1.0
Breaking Waves	1.4	3.3	1.8	1.0
Non-breaking Waves	0.2	0.7	0.8	1.0

According to Esteban et al. (2007), eq. 3 would thus become:

$$p_e = \alpha_a \frac{2W_e}{3t_e} \qquad : t_e \le \frac{1}{3}B$$

$$p_e = \alpha_a \frac{2W_e}{B}(2 - 3\frac{t_e}{B}) \qquad : t_e > \frac{1}{3}B \qquad (6)$$

Therefore, by using this corrected equation and the method of Esteban et al. (2007) it is possible to calculate the vertical movement of the breakwater after a given storm.

3.RESULTS

Using the methodology proposed a computer simulation based on that of Esteban et al. (2007) was developed. The simulation computed the vertical movement at the back of the caisson for a representative 2 hour long storm (720 waves) for each of the four configurations considered. The simulation generated irregular random waves using a Rayleigh distribution, with a deepwater significant wave height $H_{1/3}$=5.7m and an average period T=10sec. The Factor of Safety of the prototype non-armoured caisson breakwater was estimated to be 0.67 against sliding, meaning that significant sliding and deformation in the rubble mound would be expected from this storm.

The probability distribution functions of vertical movement for each of the four armour configurations are shown in Fig. 4. Essentially, the parameter α_a reproduces the greater forces that Esteban et al. (2009) report for overtopping and breaking waves for Configurations A to C. The probability distribution functions of vertical movement thus shift progressively to the right, especially for Configuration B, where the magnification effect was found to be the greatest.

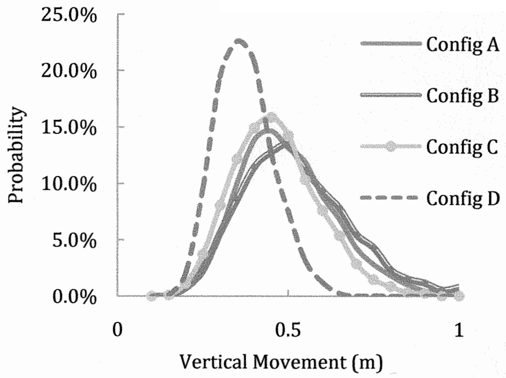

Fig. 4. Probability distribution functions of vertical movement at the heel of the caisson

4.DISCUSSION

The estimation of the correct load exerted by the wave on the foundations of a caisson breakwater is essential for the correct calculation of the vertical deformation of the rubble mound, as shown by Esteban et al. (2007). These authors showed how the "church-like peaks" are responsible for most of the vertical deformation, and for this reason the correct prediction of when these peaks appear is of great importance. Figs. 2 and 3 show how it is important for the practicing engineer to determine which type of loading can be expected, and whether the crest of this wave will be higher than the caisson ("overtopping" wave) or not. The shape of the wave at the time when it hits the caisson clearly affects the load time history. Non-breaking waves exert mainly hydrostatic pressure, while breaking waves exert the characteristic "church-roof" loading. Thus, it is clear that partially failed armour layers can induce breaking on a wave that would otherwise not be as damaging to the breakwater. In these cases it is important whether the waves break on the caisson directly or into the armour, as the armour can successfully dissipate some of the energy of the waves (Esteban et al. 2009). For the case of "overtopping" waves, the presence of armour makes the wave pile even higher on top of the breakwater, and produces even higher loads than when little or no armour is present. However, in this case there is little difference between configurations A and B, as the crest of the wave is higher than the caisson, and hence waves break just above the caisson, mitigating the breaking wave effect. These effects explain the differences in the probability distributions of vertical deformation observed in Fig. 4.

However, care must be exercised when using the methodology of Esteban et al. (2007). This model uses fairly simplistic soil mechanics theory, which unfortunately has problems in dealing with impulsive loading. The crucial part of the load which causes the deformation to the foundations lasts for fractions of a second, and most soil mechanics theories deal with loads being applied over a long period of time (not rapid cycles of loading and unloading). Also to be noted is that the entire soil is submerged in water and can be considered to be in the fully drained condition. All these points raise questions as to the validity of using normal soil mechanics theories in the model proposed.

The accuracy of the soil mechanics parameters used is also an important factor in the final outcome of the simulation. Here, the soil mechanics factors used were the same as in Esteban et al. (2007), though these authors note that these were based in small scale laboratory measurements, and due to boundary effects there could have large errors in the final gravel parameters obtained. The parameters used have a big effect in the final computed deformation of the rubble mound, and hence the choice of appropriate ones is of paramount importance if the model is to be accurate.

Scale effects could also be a possible significant source of discrepancy between the results of the model and real life. The bearing capacity of the foundation is governed by the amount of friction between gravel, and thus the shape and contact area between particles can greatly affect this parameter. It is not clear to what extent the values by Esteban et al. (2007) can translate into the real life values due to these differences.

It is interesting to note that according to the results of this paper, it appears that although armour layers can contribute to the stability of the caisson for the case of non-breaking waves, they appear to increase damage for other types of waves. It is thus very important to correctly design the armour layers, as if these are not able to take the wave loadings by themselves their presence can negatively affect the stability of the caisson. This effect, however, is still not clearly understood, and should be further clarified in the future.

5.CONCLUSIONS

The present results attempt to extend the level III reliability design method of Esteban et al. (2007) to the case of armoured caisson breakwaters. The methodology presented is able to estimate the vertical deformations at the heel of such a structure, and highlights the importance of correctly designing armour layers and transition areas between a part of a breakwater with armour and another without it. The incorrect design of such layers can result in breaking and overtopping waves exerting far more considerable forces to the caisson that what the Goda formula would suggest for a non protected caisson. The present work thus can help to explain and quantify mathematically a phenomenon understood intuitively by many practicing engineers.

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REFERENCES

Esteban, M., Takagi, H. and Shibayama, T. (2007). Improvement in Calculation of Resistance Force on Caisson Sliding due to Tilting. *Coastal Engineering Journal*, Vol. 49, No.4 (2007) pp 417-441

Esteban M., Takagi H. and Shibayama T. (2009) Modification of the Goda Formula to Evaluate the Movement of a Caisson Breakwater Protected by Failed Armour, *Proc. Of Coasts, Marine Structures and Breakwaters 2009*, 16-18 Sept, Edinburgh, UK.

Goda, Y. (1974). New wave pressure formulae for composite breakwater, *Proc. 14th Int. Conf. Coastal Engineering*, Copenhagen, ASCE, pp.1702-1720.

Goda, Y (1985). Random Seas and Design of Maritime Structures, University of. Tokyo Press.

Kim, T.M., Takayama, T. & Miyawaki, Y. (2004) Laboratory Experiments on the Sliding Distance and Tilting Angle of a Caisson Breakwater Subject to Wave Impact, *Proc. 29th Int. Conf. on Coastal Eng.*, ASCE, pp 3762-3773.

Kim, T.M., Takayama, T. (2003). Computational Improvement for Expected Sliding Distance of a Caisson-Type Breakwater by Introduction of a Doubly-Truncated Normal Distribution, *Coastal Engineering Journal*, Vol.45, No.3 (2003) 387-419.

Shimosako, K., and Takahashi, S. (1999). Application of deformation-based reliability design for coastal structures –Expected sliding distance method of composite breakwaters -. *Coastal Structures '99*, ed. I.J. Losada, Spain, Balkemare, pp. 363-371.

Takahashi,S., Tanimoto, K. & Shimosako, K. (1994) A Proposal of Impulsive Pressure Coefficient for Design of Composite Breakwaters, *Proc. of the International Conference of Hydro-Technical Engineering for Port and Harbour Construction*:489-504

Takagi, H. and Shibayama, T. (2006). A new approach on performance-based design of caisson breakwater in deep water, *Annual Journal of Coastal Engineering*, JSCE, Vol.53, pp.901-905 (in Japanese)

Van der Meer, J.W. (1988). Deterministic and Probabilistic design of a breakwater armour layers. *J. Waterway Port Coastal Ocean Eng.* ASCE, 114, 1, pp 66-80.