COMPUTATIONAL ESTIMATION OF CAISSON SLIDING AND TILTING OF SUSAMI WEST BREAKWATER DUE TO TYPHOON TOKAGE

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近年台風災害の頻発が上がりました。２００４年１０月２０日に台風トカゲが西周参見の防波堤を破損しまりました。Kim et al. (2005) はこの破損の記録をとりました。本論に西周参見の防波堤の破損は算出記載する。Esteban et al. (2007) は波浪の作用によって生じるケーソン式防波堤のマウンドの変形手法を提案した。ブシネスクの解を使用する簡便な土質力学の理論を適用することで，ケーソン式防波堤の鉛直と滑り込み（Sliding）変位を予測する。

Key Words : *Susami West, composite breakwater, caisson sliding, caisson tilting, failure*

1. INTRODUCTION

There are many documented cases of failures of caisson breakwaters in recent times, with sliding of the upright section representing the majority of these failures (Goda 1999[1]). One of the most well documented cases according to this author is that of the Mustafa breakwater at Algiers Harbour in Algeria. This structure was built at a water depth of 18 to 20m using cyclopean blocks designed to resist waves 5.0 m in height and with a T of 7.4s. A large storm in February 1934 (with a peak wave height of 9m and period of 13.5s) damaged a 400m section of the breakwater. Interestingly three quarters of the failed sections fell towards the seaside and only a quarter towards the harbour side. The reason for this according to Oumeraci (1992)[2] lies in the possibility of a slip failure through the seabed. However this type of failure is relatively rare, with Takahashi et al. (2000)[3] citing how in more than half of the major breakwater failures that occurred between 1983 and 1991 in Japan sliding was recorded. These authors also cite a survey of damage to major caisson breakwaters conducted by the Bureau of Ports and Harbours (BPH), which indicates how 23 caissons were damaged in the period 1991-2000, with 75% of them suffering meandering sliding, and 25% being damaged due to wave-induced strong currents. As a total of 9644 caissons existed in Japan during this time only about 0.2% suffered some type of problem, which led these authors to conclude that the probability of encountering sliding over a 50-yr lifetime is about 1%.

Recently Takagi (2007)[4] studied the failure of the breakwater at Hakodate Port in Hokkaido in 2004, believed to have failed due to bearing capacity failure attributed to standing wave pressures at key points along the breakwater. This author also introduced a new concept called the "Expected Occurrence in Frequency (EOF)" to asses the degree of potential risk of a given storm against each of the main 4 types of failure mode (sliding, overturning, bearing capacity and circular slip).

Susami West Breakwater, a composite caisson breakwater located in Japan, was damaged by high waves caused by typhoon Tokage on the 20th of October 2004. The failure was reported by Kim et al. (2005)[5], who recorded the displacement of the caisson breakwaters and analysed the failure mode, characterised by the sliding/tilting of the caissons and the removal of the concrete armouring blocks. By using the method of Kim and Takayama (2004)[6] they were able to reproduce the sliding distance of the caisson, though the method used could only be applied as the tilting at the end of the storms was know (this method provides no way to forecast the tilt in a caisson).

2. TYPHOON TOKAGE

(1) Background and location

The typhoon started forming on October 12th 2004 around 480 miles east-southeast of Guam and eventually made landfall over Tosa-Shimizu on the 20th of October 2004, near the southern tip of Shikoku. The highest measured wind gust was 142 mph (63.7 m/s) at Unzendake, Nagasaki. In the Wakayama Prefecture 45 sites and 25 fishing ports (including more than 30 fishing boats) were damage by Typhoon Tokage. One of the fishing ports damaged was that of Susami.

(2) Estimation of wave climate

Kim et al (2005) [5] used GFS ("Global Forecast System") and SWAN ("Simulating Waves Nearshore") in order to estimate the heights of the offshore waves due to typhoon Tokage. Based on this data they used the EBED model by Mase (2001) [7], to estimate the incident wave at Susami West Breakwater, obtaining a $H_{1/3}$ of between 6 and 6.3m and a T=14s around the breakwater head and in the area were the breakwater joins the land. For the central area of the breakwater they obtained $H_{1/3}$<6m, which explains why this area was not as badly damaged as the other areas. Fig. 1(a) and 1(b) shows the cross section of the breakwater before and after the storm. Although Kim et al. (2005) [5] actually provide no data of the tilt of the caisson, Fig. 1(b) could be approximated by careful examination of the photographs provided by these authors.

As the design wave height of the breakwater was 3.8m (4.9m for the head section) it is normal that the caisson suffered significant damage.

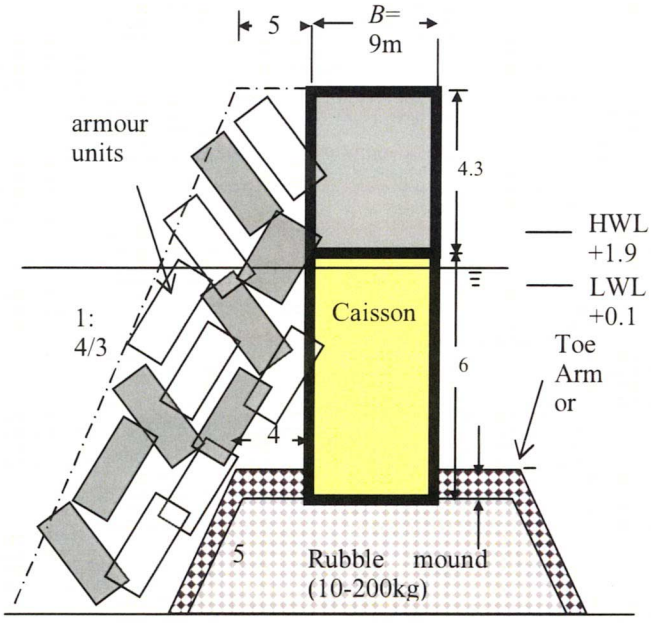

Fig. 1(a). Design Cross Section of Susami West Breakwater

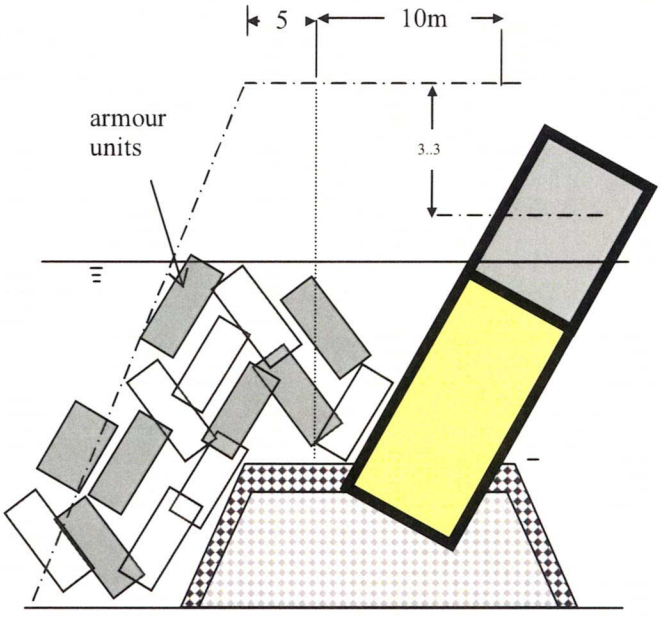

Fig. 1(b). Cross Section of Susami Breakwater after the storm.

3. COMPUTATIONAL METHOD

The methodology of Esteban et al. (2007) [8], was used to estimate the damage to Susami West Breakwater.

Shimosako and Takahashi (2000) [9], proposed that the equation of motion that describes the sliding should be:

$$(\frac{W}{g} + M_a)\ddot{x}_G = P - F_R - F_D \qquad (1)$$

where P is the horizontal wave force, $\ddot{x}_G$ the acceleration at the centre of gravity of the caisson, M_a the added mass, F_R the frictional resistance force, F_D the force related sliding velocity including the wave-making resistance force, W the caisson weight in air and g the gravity.

In addition, a force F_W induced by the rotation of the caisson and the wedge of material accumulated behind the caisson due to sliding must be included:

$$(\frac{W}{g}+M_a)\ddot{x}_G = P - F_R - F_D - F_W \quad (2)$$

This F_W is similar to the force $R(\theta(t))$ first introduced by Kim and Takayama (2005) [10], which was proportional to the weight of the material above the hypothetical sliding plane of the caisson. Esteban and Shibayama (2006) [11] proposed an alternative way to calculate this force based on the vertical deformation of the caisson and a generalized bearing capacity of the foundation gravel in the horizontal direction.

In the simplified model of Shimosako and Takahashi (2000) [9], it is assumed that the friction coefficient μ takes a constant value i.e. it represents both the static and dynamic coefficients. Takagi and Shibayama (2006) [12] showed quantitative evidence that F_D can be neglected if the duration of the effective impact is small enough. Consequently the above equation can be rewritten in the form:

$$(\frac{W}{g}+M_a)\ddot{x}_G = P + \mu U - \mu W' - F_W \quad (3)$$

where W' is the caisson's weight in water and U is the uplift force.

In order to evaluate the vertical displacement at the back of the caisson a similar principle to that used in the sliding calculation is followed. Esteban et al. (2007) [8] showed how this can be given by the expression:

$$\left(\frac{W}{g}+M_a\right)\ddot{x}_E = \left(\frac{2\cdot P_f + W'}{B}s\right) - q_U \cdot s \quad (4)$$

where $\ddot{x}_E$ is the acceleration at the edge of the caisson, B is the width of the caisson, P_{fs} is the total pressure applied to the foundation by one wave, s is the section over which this pressure is applied and q_U is the bearing capacity of the foundation. Crucial to obtain accurate results is the value of q_U, which depends on the void ratio of the foundation material.

This method was developed to determine the movement of caisson breakwaters subjected to wind waves, though the breakwater analysed had wave dissipating concrete units placed in front of it. In their analysis Kim et al. (2005) [5] ignore the effect of these blocks as they state that their effect on caisson sliding has not been clarified up to now. By doing this they claim to provide a conservative answer of the displacements that can be expected for a storm of the given intensity.

However, in order to investigate the possible effects that the wave-dissipating concrete units could have on the caisson displacement, two different cases were investigated in the present study:

- Case 1: The concrete blocks do not protect the caisson against the force of the wave, but they reduce the depth in front of the breakwater (d) to 0.7m. Hence the caisson is treated as if the water depth directly in front of it was quite low.
- Case 2: The concrete blocks in front of the caisson are completely ignored in the simulation.

Tables 1 and 2 show a summary of the parameters used in the computation of the caisson deformation.

Table 1 Basic Parameters of Simulation

Parameter	Symbol	Value	Unit
Water depth	h	7	m
Incident Significant Wave Height	$H_{1/3}$	6.3	m
Weight of caisson in air	W	180	t/m
Duration of storm	-	2	hrs
Unit mass of sea water	ρ	1.03	t/m^3
Incident angle of wave to normal of breakwater	-	0	degrees
Friction factor	μ	0.6	-
Density of caisson	ρ_c	2.177	t/m^3

Table 2 Soil mechanics Parameters

Parameter	Symbol	Value	Unit
Active depth of foundations	d_z	1	m
Initial void ratio of gravel	e	0.6641	-
Density of gravel	ρ_s	2.002	t/m^3
Angle of friction of gravel	ϕ	35	degrees

Table 1 shows how the computation used a $H_{1/3}$=6.3m for a storm duration of 2 hours. These parameters are similar to those used by Kim et al. (2005) [5]. The computational methodology of Esteban et al. (2007) [8] essentially reproduces each of the waves that occurs during the whole duration of the storm and calculates the displacements that occur due to each individual wave using eqs 1 to 4. The caisson thus fails gradually, with each wave moving it by further away from its original position. Due to the refraining effect of tilting on first described by Kim and Takayama (2005) [10], the caisson movement gradually slows down through the storm. The methodology proposed by Esteban et al. (2007) [8] then uses a Monte Carlo simulation to obtain a probabilistic answer of the expected distribution of sliding and tilting.

4. COMPUTATIONAL RESULTS

Figs. 2 and 3. show the probability distributions of sliding and vertical movement at the back of the caisson, respectively, for each of the two cases considered. From these it can be seen how the deformations expected in Case 1 are lower than those for Case 2. Although with regards to sliding both cases provide an accurate estimation of the range of displacement that actually took place, in terms of vertical displacement Case 1 cannot predict displacements of 3.3m. Case 2 (ignoring the wave-dissipating concrete units) produces a range of vertical displacement with an average of around 0.6m and a maximum value of 2m, which appears to be of the right order of magnitude to that observed in the photographs.

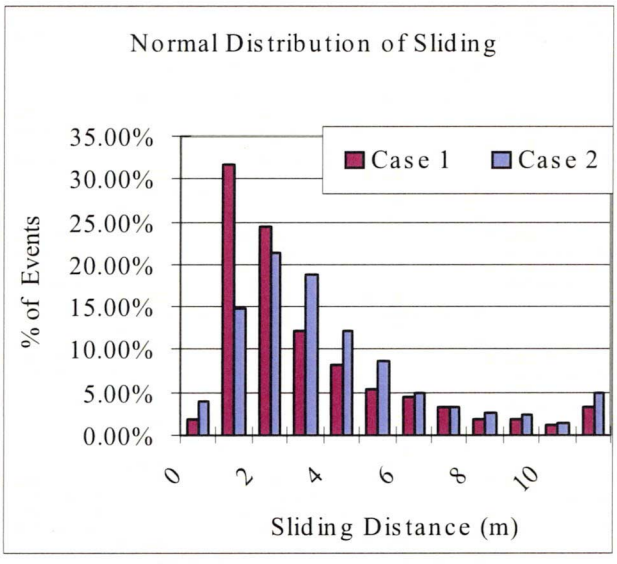

Fig. 2. Probability distributions of sliding

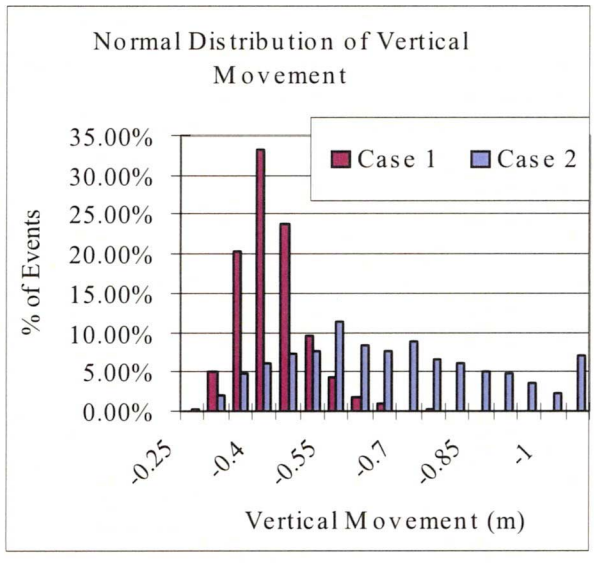

Fig. 3. Probability distributions of vertical displacement

5. COMPARISON WITH RESULTS OF KIM AND TAKAYAMA (2005)

Kim and Takayama (2005) [5] computed the expected sliding distance using two different methods:

1. Kim and Takayama (2003) [13], is a simple modification of the method of Shimosako and Takahashi (1998[14], 1999[15]) but with the introduction of a doubly-truncated normal distribution
2. Kim and Takayama (2004) [6], which considers the effect of caisson tilting on the sliding distance.

For case 1 (tilting is not included) their computation gives sliding values of over 20m, indicating that the caisson should have been washed away, which clearly did not happen. Case 2 on the other hand is able to reproduce the sliding distances that actually took place (between 6 and 10 meters). However Kim and Takayama (2004) [6] provide no way of measuring the tilting of the caisson. Hence although it is possible to reproduce the results of a caisson that has already failed (as the final angle of tilting is known) it is not possible to predict what kind of hypothetical deformation would take place against different possible types of waves. It is very important that a model exists that allows engineers to forecast the deformation that could take place in order to correctly assess the risk to the population and areas a caisson is built to protect.

The model of Esteban et al. (2007) [8] is able to

predict both the range of sliding and tilting that can be expected. Each of the caissons that made the breakwater slid by a different amount, falling within the probability distribution curves shown in Fig. 2. Though by looking at the photographs the maximum vertical displacement appears to be almost 3.3m, most of the caissons had much more limited vertical deformation, agreeing well with the result in Fig. 3. It thus appears that although the model failed to predict the maximum vertical deformation observed, it was able to predict adequately the range of most of the vertical deformations that took place.

6. DISCUSSION

There are a number of sources of uncertainty in the present analysis. First and foremost regards the value of $H_{1/3}$ that was used, which was derived from a computer model based on the deepwater wave height. As there was no recording of the actual $H_{1/3}$ during the storm, there is a strong possibility that the actual wave forces could have been different. This, if fed into the model, could have resulted in greater vertical displacements while only marginally changing the sliding distance. Also, to be noted is that Kim and Takayama (2005) [5] use a 2 hr storm duration in their computation, without stating whether this was the actual duration or not. However, due to the fact that the movement of the caisson gradually slows due to the restraining effect of caisson tilting, it is unlikely that even if the storm was longer the overall effect on the computation would have been greatly changed. Essentially the first half an hour or so of the computation produces the greatest movements in the caisson.

The friction factor is very important in determining the final probability distribution function of sliding and tilting. This factor is given the same probability distribution used by Shimosako and Takahashi (2000) [9]. The other parameters given in table 1 are also varied stochastically according to the parameters indicated by these authors. Stochastic variations in these parameters (and specially the friction factor) are responsible to a great extend to the differences in expected movements shown in Figs. 2 and 3).

The gravel parameters that were used in the present computation were the generic values for typical gravel foundations and hence there is a possibility that these differ from the actual values at Susami West Breakwater. However, it is unrealistic to attempt to carry out tests on the type of foundation material present at the breakwater due to the cost involved in doing so. Differences between these parameters and the actual parameters can account for some of the reasons why the maximum vertical displacement that occurred cannot be successfully predicted by the present model. If the actual strength of the rubble mound was weaker than that used in the present model then it can account for this one occasion of greater movement. A very important consideration is to what extent the foundation of the breakwater was consolidated during the construction phase. The values used in Table 2 assume that the foundation was adequately compacted. However, if this was not the case then the foundation could have significantly lower strength in some areas, which could also help to explain some of the movements.

The active depth of foundations is a value that is particularly crucial to the final determination of the vertical displacement. Esteban et al. (2007) [16] suggest how a value of 1 is appropriate for most small or medium size breakwaters, although this is a simplification of a complex soil deformation mechanism that takes under the soil. This value of 1 should probably be altered stochastically though at present there is no data on what would be the appropriate parameters.

The most crucial uncertainty lies however in what effect the wave-dissipating concrete units have in the simulation. It is not clear at what stage of the storms these blocks failed, though it could be assumed that the failure was progressive throughout the storm. Laboratory tests should be carried out to ascertain what protection these blocks still offer to the caisson after they have failed.

The results of the present simulation suggest that it appears the armor units would offer very little protection once they have been partially removed as shown in Fig. 1(b). That would explain why Case 2 provides much better results, and it could be that Case 1 (reducing the water depth in front of the caisson to 0.7m) does not accurately reproduce the waves that would have arrived at the caisson. Experiments on exactly what degree of protection these caissons can offer should be carried out, as the combined mode of failure of a composite type breakwater is still not properly understood.

All these factors explain why the present results should be viewed with a degree of caution, although the sliding distances of all blocks and the majority of the tilt angles were successfully predicted by the model proposed by Esteban et al. (2007) [8].

7. CONCLUSIONS

The model proposed by Esteban et al. (2007) [8] was able to replicate the damage that actually took place at Susami West Breakwater in 2004. Though one of the caissons suffered greater vertical

movement that what the model could predict, all the other caissons appeared to fit into the computed probability distribution functions. All of the caisson's sliding distances fell within the obtained distributions, and thus it appears that the model is adequate for forecasting the damage that would take place on a breakwater that was subjected to $H_{1/3}$ higher than it was designed for.

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