

WAVE-INDUCED INSTANTANEOUS AND RESIDUAL PORE PRESSURE IN THE SANDBED UNDERNEATH A CAISSON BREAKWATER: PROCESSES LEADING TO LIQUEFACTION FAILURES

Matthias Kudella¹; Hocine Oumeraci¹ Maarten de Groot²

1. INTRODUCTION

Safe design of caisson breakwaters has to examine all processes, which might lead to partial or total failure. Due to its complexity especially the geotechnical aspects around the foundation have to be considered very carefully (see Oumeraci et al. 2001). One of these aspects is the increase in pore water pressure inside the soil, which might occur if the structure is subjected to cyclic load and which might increase up to a critical value, thus resulting in a significant reduction of shear strength. This phenomenon is called soil liquefaction.

While an essential amount of knowledge about the oscillating pore pressure around a mean value due to wave action on a seabed without any structure has meanwhile accumulated, the influence of marine structures on pore pressure generation is still relatively unknown. This also holds for the residual set-up of pore pressure, which is therefore not yet implemented satisfyingly in design formulae.

Although significant increase of residual pore water pressure mostly occurs in saturated soils during earthquakes, partial liquefaction induced by waves may also lead to complete failures of the foundation of the breakwater as reported for instance by Zen et al. (1986). In the presence of a structure the stress, which is transferred into the soil, results directly from pressure variations on the seabed due to waves and indirectly from pressure variations due to the motions of the structure subject to breaking and non-breaking wave load.

To enhance the sustainability of caisson breakwaters the processes susceptible to lead to liquefaction inside the sand-bed below a caisson breakwater have been investigated in large-scale tests within the European project LIMAS (Liquefaction Around Marine Structures). Main focus was set on the observation and analysis of instantaneous and residual pore pressure in the sandy subsoil.

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.

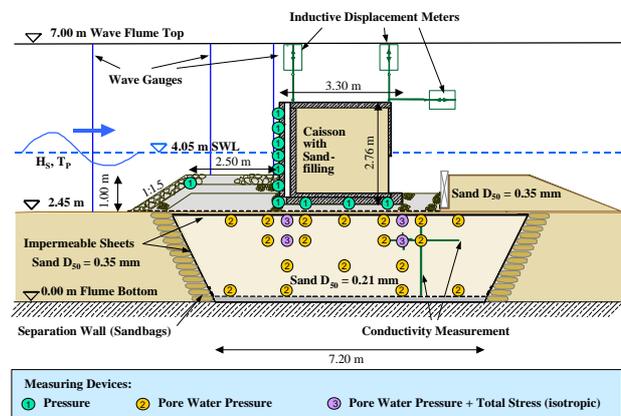


Fig. 1: Experimental set up and location of measuring devices at the breakwater model

The wave conditions are recorded by wave gauges installed shoreward and in front of the breakwater. The tests were carried out with regular waves ($H_s=0.4\text{m}-0.9\text{m}$ and $T=4.5\text{s}-6.5\text{s}$) and wave spectra ($H_s=0.4\text{m}-0.9\text{m}$ and $T_p=4.5\text{s}-8\text{s}$). The water depth was kept constant on 1.60m at the toe of the shoreward breakwater berm. The wave load on the caisson was determined by integrating discrete pressure ordinates, which were measured with pressure transducers. The response of the caisson was measured with 3 inductive displacement meters and separated afterwards in an oscillatory motion and a residual displacement resp. settlement. In order to obtain noticeable residual excess pore pressure medium fine sand ($D_{50}=0.21\text{mm}$) was used as subsoil with an initial relative density of about $D_r\approx 0.45$. The immediate dissipation of residual pore pressure was prevented by increasing the drainage period artificially by enclosing the sand underneath in impermeable sheets, which were simulating the presence of

¹ Leichtweiß-Institute of the Technical University in Braunschweig, Beethovenstr. 51a, 38106 Braunschweig, Germany e-mail: M.Kudella@tu-bs.de; resp. H.Oumeraci@tu-bs.de

² GeoDelft, Stieltjesweg 2, 2628 CK Delft, The Netherlands; e-mail: M.B.deGroot@geodelft.nl

nearly impermeable horizontal clay layers. The pore water pressure underneath the breakwater was recorded with 22 pressure transducers supplemented by the measurement of total stress at 4 locations. The change in porosity was simultaneously measured with density rods, recording the electrical conductivity of the soil in the area underneath the shoreward edge of the caisson.

3. EXPERIMENTAL RESULTS

The response of the pore water pressure due to load on the seabed can be separated in two parts i) the oscillating pressure around a mean value and ii) the mean residual increase of excess pore pressure. Fig. 2 shows the oscillating pore pressure response due to significant rocking motions of the caisson immediately after a strong impact load by a breaking wave. With increasing distance to the seabed surface the amplitude decreases. Under the seaward edge (unloading case) the pressure amplitude is even lower than under the onshore edge (loading case) although the amplitudes of the corresponding vertical motions have the same absolute value.

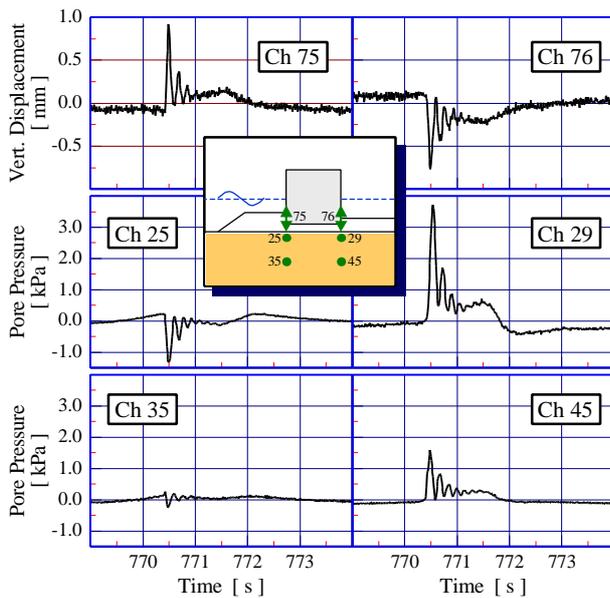


Fig. 2: Vertical oscillations of the caisson edges and pore pressure response ($H=0.7$, $T=6.5s$)

An important result of the tests is, that under the given drainage conditions only high impact loads on the caisson and the subsequent motions of the caisson structure were able to generate significant residual pore pressure. The effect of direct wave action on the seabed was negligibly small.

Residual pore pressure set-up and dissipation are responsible for the rearrangements inside the soil

skeleton resulting in densification of the seabed in the course of the tests. As a consequence the sensitivity against further residual pressure generation decreases. In Fig. 3 this effect is shown for two tests with the same wave parameters ($H=0.7m$, $T=6.5s$). The reduced increase in residual excess pore pressure corresponds to the lower settlement of the caisson.

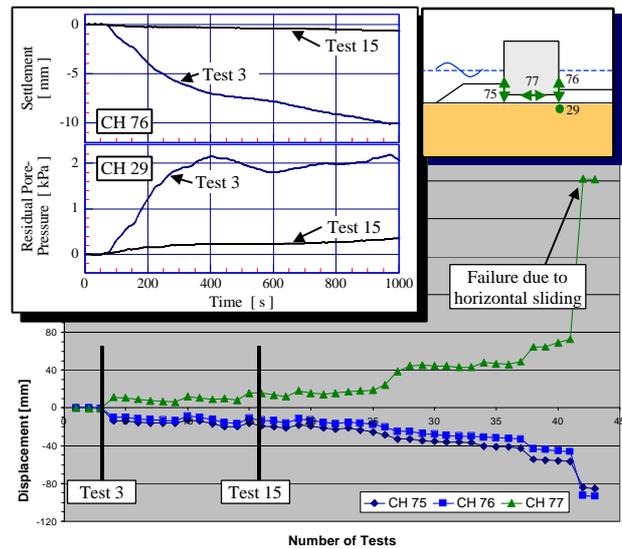


Fig. 3: Development of settlements and residual excess pore pressure at different density states of the subsoil

The processes, which are likely to cause liquefaction, will be described in more detail in the paper. Analytical solution supplemented by empirically derived terms will also be given in the paper.

4. REFERENCES

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