PORE PRESSURE DEVELOPMENT IN THE SAND BED UNDERNEATH A CAISSON BREAKWATER

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The first results of large-scale model experiments in a wave flume are discussed. These experiments are concerned with the study of the generation of transient and residual pore pressure in a seabed beneath a caisson breakwater subject to both pulsating and breaking wave loads. The simulated seabed and drainage conditions correspond to those encountered in a loose sand bed with thin clay or silt layers. Even under such unfavorable conditions total liquefaction due to residual pore pressure could not occur during the experiments. It is shown that the residual pore pressure is essentially generated by the caisson motions due to breaking wave loads and that they are closely related to residual soil deformation, which may lead to the collapse of the breakwater.

1. Motivation and Objectives

Soil liquefaction has often been suggested as one of the causes of failure of monolithic breakwaters and further marine structures (Zen et al. 1986, Chaney & Fang 1991). Soil liquefaction 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, because the shear strength becomes zero as a result of pore pressure build-up reaching the initial effective vertical stress. If the effective stress is only reduced without completely vanishing, the term "partial liquefaction" is often used. For the foundation of marine structures two mechanisms leading to total or partial liquefaction may be distinguished:

1. *Upward pressure gradient and flow beneath a wave trough:* A lift is generated in the top layer of the seabed which may exceed the overburden weight, thus resulting in a transient liquefaction during the passage of the wave trough. This mechanism is particularly relevant for the top layer of the seabed around the marine structure.
2. *Build-up of pore pressure:* As a result of successive wave loads and unfavorable drainage conditions the pressure induced in the pore fluid by each load event may accumulate to residual pore pressure. This mechanism is particularly relevant for sandy seabed beneath monolithic structures, such as caisson breakwaters where high shear stresses may develop over a large depth.

However, a total residual liquefaction, as this is often the case for earthquake loading, is unlikely to occur for storm wave loading only. This was confirmed by the results of the analysis of more than 20 failures experienced by vertical breakwaters (Oumeraci 1994). The conclusions stressed the relative importance of the contribution of the geotechnical failure modes, but excluded any occurrence of total residual liquefaction beneath caisson breakwaters. However, under the combined action of both wave and caisson motions, residual soil deformations were expected to occur, which may lead to a considerable build-up of pore pressure beneath the caisson (Oumeraci 1994, Oumeraci et al. 2001).

The present paper is therefore aimed at discussing some results of large-scale model experiments, which were conducted in the Large Wave Flume (GWK) of Hannover in order to study the generation of transient and residual pore pressure in the seabed beneath a caisson breakwater. Particular focus is put on the process, which may lead to residual pore pressure generation and to residual soil deformations. Both pulsating cyclic wave loads and successive breaking wave impacts are considered.

2. Relevant Processes and Requirements for Experimental Setup

The generation of residual pore pressure in the sand bed beneath a caisson breakwater depends on the type of waves and wave loads as well as on several parameters describing the breakwater and its foundation. Therefore, a brief overview of the processes involved in the wave-structure-seabed interaction will first be given, which allows defining the main requirements to be met by the experimental setup in the Large Wave Flume for the generation of residual pore pressure.

2.1. *Processes and Parameters*

The response of the seabed beneath a caisson breakwater subject to sea waves may be decomposed into two modes:

- *Wave motion mode:* The wave motions are transferred directly through the rubble foundation inducing time dependent pressures in the seabed.
- *Caisson motion mode:* The caisson motions induced by the wave load are transferred to the seabed as total stresses.

Superposition of these two modes provides the resulting initial load at the seabed surface. The energy input into the whole system is determined by the waves in front of the structure and their interaction with the breakwater.

The *wave motion mode* is characterized by the time dependent pressure $p_0(t)$ on the seabed surface, which is essentially governed by the water depth and the wave parameters. For the transmission of the wave motions through the rubble foundation, $p_0(t)$ is also determined by the thickness h_r and the porosity n of the mound.

The *caisson motion mode* is conditioned by the wave load on the caisson as well as by the characteristics of the caisson structure and its foundation. Depending on the magnitude, duration and frequency of the wave loads as well as on the dynamic characteristics of the structure and its foundation, the caisson structure will experience oscillatory motions and permanent displacements (Oumeraci et al. 2001).

The oscillatory motions, particularly those in vertical direction, may strongly affect the development of the uplift pressure on the bottom slab, and thus the pore pressure development in the rubble foundation (Oumeraci et al. 2001). These caisson motions are transmitted to the seabed surface as time dependent total stresses $\sigma_z(t)$, which consists of the effective stress $\sigma'_z(t)$ and the motion induced pore pressure $p_{0,c}(t)$. Vertical and rotational motions of the caisson will result in total stress variation $\sigma_z(t)$, while horizontal motions will induce initial shear stress $\tau_0(t)$ at the seabed surface.

Residual pore pressure generation depends on both wave load conditions and soil characteristics of the seabed. Most of the relevant soil parameters are not only interdependent, but also depend on the stress level and the stress history. Moreover, there is a strong interaction between the development of pore pressure and soil parameters (De Groot & Meijers 2004).

Beside the oscillatory motions, small permanent displacements caused by sufficiently high single load events may also occur. A stepwise accumulation of these displacements during a storm may lead to the ultimate collapse of the breakwater (Oumeraci et al. 2001). The mechanisms of such a stepwise accumulation are, however, not yet fully understood.

2.2. *Requirements and Implications for the Planned Experimental Setup*

Due to the relatively low frequency of cyclic wave loads and the drainage conditions of common sand beds, significant residual pore pressure is unlikely to occur. Therefore, more unfavorable conditions than those commonly encountered in prototype must be reproduced in the model in order to achieve a significant build up of pore pressure in the seabed. Focus was particularly put on the following factors, which increase the potential for residual pore pressure (for more details see Kudella & Oumeraci 2004):

1. *Large ratio of pore water stiffness and constraint modulus of the soil skeleton*: A significant transmission of pore pressure in the seabed is only ensured, if the bulk modulus of pore water is larger than the constraint

modulus of the soil skeleton. As the stiffness of pore water is strongly affected by the saturation S_r, the sand beneath the caisson was flushed to achieve the highest practicably feasible S_r-value.

2. *Appropriate relative density of sand bed*: Following the flushing process, preliminary tests (Test Phase 1) using regular waves with increasing wave steepness H/L were used to obtain a relative density that could well be present in a natural fine sand seabed, such as $D_r \approx 0.4$.
3. *Large ratio of characteristic period of drainage T_{drain} and wave load T_{load}*: To achieve a significant pore pressure build-up between successive load events, the period of the characteristic drainage period should be much larger than the cyclic wave load T_{load}. The dissipation rate of the pore pressure is governed by the hydraulic permeability k, which itself is essentially determined by the grain size distribution. Therefore fine sand was selected. Preliminary calculations made clear, however, that this would not be sufficient. To increase the drainage period impermeable PEHD-sheets with gaps at the sheet joints were built in the sand bed. This could be considered to simulate possible clay or silt layers in the sand (see Fig. 1). The upper sheet was installed after Test Phase 1 to enable the densification mentioned under item (2) above.
4. *Large ratio of cyclic shear stress τ_{cy} and vertical effective stress σ'_{v0}*: Experimental investigations with $D_r \approx 0.6$ have shown that under undrained conditions liquefaction was achieved after 30 loading cycles (Ishihara 1993) with $\tau_{cy}/\sigma'_{v0} = 0.15$. For the planned model tests under partially drained conditions a maximum ratio underneath the caisson near the sand surface was approximately determined to 0.22.

3. Experimental Setup and Test Procedure

Essentially based on the requirements discussed in the previous section and on practical feasibility considerations, the main characteristics of the experimental setup and procedure were derived.

The cross-section of the breakwater model, including the position of the transducers used at the caisson and its foundation are shown in Figure 1. The sand beneath the caisson is selected as fine as practicably feasible with D_{50} = 0.21mm, D_{10} = 0.13mm and U =1.69. The initial relative density, resulting from the wave action in Test Phase 1, is estimated to an average value of $D_r = 0.21$. In spite of the flushing process, the achieved saturation was still below $S_r = 1$.

The seaward berm consists of a 35 cm thick armor layer, a 20 cm filter layer and a 45 cm core. The caisson is placed on a 20 cm thick rubble layer. More details are given by Kudella and Oumeraci (2004).

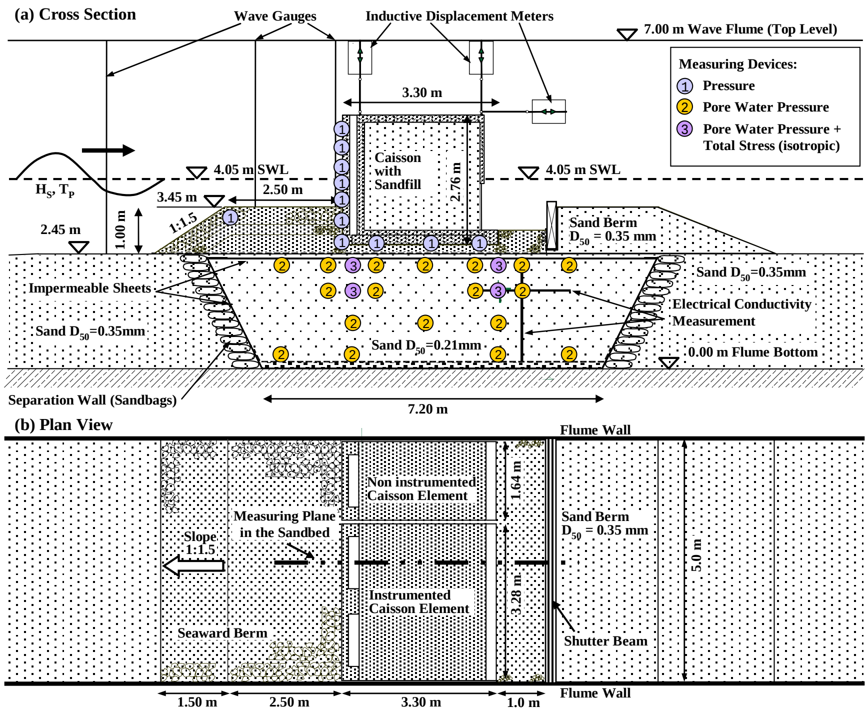

Figure 1. Model setup with the location of the measuring devices.

In the sand bed beneath the caisson, 26 pressure transducers for the measurement of pore water pressure and total stresses and soil density rods for the determination of changes in seabed porosity are installed on a fixed frame (Fig. 1, see Kudella and Oumeraci 2004).

For the measurement of the wave loading and the dynamic response of the caisson, a total of 14 measurement devices are installed at the caisson, including: (i) ten water pressure transducers for the determination of the wave load on the caisson, (ii) three displacement meters for the dynamic response and (iii) a wave gauge for wave run up and run down at the caisson front (Fig. 1). A further wave gauge over the berm and a pressure transducer at the outer edge of the berm provide the input pressure and wave just before reaching the measuring area. Further 18 wave gauges are installed along the flume.

The test program includes two phases: a preliminary Test Phase 1 without the breakwater and a main Test Phase 2 as illustrated by the model setup in Figure 1. For both phases the water depth over the horizontal seabed

(h_s = 1.60 m) is kept constant. For Phase 2 the water depth directly at the caisson front was also kept constant (h_1 = 0.60 m). The test program for both Phases 1 and 2 is given in Table 1.

Table 1. Test program with regular waves (R) and TMA wave spectra (S).

Generated Wave Height H or H_s [m]	Test-Phase	Wave Period T or T_p [s] 4.5	5.5	6.5	7.0	8.0
0.4	1		R			
	2	R / S	R / S	R / S		
0.5	-					
	2	R / S	R / S	R / S		R / S
0.6	1	R	R			
	2	R / S	R / S	S		S
0.7	-					
	2	S	R / S	R / S		S
0.8	1	R				
	2	S	R / S	R / S		
0.9	-					
	2		S	R	R	
	Water depth h_s=1.6m, h_1=0.6m					

The objective of Test Phase 1, which includes only the instrumented seabed, is to compact the loose sand bed and to get a first insight into the response of the seabed to direct wave attack. Therefore, only regular waves were used.

For Test Phase 2, which includes both the instrumented caisson breakwater and the instrumented seabed (Fig. 1), regular and irregular waves are used. Considering the model setup in Figure 1 and based on the PROVERBS parameter map (Oumeraci et al. 2001) for the definition of the type of wave loads, the program in Table 1 is devised to obtain both pulsating wave loads and breaking wave impact loads. Some tests are repeated to investigate the effect of relative density changes on the generation of pore water pressure in the seabed.

4. Experimental Results, Analysis and Discussion

The analysis of the results is focused here on the tests of Phase 2 (Fig. 1). For different types of wave loads, the effect of caisson motions on transient and residual pore pressure generation as well as the relationship between residual pore pressure and residual soil deformation are addressed below.

4.1. *Wave Load Classification*

Following the PROVERBS parameter map (Oumeraci et al. 2001) a basic distinction is made between (i) pulsating wave loads for which the expected response of the breakwater and its foundation is small enough for quasi-static approaches to be applied and (ii) breaking wave impact loads for which larger caisson motions are expected; i.e. the load time history is most relevant for the response of the breakwater, so that dynamic stability analysis must be applied.

For the tested water depths $h_s = 1.60$ m and $h_1 = 0.60$ m, pulsating wave loads are obtained for wave heights $H = 0.4 - 0.7$ m and wave periods $T = 4.5$ s - 8.0 s while impact loads are obtained for steeper waves with $H = 0.6 - 0.9$ m and $T = 4.5 - 7$ s (see Table 1). These wave loads are transmitted *directly* to the seabed (*wave mode*) and *indirectly* through the induced motions of the caisson (*caisson motion mode*). Since the latter mode is dominant in the seabed beneath the breakwater, particular focus will be put in the following on the effect of caisson motions on the seabed response.

4.2. *Effect of Caisson Motions on Transient and Residual Pore Pressure Generation*

To examine the relative influence of caisson motions and direct wave load on pore pressure generation, three vertical planes located at the rubble berm (I), the seaward edge (II) and the shoreward edge (III) of the caisson are considered. At these three locations pore pressure in the seabed (as a response to wave and caisson motion), pressure on the berm and caisson motions at both seaward and shoreward edge are recorded. A comparative analysis of the pore pressure in the seabed at these three locations for both pulsating and impact load is expected to provide the required insight into the relative importance of *direct* and *indirect* generation of transient and residual pore pressure in the seabed.

4.2.1. *Transient Pore Pressure Generation*

A detailed examination of the transient pore pressure generation in the upper sand layer at Locations I - II is shown in Figure 2 for *pulsating wave load*. At Location I the effect of the caisson motions on the pore pressure response is relatively small, but still recognizable at the slightly reduced peaks of the pore pressure events (solid line in Fig. 2c compared to solid line in Fig. 2a).

However, beneath the seaward edge (Location II), the comparison of the uplift pressure $p(t)$ recorded at the bottom slab (dashed curve in Fig. 2a) and the pore pressure response recorded at P25 (dashed curve in Fig. 2c) clearly shows that the pore pressure generation is largely dominated by the caisson motions. As the caisson motion starts (Events 3 and 4) the pore pressure begins to decrease and even turns negative, reaching a minimum value just at the peak of the caisson motion. Between Events 3 and 4, where no noticeable motions are observed (Fig. 2b), the pore pressure curve (dashed line in Fig. 2c) follows almost exactly the shape of the uplift pressure curve (dashed line in Fig. 2a), implying that here the *wave mode* dominates.

For *pulsating wave load* it can be concluded that (i) even small vertical caisson motions will affect the transient pore pressure generation in the seabed beneath the caisson structure significantly and that (ii) the contribution of the caisson motions to this generation strongly will decrease with increasing distance from the caisson front.

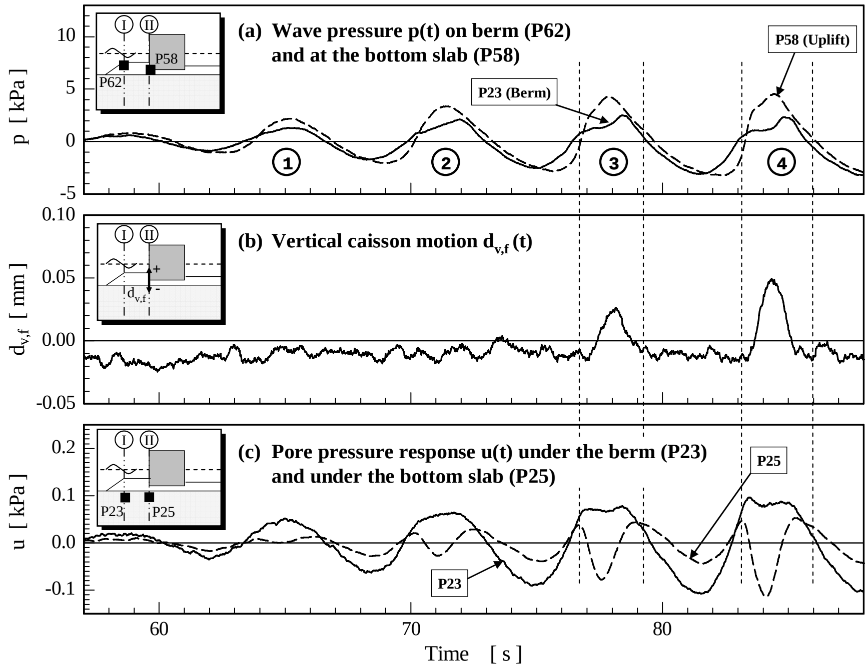

Figure 2. Relative contributions of “wave mode” and “caisson motion mode” to transient pore pressure generation in the seabed for pulsating wave load (H = 0.4 m, T = 6.5 s, h_s = 1.6 m).

For *breaking* wave *impact load* a similar procedure to Figure 2 is shown in Figure 3 in order to check whether and up to which extent the conclusions which have been drawn above for pulsating wave load also hold for breaking wave impact load. In the same way as in Figure 2, the pore pressure responses $u(t)$ in the upper layer of seabed beneath the seaward edge (P26) and at Location I (P23) are plotted in Figure 3, together with the applied wave pressure $p(t)$ on the berm (P62) and under the bottom slab (P59), including the vertical caisson motions $d_{v,f}(t)$ at Location II.

At Location I, the pore pressure trace $u(t)$ in P23 follows fairly closely the trace of the pore pressure $p(t)$ on the berm (see Figs. 3a,c), suggesting that the pore pressure $u(t)$ in the seabed and in front of the caisson (P23) is essentially generated by the direct action of the waves transmitted through the rubble foundation (*wave mode*). At Location II, however, the pore pressure trace $u(t)$ in P26 follows closely the trace of the caisson motion $d_{v,f}(t)$, but in opposite direction (Figs. 3b,c), suggesting that the pore pressure $u(t)$ in the seabed beneath the caisson edge is essentially generated by the caisson motions $d_{v,f}(t)$ (*caisson motion mode*). The direct contribution of the applied wave pressure $p(t)$ as observed in P62 is negligibly small and even smaller than in the case of pulsating wave load. Even the positive ($p_{\max}$) and negative ($p_{\min}$) peaks of the observed pore pressure trace in the rubble at Location II (P59) cause only a small disturbance in the pore pressure trace $u(t)$ in P26 (Figs. 3a,c).

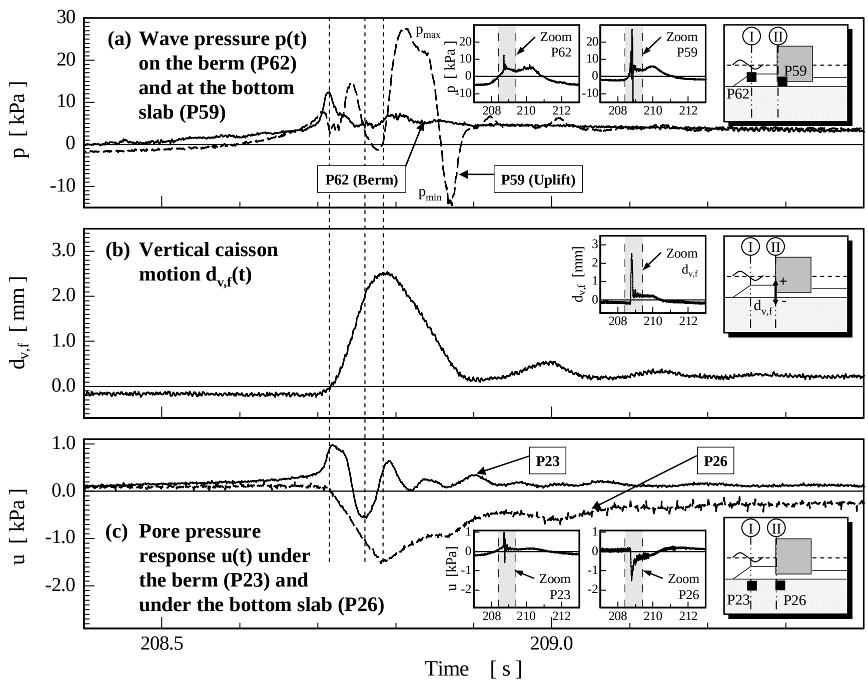

Figure 3. Relative contributions of “wave mode” and “caisson motion mode” to transient pore pressure generation in the seabed for breaking wave impact load (H = 0.9 m, T = 6.5 s, h_s = 1.6 m).

4.2.2 *Residual Pore Pressure Generation*

As already mentioned above, impermeable PEHD-sheets were placed around the sand bed beneath the breakwater in order to decrease pore pressure dissipation. Under such conditions the results described above have clearly shown that the transient pore pressures in the seabed beneath the back of the caisson are essentially generated by the caisson motions $d_{v,b}(t)$. Consequently, the contribution of the *wave mode* to residual pore pressure generation is likely to be negligible for the sand used, and the caisson motion $d_v(t)$ can be considered exclusively as the input parameter for the generation of residual pore pressure in the seabed beneath the caisson. On the other hand, it was found that a threshold value of the frequency and amplitude of the caisson motions $d_v(t)$ is required for the initiation of residual pore pressure generation, which was not reached in the case of pulsating wave loads, but largely exceeded in the case of impact loads. In fact, the latter induce caisson motions with an amplitude and a frequency which are an order of magnitude higher than those induced by pulsating wave loads (Figs. 2 and 3). Therefore, only the tests with breaking wave impact loads will be considered for the generation of residual pore pressure.

In order to illustrate the correlation between vertical caisson motions d_v and subsequent residual pore pressure response u_r in the seabed, the u_r-values (P36) which are achieved after a reference time corresponding to the first 53 loading cycles (all data points $u_r = u_r\ (d_v,\ D_r)$ within the increasing phase of residual pore pressure development) are plotted in Figure 4 against the downward amplitude of the vertical caisson motions $d_{v,b}$ at the seaward edge for different relative soil densities D_r = 0.31 - 0.45.

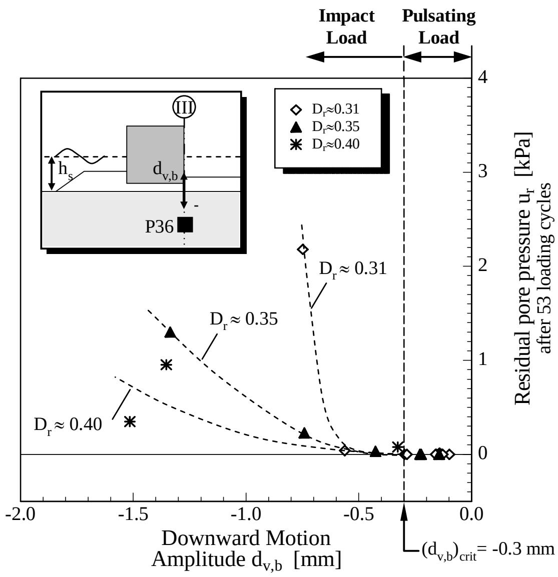

Figure 4. Effect of caisson motions on residual pore pressure for different relative soil densities D_r and both pulsating and impact loads.

The critical downward amplitude for which the generation of residual pore pressure starts, is tentatively estimated to $(d_{v,b})_{\mathrm{crit}}$ = -0.3 mm and corresponds to the boundary line between pulsating and impact wave load (Fig. 4). Once this threshold value is exceeded, the increase rate of residual pore pressure for a given sand bed is strongly determined by the relative soil density D_r. The lower the density D_r, the higher is the increase rate of residual pore pressure.

4.3. ***Residual soil deformations***

In order to illustrate and briefly discuss the correlation between residual pore pressure u_r and residual soil deformations $d_{v,b}$, the wave load $M_t(t)$ (M_t = total moment around the caisson heel), the associated vertical oscillatory caisson motions $d_{v,b}(t)$ (transient component), the transient pore pressure response $u_t(t)$ as well as the associated residual components $u_r(t)$ and $d_{v,b}(t)$ at the shoreward edge of the caisson are plotted in Figure 5 for 692 wave load cycles corresponding to a test duration of about 1.25 hour.

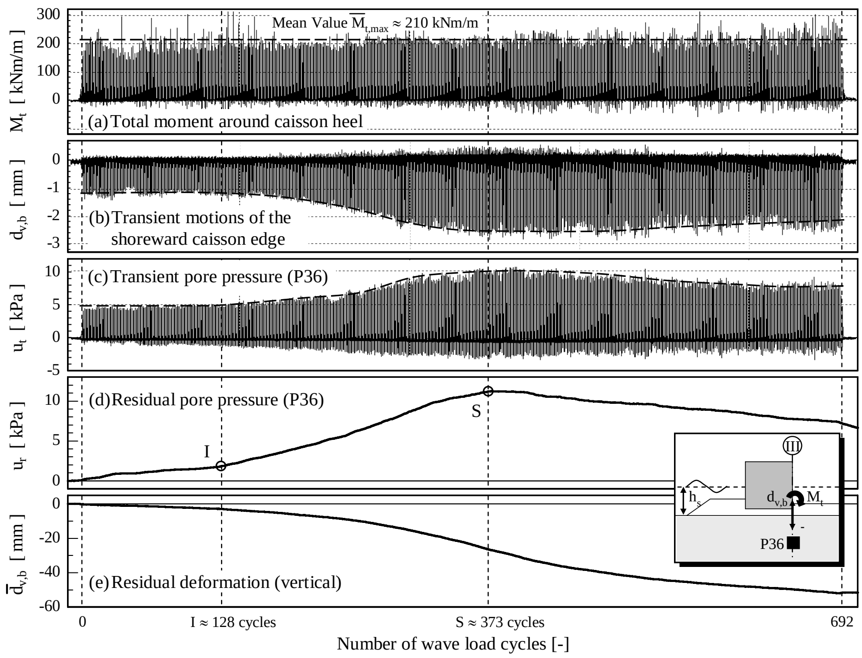

Figure 5. Wave load, pore pressure response and soil deformation (H = 0.9 m, T = 6.5 s, h_s = 1.6 m)

Although the moment peaks $M_{t,\max}$ over the entire test duration do not vary significantly around the mean value of 210 kNm/m, the transient components of the caisson motions $d_{v,b}(t)$ and pore pressure $u_t(t)$ start to increase after 128 load cycles resulting in the “Inflexion Point” I of the response curves of the residual components $d_{v,b}(t)$ and $u_t(t)$; i.e. after Point I the generation of residual pore pressure becomes more dominant and both $d_{v,b}$ and u_t increase at a higher rate up to a “Saturation Point” S where the generation and dissipation of residual pore pressure are in balance. After Point S where the residual pore pressure ratio u_r/σ'_{v0} was determined to about 0.25 (no liquefaction), the residual pore pressure decreases while the residual soil deformation (settlement) still increases. A quantitative analysis of the relative contribution of the generation and dissipation process has been conducted in Kudella and Oumeraci (2004), showing that the generation gradient of pore pressure starts to decrease after Point S due to the increasing compaction of the subsoil, while the dissipation gradient remains constant, thus leading to a decrease of the $u_r(t)$-curve after “Saturation Point” S.

5. Concluding Remarks and Future Work

Even under unfavorable drainage and soil conditions of the seabed beneath a caisson breakwater (thin clay or silt layers in a relatively loose sand bed) as well as under very severe wave load conditions of the structure (breaking wave impacts), only one fourth of the critical residual pore pressure ratio $u_r/\sigma'_{v0} = 1.0$ for total residual liquefaction could be achieved. Nevertheless, the analysis of the first results has brought some light into the processes, which may lead to partial and total liquefaction of a sand bed beneath a caisson breakwater under unfavorable conditions. Among others, it was found that

1. both transient and residual pore pressure generations are essentially due to caisson motions and that the latter should have a frequency and an amplitude which are large enough to generate residual pore pressure
2. such large and high frequent caisson motions can only be induced by severe breaking wave impacts
3. a very close correlation exists between residual pore pressure and residual soil deformations beneath the breakwater which can definitely be quantified by a more detailed analysis of the balance between the generation and the dissipation process of pore pressure.

Since critical residual soil deformations, which may lead to the collapse of the breakwater can also occur for low values of the residual pore pressure ratio u_r/σ'_{v0}, further analysis of the results will focus on the closer examination of the balance between pore pressure generation and dissipation in order to come up with some design guidance based on allowable soil deformations.

6. Acknowledgement

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