The paper was published in the proceedings of the 9th Australia New Zealand Conference on Geomechanics and was edited by Geoffrey Farquhar, Philip Kelsey, John Marsh and Debbie Fellows. The conference was held in Auckland, New Zealand, 8 – 11 February 2004.

Wave Impact on Caisson Breakwater

C. F. Leung
Associate Professor, Centre for Soft Ground Engineering, National University of Singapore, Singapore
X. Y. Zhang
PhD Candidate, Centre for Soft Ground Engineering, National University of Singapore, Singapore
F. H. Lee
Associate Professor, Centre for Soft Ground Engineering, National University of Singapore, Singapore

Summary: Previous case studies have revealed that a major cause of failures of vertical breakwaters is due to impact of wave breaking. A centrifuge model study is therefore carried out to investigate the responses of caisson breakwater subject to impulsive breaking wave. An in-flight wave simulator with high excitation frequency using servo-controlled electric actuator has been developed. Progressive seaward and landward caisson settlements and horizontal movement of the caisson breakwater were found to increase steadily with the number of load cycles with much of the deformation occurs within the first 1000 wave cycles. The increase in pore pressure in the foundation soil reduces the effective stresses and hence results in larger cyclic settlement of the caisson breakwater.

INTRODUCTION

A caisson breakwater is a box-type structure that is sunk through water to the prescribed founding depth to protect the coastline from wave attacks. Existing field case histories on vertical breakwaters revealed that many collapses of breakwaters were caused by the destructive impact forces on the breakwaters arising from wave breaking, such as the cases of Mutsu-Ogawara port (Hitachi, 1994) and Sakata Port (Takahashi et al., 1994a). Oumeraci (1994) reported that a major concern is the effects of breaking wave force on the breakwater foundation through the rocking and swaying motions of the caisson. The wave impact can lead to oscillatory motion and permanent deformations of the caisson breakwater and pore pressure build-up in the foundation soil. In the present study, a centrifuge model study is carried out to investigate the responses of caisson breakwater subject to breaking waves. A wave simulator system is developed to faciliate the application of cyclic impact wave on the caisson breakwater while the centrifuge is in-flight. The centrifuge model setup including the wave simulator and control system and details of typical test results are reported in this paper.

BREAKING WAVE FORCE

Several methods are available to estimate the impact force due to breaking waves. However, these methods tend to produce vastly different predications. By comparing the measured force on a breakwater in the field, the wave pressure distribution developed by Goda (1985) shown in Figure 1 is found to produce a more reasonable estimation of wave impact force than those earlier methods (for example, Longiniv, 1969).

In the present study, the extended Goda formula developed by Takahashi et al. (1994b) is employed to determine the breaking wave force on a vertical breakwater. The extended method is an improvement of the original one as it considers the force due to frequent wave breaking close to and at a vertical breakwater. Figure 2 shows the transition from non-breaking to impulsive wave pressure. During wave breaks, the most damaging scenario involves the impulsive pressure acting on the front face of the caisson breakwater as shown in Figure 2(c). The massive impulsive pressure only acts for a short time on the caisson during each wave break. It is rather difficult to develop a wave peddle stroke system to simulate the breaking waves shown in Figure 2(c) because of space constraint in the centrifuge model. A relatively simple wave simulator is fabricated in the present study such that it would impart an appropriate transitory load on the model caisson according to the period of the breaking wave. The magnitude of equivalent impact load is derived using the extended Goda formula.

In Singapore, the wave height during monsoon seasons is relatively short and is up to 1.5 m with a period to 5 s. Using the extended Goda formula, the breaking wave force is determined to be 3600 kN. For a centrifuge model, the approriate prototype/model scaling relationship for the breaking wave force and period can be derived as $1/N^2$ and $1/N$, respectively where N is the centrifuge acceleration field. The required centrifuge model wave parameters and force under 100 g acceleration field can hence be determined as given in Table 1.

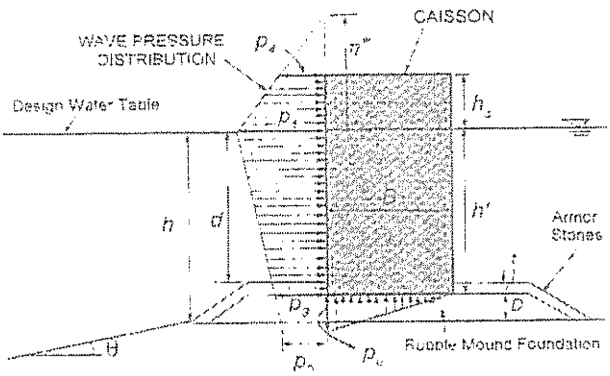
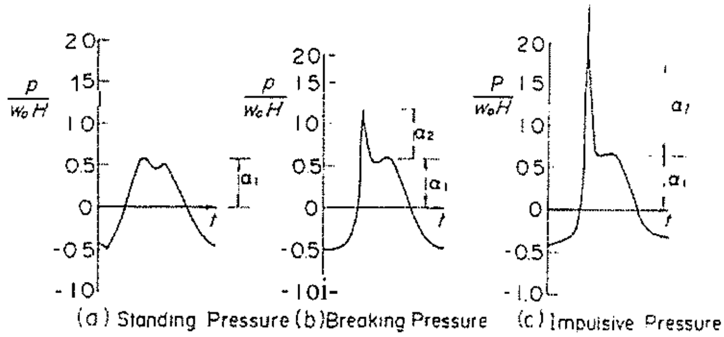


Figure 1. Distribution of Wave Pressure on Breakwater (After Goda, 1985)



Note: P/w_oH =wave induced pressure/(water unit weight x wave height)

Figure 2. Transition of Wave Pressure (After Takahashi et al., 1994b)

Table 1. Summary of Scaling Relations between Model and Prototype

Wave Characteristics	Prototype	Centrifuge Model at 100g
Wave height (m)	1.5	0.015
Wave frequency (Hz)	0.02	20
Wave period (s)	5	0.05
Wave force (kN)	3600	0.36

CENTRIFUGE MODEL SETUP

Experimental Setup

The centrifuge model tests were conducted at 100g and the model setup is shown in Figure 3. Details of the setup have been described by Leung et al. (1997) who investigated the movements of gravity caisson wharf structure subject to static dead and live structural loads using centrifuge modelling technique. The rectangular strong box model container is made of stainless steel and has internal dimensions of 570x200 mm^2 and a height of 470 mm (47 m in prototype scale). The 150-mm thick (15 m) sand bed consists of uniform medium sand with D_{10} = 0.35 mm, uniformity coefficient = 0.32, minimum void ratio = 0.72 and maximum void ratio = 1.07. When preparing the sand bed, the sand is pluviated from a hopper into the container. The relative density of sand prepared by this method falls within the range of 55% to 60%. Instead of using water as in the earlier study, silicone oil, which has a specific gravity of 0.96 and a viscosity of 100 cSt (at 25° C) is used as the pore fluid to preserve the similitude of dynamic and consolidation time scaling at 100g. Vacuum saturation method was used to saturate the sand bed.

The model caisson is 250 mm high (25 m in prototype scale), 180 mm wide (18 m) and 200 mm long (20 m) spanning across the whole width of the model container. The caisson has 20 mm (2 m) base protrusions at both ends of the caisson base. Four displacement transducers (V1, V2, V3 and V4 shown in Figure 3) are employed to monitor the settlement at the front and rear sides of the caisson. Another displacement transducer (H1) is used to measure the horizontal movement of caisson. The changes in pore pressure in the soil due to the impact force are measured at six locations, one of which (P6) is beneath the mid-base of caisson, two of which are beneath the caisson toe (P4 and P5) while another three (P1. P2 and P3) are beneath the caisson heel, see Figure 3. Two pore pressure transducers (P7 and P8) were placed inside the caisson to measure the weight of liquid inside the caisson and hence the caisson dead load can be determined.

Closed Loop Control System

The wave simulator is controlled using a controller placed on the centrifuge beam. A schematic diagram of the wave simulator control system is shown in Figure 4. The personal computer (PC) that sends signals to the controller is located in the centrifuge control room while the wave simulator system and the controller are on the centrifuge. During tests, the controller and the PC communicate via a program HyperTerminal that can transfer large files from the PC onto the controller using a serial port and hence facilitates the control of wave force when the centrifuge is in-flight. The PTS (Programmable Transmission System) software in the controller sends over 200 commands each time to ensure a rapid project completion. The input with appropriate wave force magnitude and period will be delivered from the PC to the wave simulator via the controller and the resultant wave impact force is recorded by the load cell attached on the head of torque arm of the actuator at a frequency of 300 Hz.

The wave simulator apparatus employs a load-control approach via a brushless DC/AC servo motor as shown in Figure 5. It consists of three main parts, namely torque arm, gear box and electric motor. The servo motor can produce a variety of waveform and feedback combinations. The high dynamic response is achieved by incorporating "High Energy" Samarium cobalt magnets combined with low inertia rotors. The gear box beneath the servo motor effects movement of the torque arm and hence applies a force on the caisson with a desired period.

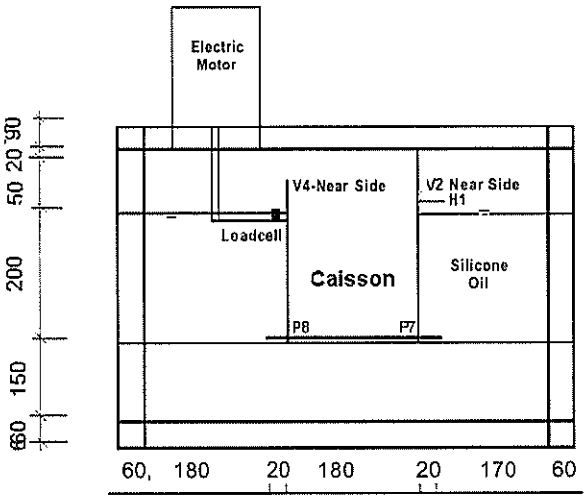


Figure 3. Experimental Setup and Instrumentation (all dimensions in mm)

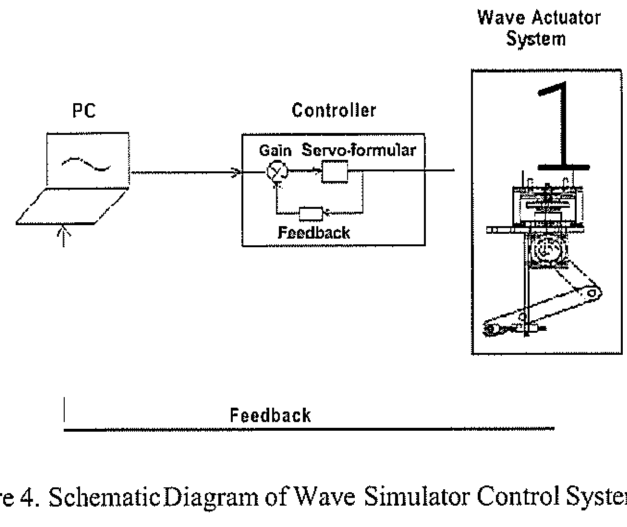


Figure 4. Schematic Diagram of Wave Simulator Control System

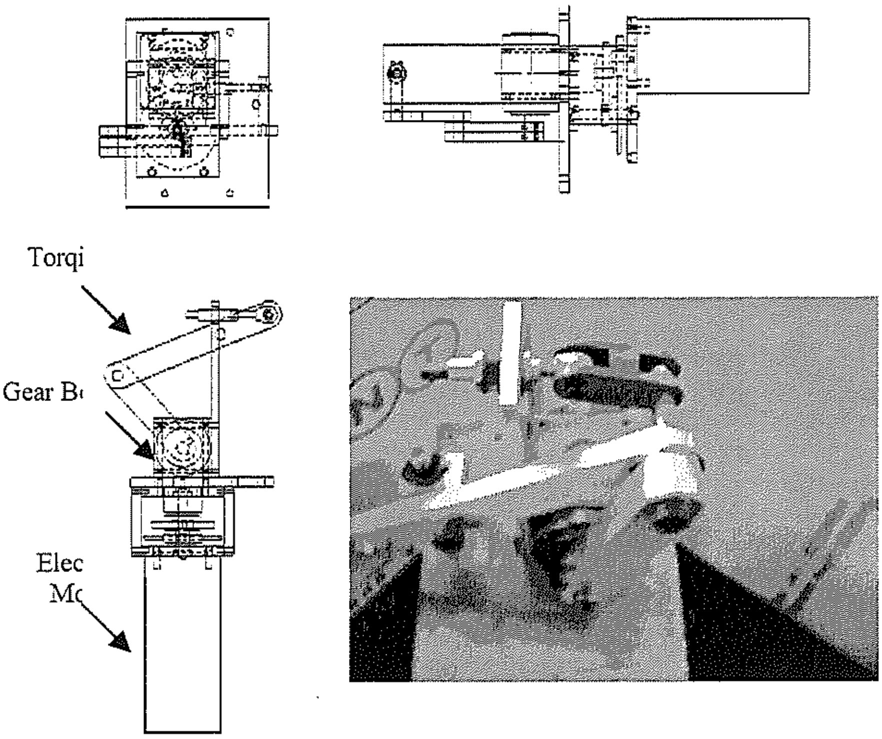


Figure 5. Design of Wave Simulator Apparatus

EXPERIMENTAL PROCEDURE AND RESULTS

There are two stages in the experiment, namely caisson infilling stage, and wave and live loading stage. The sample rate of measurement during the infilling stage is 20 Hz for each instrument, while that during wave and live loading stage is 300 Hz. In subsequent presentation, the results are reported in prototype scale. A positive vertical movement value denotes a downward settlement of the caisson while a negative horizontal movement value denotes that the caisson is moving landwards and vice versa.

Once the centrifuge model achieves equilibrium at 100g, heavy zinc chloride solution ($ZnCl_2$) is introduced into the caisson to simulate the ballasting process. Figure 6 shows that there is a rapid and large increase in caisson loading pressure and settlement. The caisson settlement increases almost linearly with load and the magnitude is about 48 mm at both landward and seaward sides of the caisson. Little horizontal caisson displacement is noted as the load applied at this stage is vertical.

Impulsive wave load was then applied onto the front surface of the caisson and this lasts for 10 minutes with about 10,000 load cycles, giving an average frequency of about 20 Hz. A typical wave-loading pattern is shown in Figure 7. At the 2^{nd} minute of wave loading, live load was applied to the caisson by infilling another appropriate quantity of $ZnCl_2$ into the caisson. By averaging the measured cyclic movements over a window of 50 wave cycles, the average movements can be computed. Figure 8 shows the average caisson pressure and movements plotted against the number of wave cycles in a semi-log scale. The movement of the caisson breakwater may be divided into three stages, which are termed hereafter as "unlocking", "settlement" and "*stabilization".

The unlocking stage is characterized by a relatively low rate of settlement increase during the first 100 cycles. A possible explanation from the microstructure viewpoint for the initial low rate of settlement is proposed below. During the course of infilling, sand particles are continuously rearranging themselves as the soil skeletons are compressed to form a denser interlocking matrix to resist higher load increments (Lambe and Whitman, 1979). Upon application of the wave load, the soil skeleton remains in compression under the action of "locked-in" stresses in both vertical and horizontal directions. Additional efforts are hence required to release the "locked-in" stresses before more deformation can take place. According to Schofield (1981), the effective stresses between interlocked soil particles may be reduced by imposing tensile strain, by increasing pore water pressure, or by cyclic loading, which leads to relaxation of stresses between particles. The latter may be applicable to caisson breakwater as there is a release of stresses during the initial stages of cyclic loading, upon which the stresses are "unlocked*.Ng and Lee (2002) also observed such an "unlocking"phenomenon in the case of cyclic loading on preloaded spudcan. It was found that even though vertical stresses have been reduced, the soil skeleton is stabilized by the 'locked-in" horizontal stresses. It took at least 20 load cycles to accomplish the unlocking stage for the preloaded spudcan. However, in the present study, it took up to 100 wave load cycles to release the "locked-in" soil stresses. This is probably attributable to the much larger "locked-in" stresses for a caisson breakwater as compared to that of preloaded spudcan and thus more wave cycles are required to release the "locked-in" stresses.

After 100 or so wave load cycles, the stress state may be on or close to the yield surface, any incremental load will lead to the stress state moving out of the yield surface and hence plastic deformation being induced. The caisson settlement increases steadily with the logarithm of wave cycles and by then approximately 90% of final settlement has taken place as indicated in Figure 8. The progressive seaward and landward settlements of the caisson breakwater increase exponentially from 200 to 800 or so wave cycles. After 800 wave cycles, there is a sudden increase in caisson settlement because live load is applied to the caisson, as indicated in the large increase in caisson pressure in Figure 8. It is also found that much of the deformations occurr in the first 1000 cycles. The progressive densification of *dry* sand subjected to cyclic loading has been reported by Silver and Seed (1971). The dynamic shear modulus was found to increase slightly with increasing numbers of cycles and soil relative density. Progressive deformations were observed in all tests with the vertical strain increasing rapidly in the initial stages and slowing down as the number of cycles increased. This is consistent with the observed progressive settlement responses in the present study. The impulsive wave loads are transferred to the sand bed through the rocking and swaying motions of the caisson breakwater and hence the soil loses part of its strength progressively, resulting in large permanent deformations. As the progressive landward settlement is larger than the seaward one, the caisson breakwater tilts landward. This is considered reasonable because during the course of wave loading, the caisson breakwater tends to rotate landwards and this leads to a larger compressive stress under the caisson heel. The trend of the horizontal movement of caisson is the same as the settlement and the caisson slides 32 mm landwards.

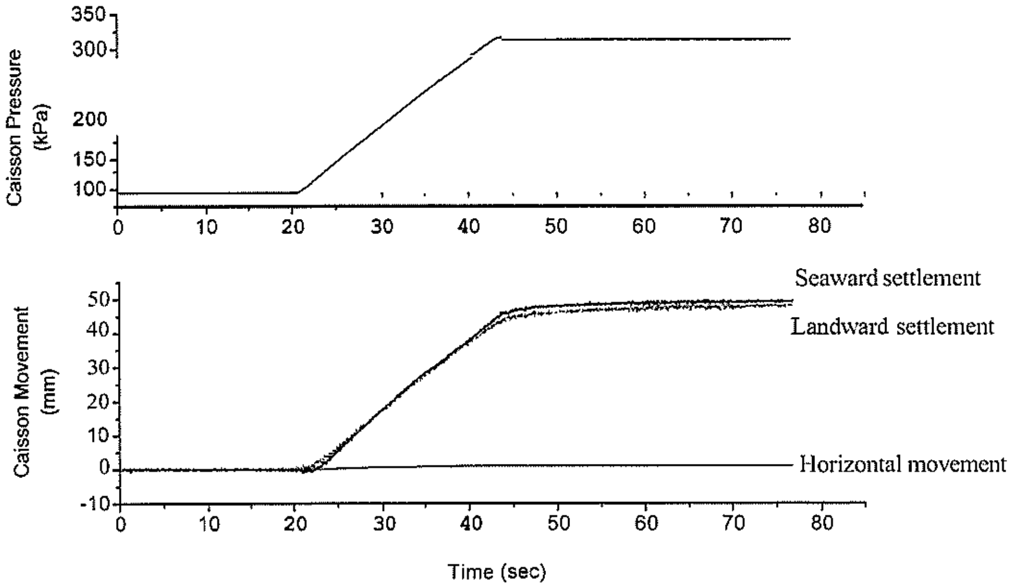


Figure 6. Caisson Movements and Pressure in Infilling Stage

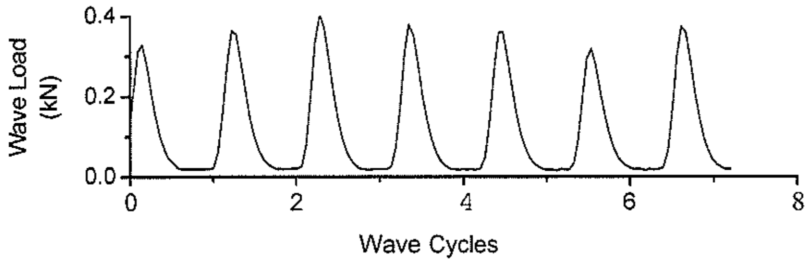


Figure 7. Wave loading Pattern (wave load in model scale)

After the "settlement" stage, the caisson breakwater enters the "stabilization" stage where the settlement value starts to tail off towards an asymptotic value. This happens after about 1000 cycles of wave loading. The rate of increase in settlement in this stage is very low. Towards the end of the "stabilization" stage, virtually no progressive movement is detected indicating the soil has a constant void ratio.

Figure 9 shows the average pore pressure-log time responses obtained by averaging readings for each 50 wave cycles with the location of pore pressure transducers given in Figure 3. Although some transducers may have moved during the test, their movements are unlikely to affect the respective pore pressure responses significantly. It is found that before 100 or so wave cycles, the excess pore pressure build-up is quite low during the "locked-in" stage. The pore pressures then reach the peak value in the 100 or so wave cycles. After about 1000th wave cycles, the second peak excess pore pressures emerge due to further infilling of $ZnCl_2$ solution. After this, the positive pore pressures appear to dissipate gradually and fluctuate around the hydrostatic value. The increase in positive excess pore pressures reduces the effective stresses in the oil-saturated sand bed and resulted in larger cyclic settlement of the caisson breakwater. This behaviour is similar to that reported by Luong and Sidaner (1981) who postulated the CL (characteristic state line) divides the sand behaviour into two regimes: (a) sub-characteristic domain--: densification or generation of positive excess pore pressure for stress path cycling below the CL; (b) sur-characteristic domain--: dilatancy or generation of negative excess pore pressure for stress path crossing above the CL. Hence, the densification mechanism may be dominant in the sand skeletons under the stressed zone. Further studies should be carried out to investigate the effects of breaking waves on the behaviour of caisson breakwaters supported on sand of different relative densities.

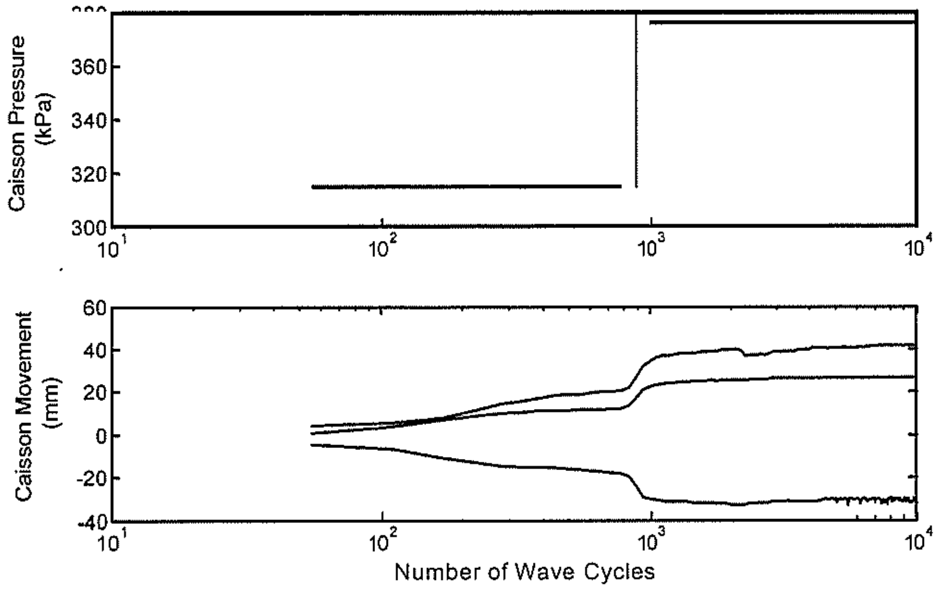


Figure 8. Average Caisson Movement during Wave and Live Load Stage

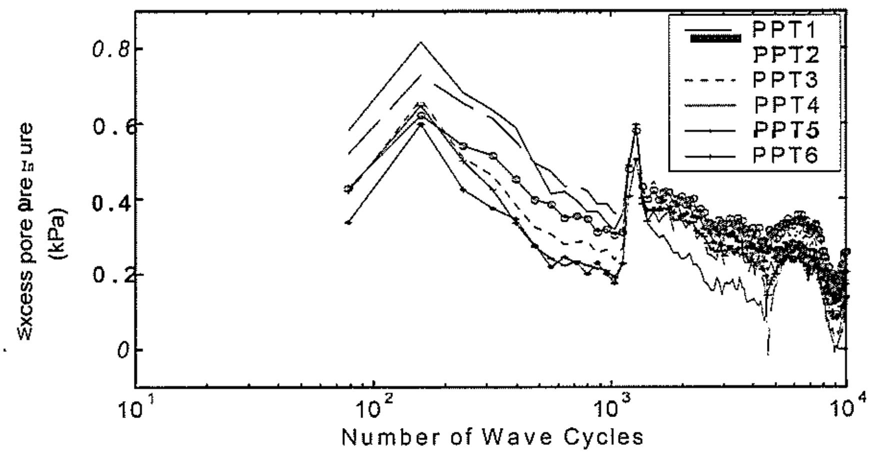


Figure 9. Average Excess Pore Pressure History during Wave and Live Load Stage

CONCLUSIONS

Previous studies have established that breaking waves can be detrimental to foundations supporting vertical caisson breakwaters. In view of this, a wave simulator control system was developed to enable the application of loads simulating breaking waves on the caisson breakwater during centrifuge flight. The development of the actuator is described in detail in this paper. Centrifuge model tests were then carried out to evaluate the performance of caisson breakwaters subjected to impulsive wave loads. The magnitude of the impulsive wave force is calculated using the Goda formula. The results of one typical test are reported in detail in this paper. It is established that:

(1) The caisson movements increase steadily with increasing number of wave cycles and can be divided into three stages termed "unlocking", "settlement" and "stabilization".

(2) The impulsive wave loads are transferred to the sand bed through the rocking and swaying motions of the caisson breakwater and hence the soil loses part of its strength progressively, resulting in increasing permanent caisson deformations.

(3) Based on excess pore pressures measured in the sand bed, it is found that densification mechanism may be dominant in the sand skeletons under the stressed zone when the caisson breakwater is subject to wave and live loading.

ACKNOWLEDGEMENTS

The study is sponsored by the National University of Singapore Research Grant number R-264-000-119-112. The assistance of Centrifuge Laboratory staff in the conduct of centrifuge model tests is also gratefully appreciated.

REFERENCES

Goda, Y. (1985). "Random seas and design of maritime forces," *University of Tokyo Press,* Tokyo, Japan.

Hitachi, S. (1994). "Case study of breakwater damages Mutsu-Ogawara port," *Proc. Int. Workshop on Wave Barriers in Deepwaters, Port and Harbour Research Institute,* Yokosuka, Japan, pp. 308-329.

Lambe, T. W. and Whitman, R.V. (1979). "Soil mechanics," *SI version. John Wiley & Sons.*

Leung, C.F., Lee, F. H. and Khoo, E. (1977). "Behavior of gravity caisson on sand". Journal of Geotechnical and GeoenvironmentalEngineering, ASCE, Vol. 123, pp. 187-196.

Loginov, V.N. (1969). "Nonlinear oscillations of vertical breakwaters subject to wave impact loads," *Trudy Soiuzmorniiproekta.* Vyp.2, pp. 47-59 (in Russia).

Luong, M.P. and Sidaner, J.F. (1981). "Undrained behaviour of cohesionless soils under cyclic and transient lading," *Proc. Int. Conf. Recent Advances in Geotech. Earthquake Engineering and Soil Dynamics,* Missouri, Vol.1, pp.215-220.

Marinski, J. G. & Oumeraci, H. (1992). "Dynamic response of vertical structures to breaking wave forces-review of the CIS design experience", *Proc. of 10th Coastal Eng. Conf.*, ASCE, pp.1357-1370.

Ng, T.G and Lee, F. H. (2002). "Cyclic settlement behaviour of spudcan foundations," *Geotechnique* ***52,*** No. 7, pp. 469-480.

Oumeraci, H. (1994). "Review and analysis of vertical breakwater failures – lessons learned', Costal Engineering, Vol. 22, pp.3-39.

Schofield, A.N. (1981). "Dynamic and earthquake geotechnical centrifuge modelling," *Proc. Int. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics,* Missouri, Vol.3, pp. 1081-1100.

Silver, M. L. and Seed, H. B., (1971). "Volume changes in sands during cyclic loading," *J. of Soil Mechanics and Foundations Div.*, ASCE, Vol. 97, No SM9, pp.1171-1182.

Takahashi, S. Tanimoto, K. and Shimosako, K. (1994a). "Dynamic response and sliding of breakwater caissons against impulsive breaking wave forces," *Proc Int. Workshop on wave Barriers in Deepwaters, Port and Harbor Research Institute,* Yokosuka, Japan, pp. 362-399.

Takahashi, S., Tanimoto, K. and Shimosaka, K. (1994b). "A proposal of impulsive pressure coefficient for design of composite breakwaters," *Proc. of International Conference on Hydro-technical Eng. For Port and Harbor Construction,* Port and Harbor Res. Inst., pp. 489-504.