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DEVELOPMENT OF RESIDUAL PORE PRESSURE IN THE SANDBED BENEATH A CAISSON BREAKWATER

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1. INTRODUCTION

Soil liquefaction has been suggested as one of the causes of failure of monolithic breakwaters and further marine structures. This phenomenon is generally defined as the state of the soil where the effective stress completely vanishes causing the soil-water mixture to behave like a liquid; i.e. the shear strength of the soil becomes zero as a result of the pore pressure build-up reaching the value of the initial effective stress. If the effective stress is only reduced without completely vanishing, the term "partial liquefaction" is often used. Especially the gradual increase in mean pore water pressure becomes a crucial phenomenon as it may considerably affect the stability of the foundation over a long period.

However, liquefaction solely induced by storm waves is expected to occur transiently (*instantaneous* liquefaction) and may not affect the foundation stability. For complete or even partial liquefaction to occur under storm waves, very unfavourable loading and drainage conditions for the foundation would be required, which are rarely encountered for common marine structures. This was confirmed by the results of the analysis of more than 20 failures experienced by vertical breakwaters (Oumeraci 1994). The conclusions of that study stressed the relative importance of the contribution of the geotechnical failure modes, but excluded any occurrence of complete residual liquefaction beneath caisson breakwaters. However, under the combined action of both wave and caisson motions, a considerable build-up of pore pressure beneath the caisson may occur, induced by residual soil deformations (Oumeraci 1994, Oumeraci et al. 2001). This was confirmed by large-scale model tests on a caisson breakwater performed within the European project LIMAS (Liquefaction Around Marine Structures, see Kudella & Oumeraci, 2004).

2. EXPERIMENTAL SET-UP AND TEST CONDITIONS

The tests were conducted in the Large Wave Flume (GWK) in Hanover, Germany. The flume has a length of 307m, a width of 5m and a depth of 7m. The model of the caisson breakwater was located about 240m from the wave generator.

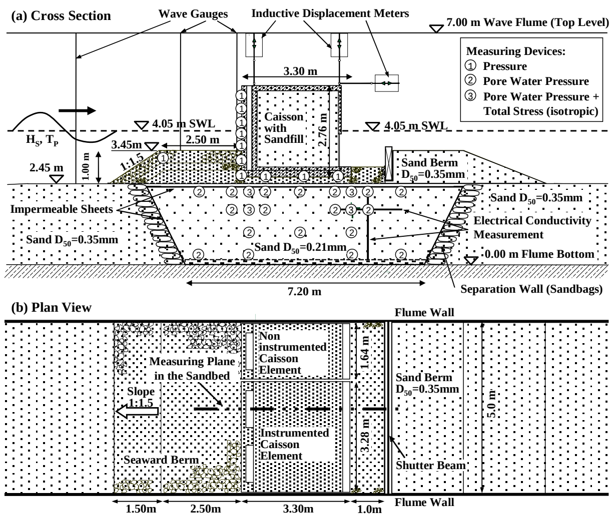

Fig. 1: Experimental set up and location of measuring devices at the breakwater model

The applied measuring devices provided information about the wave conditions, the wave load at the structure, the caisson motion and the induced pore water pressure and mean total stress inside the soil. The tests were carried out with regular waves (H=0.4m-0.9m, T=4.5s-6.5s) and wave spectra (H_s=0.4m-0.9m, T_p=4.5s-8s). The water depth was kept constant on 1.60m at the toe of the shoreward breakwater berm. In order to simulate unfavourable drainage conditions of the soil comparable to a loose sand bed with thin clay or silt layers the sand underneath the breakwater was enclosed in almost impermeable sheets (Kudella & Oumeraci, 2004).

3. DEVELOPMENT OF RESIDUAL PORE PRESSURE

The generation of the resultant residual pore pressure $u_r(t)$ inside the foundation soil is determined by two basic processes: The time dependant pore pressure generation process $\Psi(t)$ (generation rate $+du_r/dt$) and the dissipation process $\Phi(t)$ (dissipation rate $-du_r/dt$):

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$$u_r(t) = \Psi(t) - \Phi(t) \qquad (1)$$

The generation process is essentially determined by the loading conditions, which can be decomposed into two modes: i) the *wave motion mode*, where the wave motions are transferred directly through the rubble foundation inducing time-dependant pressures at the seabed and ii) the *caisson motion mode*, where the caisson motions induced by the wave load are acting as total stresses at the seabed-structure interface. It was shown that for the investigated test conditions, the caisson motion mode is clearly responsible for any increase in mean pore pressure (Kudella & Oumeraci, 2004).

The dissipation process is mainly influenced by the characteristic drainage period T_{Drain} of the soil. For the investigated structure, T_{Drain} was derived empirically, but according to Oumeraci et al. (2001) it can also be approximated by

$$T_{Drain} \approx \frac{d^2}{c_v} \qquad (2)$$

with d = length of the drainage path and c_v = consolidation coefficient. The pressure reduction due to dissipation can be described by an exponential damping function (Fig. 2).

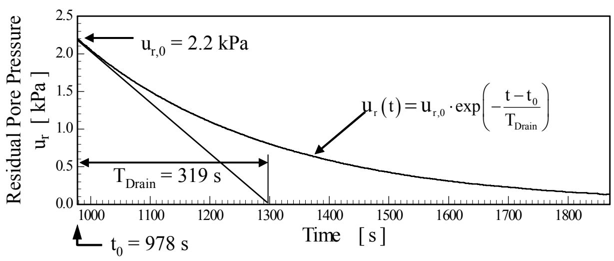

Fig. 2: Pressure dissipation after a selected test with exponential fit-function

Within the time period (t_i - t_{i-1}), the pressure reduction $u_r(t_{i-1})$ - $u_r(t_i)$ is determined by

$$u_r(t_i) = u_r(t_{i-1}) \cdot \exp\left(-\frac{t_i - t_{i-1}}{T_{Drain}}\right) \qquad (3)$$

The generation rate and the dissipation rate are not constant as the pre-shearing effect, together with its influence on the relative density and the hydraulic permeability of the soil, will affect both processes with the tendency of increasing the resistance against further pore pressure build-up.

The generated pore pressure can be determined, if the measured residual pore pressure is superimposed by the dissipation rate. This corresponds to the completely undrained situation.

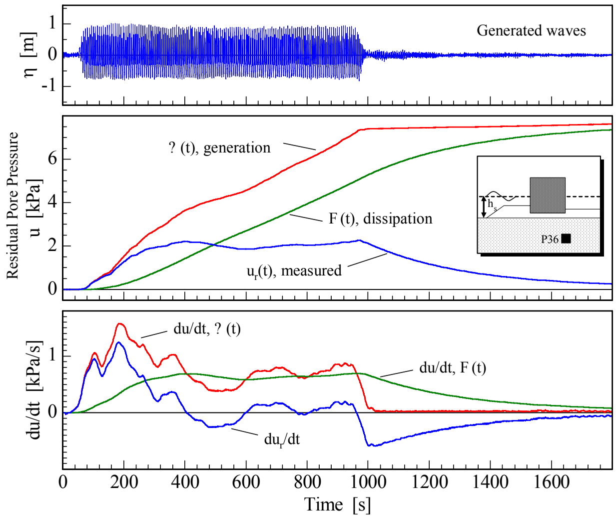

Fig. 3: Development of generated, dissipated and resultant residual pore pressure with corresponding rates of change for a selected test at location P36

The paper will present a closer examination of the balance between pore pressure generation and dissipation in order to come up with some design guidance of caisson breakwaters based on allowable caisson movements and soil deformations. This might contribute to a better assessment of safety and serviceability of marine structures.

4. REFERENCES

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