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Clemente Cantelmo, William Allsop and Scott Dunn

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WAVE PRESSURES IN AND UNDER RUBBLE MOUND BREAKWATERS

Clemente Cantelmo[1], William Allsop[2] and Scott Dunn[3]

1 Engineer, HR Wallingford, Howbery Park, Wallingford, OX10 8BA, United Kingdom; c.cantelmo@hrwallingord.co.uk; Tel: +44 (0)1491 822247
2 Technical Director, HR Wallingford, nwha@hrwallingord.co.uk; Visiting Professor University of Southampton;
3 Principal Engineer, HR Wallingford, sdu@hrwallingord.co.uk; +44 (0)1491 822347

Key Words
Rubble mound breakwater, Internal pressures, Modelling, Pore pressure attenuation, Wave damping

Abstract
Laboratory tests have been conducted to investigate flow motion and pore pressure attenuation within rubble mound breakwater. A typical multi-layered rubble mound was tested in a wave flume under regular and random wave conditions. Measurements of wave pressures have been used to validate semi-theoretical and numerical approaches describing wave damping in porous media. Data are to be used to define a reliable model able to predict the behaviour of porous structures under wave attack. The ultimate aim of the research initiative is to define an innovative design procedure for coastal and maritime structure coupling hydro-dynamic with geotechnical models, and thus to assess the overall structure stability and the foundation soil liquefaction risk.

1 Introduction

Structural stability and integrity of marine structures such as caissons, seabed structures, pipelines or rubble mound breakwaters can be strongly affected by the foundation soil behaviour. Wave induced residual or instantaneous liquefaction can cause loss of soil strength in saturated granular materials with catastrophic consequences such as tilting of caissons or shear failure of breakwater slopes. Numerical modelling of wave-induced liquefaction has increased in sophistication since 1970's, but still requires substantial improvement. In this study, the finite element program, SWANDYNE II (HR version), has been employed to investigate the liquefaction potential of soil around marine structures, but has not yet been applied to rubble mound.

The SWANDYNE model has been previously validated (Chan, A. 1988, P.L. Vun 2005, Dunn et al 2006) against caissons and pipelines.

A new design procedure, aiming to couple SWANDYNE with an hydrodynamic model is now being developed for design of coastal and maritime structure. Such an approach has been successfully applied to caisson breakwater design where a CFD numerical model was employed to predict wave pressures on the caisson. A time series of forces acting on the structure was extracted from the CFD simulation and used as input to run SWANDYNE in order to predict the behaviour of the structure and its foundation under cyclic loadings. It has however been found very difficult to apply a similar approach to rubble mounds breakwaters due to the lack of information on pore pressures within the porous medium under wave attack. The first step in these studies was therefore to gather information on porous structures behaviour under wave attack.

Experimental studies were undertaken at HR Wallingford to validate numerical and theoretical approaches to predict pore pressure variation within a porous medium under cyclic wave-induced loadings. The paper describes physical model tests on a typical rubble mound breakwater, and presents tests results used to validate numerical and semi-theoretical theories describing wave induced flow motion within porous media.

2 Methodology: Description of laboratory tests

Model tests (at a nominal scale of 1:25) have been performed in a 45 metre long flume, distance from paddle face to endwall 41.0 metre. A bathymetry slope of 1:26 starting at x=16.17 m from the paddle runs up to x=34.84 m, see Figure 1:

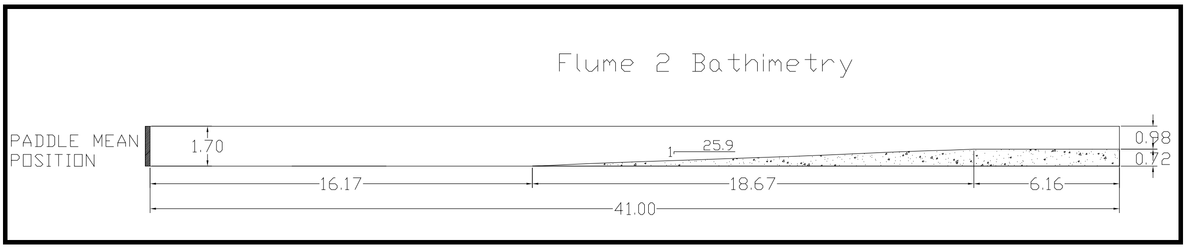

Figure 1 Model test wave flume bathymetry

Before running tests, an absorbing beach was placed to absorb incoming wave energy during wave calibration tests, avoiding reflection from the wall at the end of the flume.

Then, before construction of the rubble mound itself, 10 pressure transducers were placed in a trench formed in the bottom of the flume, and a wood frame supporting 10 other pressure transducers was positioned at the middle cross section of the structure.

Figure 2 Layout of pressure transducer frame within model rubble mound

Subsequently quarry run material, filter and armour layers were placed to form the rubble mound breakwater (Figure 3). Once construction of the tests structure was complete, supplementary measuring devices were installed.

Figure 3 Final model of rubble mound breakwater

The complete flume instrumentation consisted of:

- 10 wave probes
- 1 run up probe
- 2 overtopping probes
- 20 pressure transducers

To assist flow observations, a small plastic dye tank was placed above the rubble mound crest. The tank gradually released black dye through a narrow pipe into the pore water within the mound, allowing clearer video-taping of the movement of the phreatic surface during testing. Free surface oscillations were measured using twin wire wave gauges along the flume as shown in Figure 4.

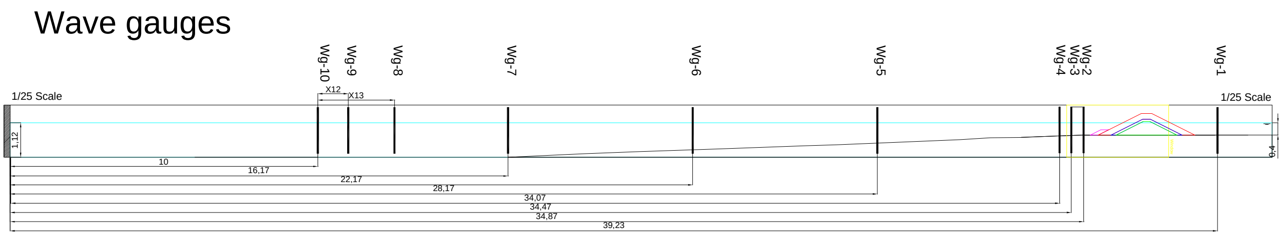

Figure 4 Wave gauge and test structure locations

The arrangement of pressure gauges is shown in Figure 5. These transducers were placed in order to analyse the pressure excursion along vertical and horizontal arrays.

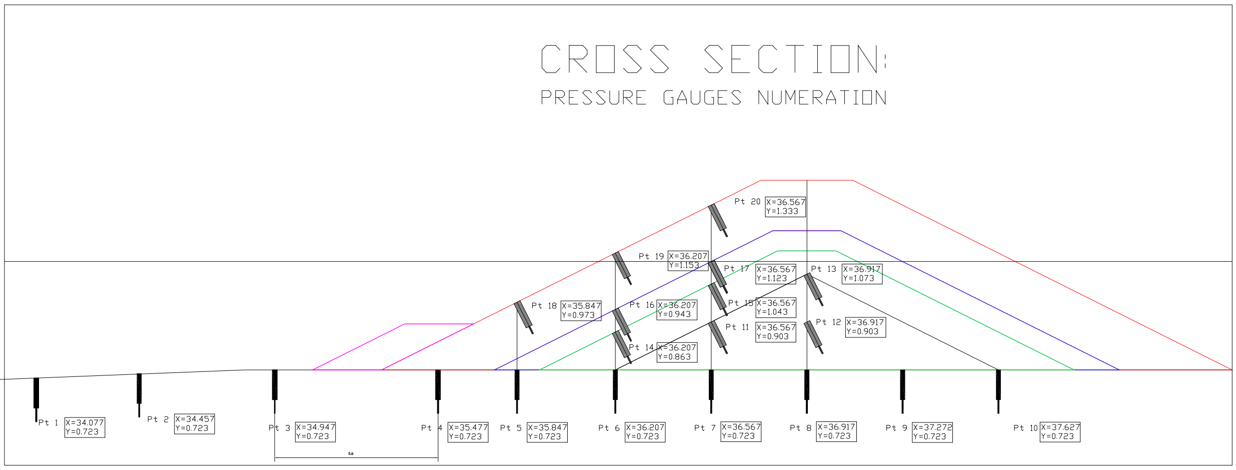

Figure 5 Pressure gauge arrangement

Wave run-up levels were measured with a wire gauge parallel to the front slope. Overtopping was measured by means of 2 probes; a first probe was placed on the crest of the breakwater and gave out an electric signal each time an overtopping event occurred, discharges were then collected in a tank placed at the back of the rubble mound, in turn equipped to measure water levels inside the tank, from which overtopping volumes could be calculated.

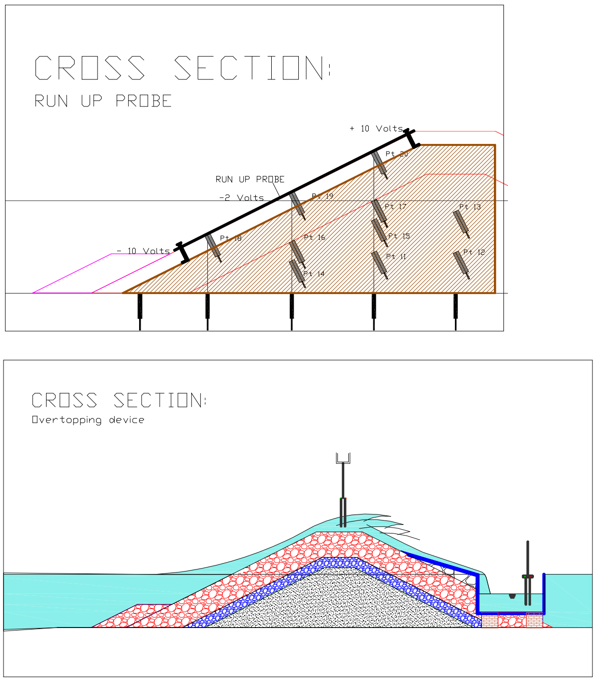

Figure 6 Wave run-up and overtopping measurement devices

Tests were run with both random and regular waves. Several water levels, wave heights, wave periods and wave steepness were tested to indentify the effects of each parameter on flows / pressures within the porous medium as the wave approaches the structure. The main wave conditions simulated in the flume are summarised in Figure 7; H_s = 0.10-0.21m for random waves, T_m = 1.2-3.2 seconds. Regular waves were run with H=0.04-0.08 m and T=1.25-3.12s. Water levels of 0.20, 0.30 and 0.40 m at the toe of the structure were tested.

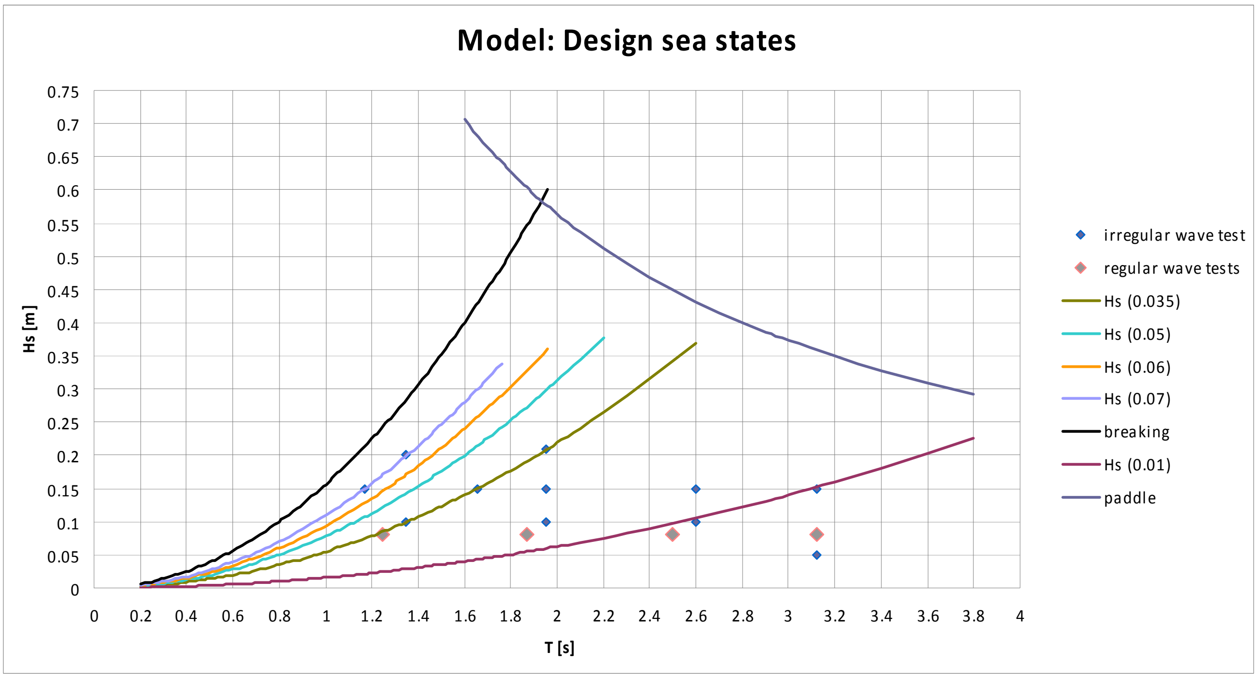

Figure 7 Matrix of wave conditions used

3 Test results

The main aim of the study was to investigate pore pressure fluctuations within the rubble mound. Example of pore pressure time series recorded at the toe and at the bottom of the structure (shown in Figure 8) from pressure transducers 1 to 5 at the toe of the mound (see Figure 5). It can be seen in Figure 8 that the wave amplitude increases as the wave approaches the structure, then the wave breaks on the frontal slope and as we move further inside the porous medium (transducers 6 to 10) wave energy is dissipated due to the friction resistance.

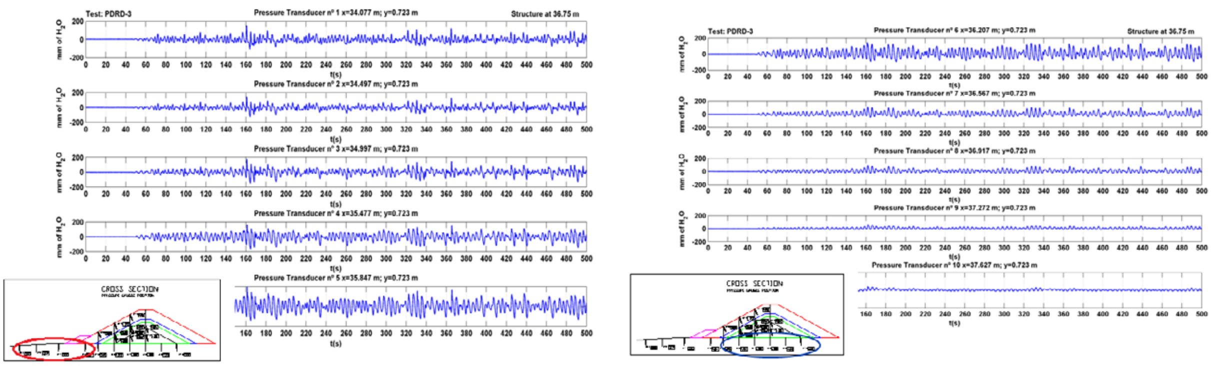

Figure 8 Pore pressure time series along the bottom of the flume

As mentioned, the arrangement of the pressure gauges shows most of the wave attenuation within the porous mound. Similar time series in Figures 9-11 show wave damping over vertical arrays of pressure transducers, Figure 9 recorded along the second array (pressure transducers 6, 14, 16 and 19); Figure 10 shows the third array and Figure 11 along the fourth. Moving from the top transducer to the bottom there is a decay of the wave amplitude due to the non linearity of the wave. Such effect is not however of great importance for small amplitude waves, but is more visible for larger amplitude waves. No perceptible time lag appears along the vertical, signals show that peak pressures are generally registered at the same time.

Moving on to Arrays 3 and 4 further into the structure, wave propagation into the mound shows delays relative to the "run-up" signal (which shows the water level running up and down the frontal slope), and the loss of energy due to the friction resistance, which is clear from reduction of the amplitude of oscillation.

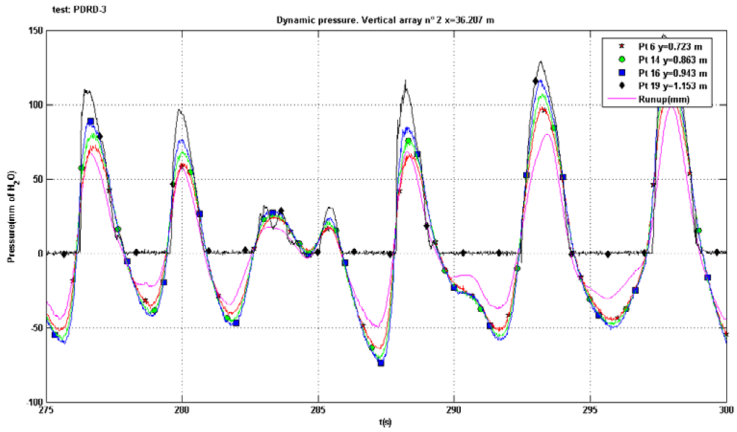

Figure 9 Pore pressure time series along vertical array 2

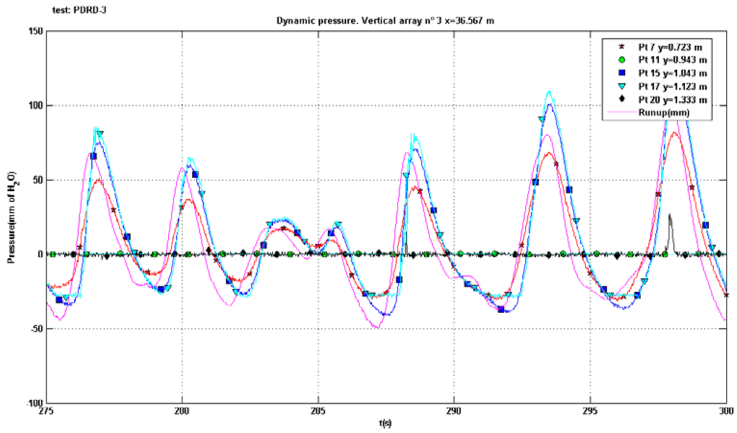

Figure 10 Pore pressure time series along vertical array 3

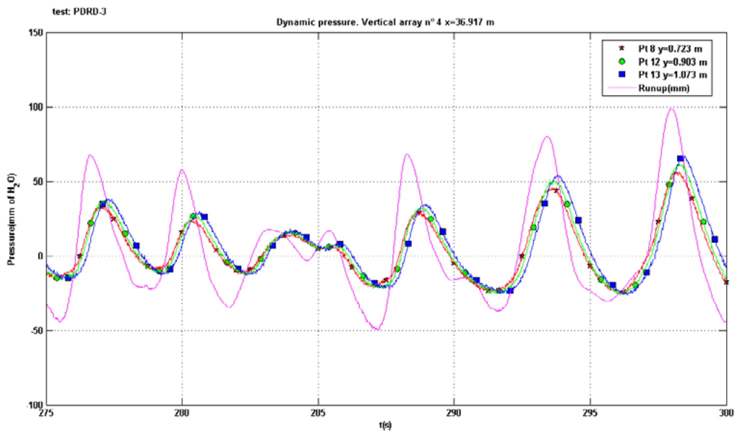

Figure 11 Pore pressure time series along vertical array 4

Wave-induced pressure evolution along the bottom of the flume is shown in Figure 12 and 13 for long and short period waves. The RMS pressure trend is representative of the main wave-structure interaction: long waves surge on the frontal slope and then dissipate energy as they propagate into the structure, the peak pressure is reached on the front slope. Short waves reach their peak early and then collapse or plunge on the structure dissipating energy. These effects are less clear for the maximum wave pressure, as these depend on extreme events and are less representative of the general behaviour of waves.

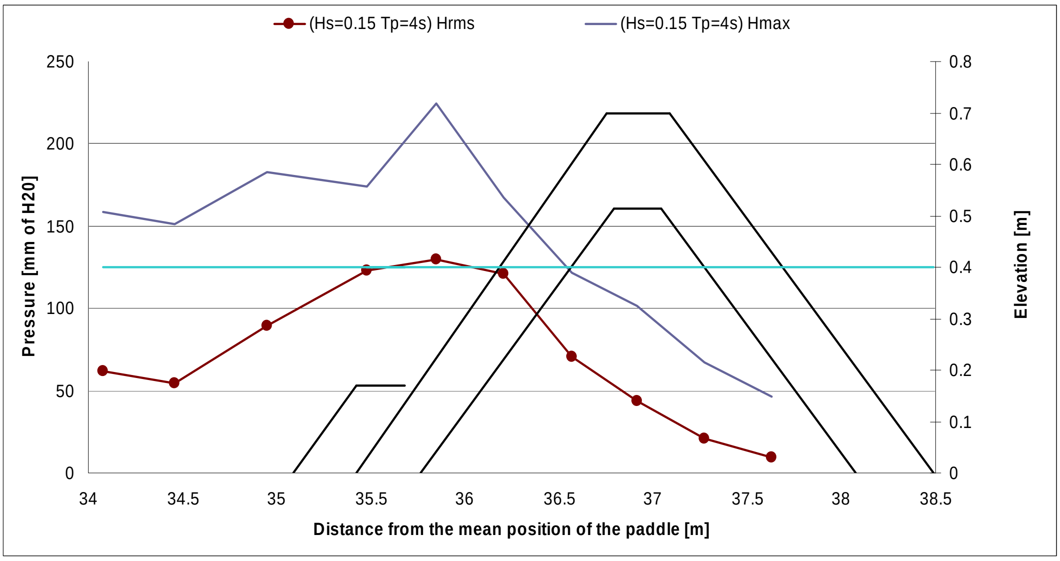

Figure 12 Max and RMS dynamic pressure evolution along the bottom of the flume. Long wave test

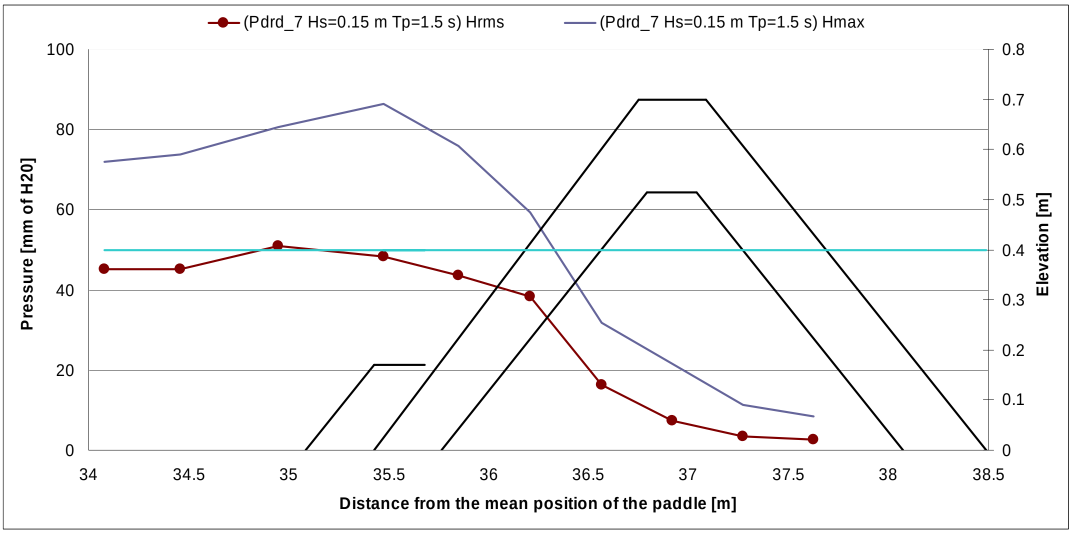

Figure 13 Max and RMS dynamic pressure evolution along the bottom of the flume. Short wave test

The vertical variation of max and RMS wave pressure are shown in Figure 14. Decay of maximum wave pressure excursions with reducing elevation are more significant, but are less evident for RMS pressures.

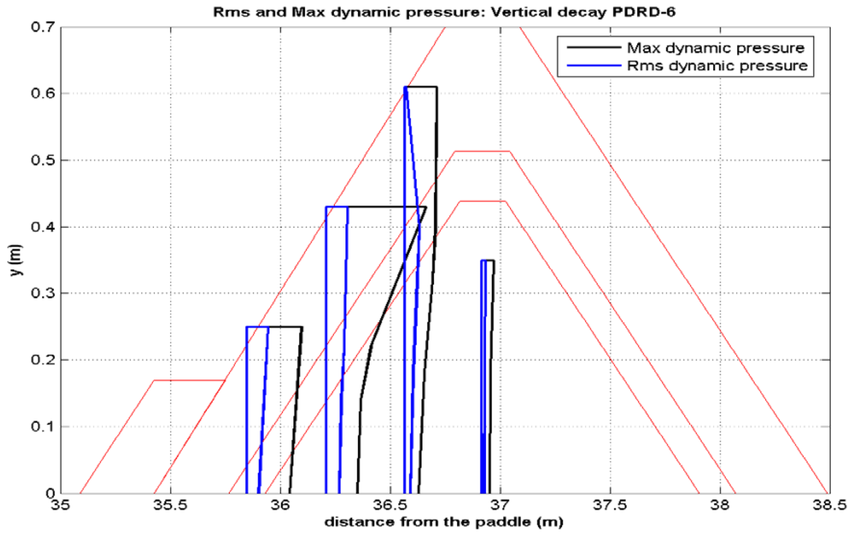

Figure 14 Max and RMS dynamic pressure vertical profile

4 Theoretical approach

Wave propagation into rubble mound breakwaters has been investigated by Hall (1991, 1994) in small scale experiments and by Buerger et al. (1988), Oumeraci & Partenscky (1990) and Muttray et al. (1992, 1995) in large scale experiments. Field measurements have been conducted at the breakwater at Zeebrugge (Troch et al., 1996, 1998), prototype data, experimental data and numerical results have been analysed by Troch et al. (2002). The water surface elevations inside the breakwater and the amplitude of the pore pressure oscillations decrease exponentially in the direction of wave propagation (Hall, 1991; Muttray et al., 1995).

The water surface elevations, the pore pressure oscillations and the wave setup increase with increasing wave height, wave period and structure slope (Oumeraci & Partenscky, 1990; Hall, 1991). They decrease with increasing permeability of the core material and with increasing thickness of the filter layer (Hall, 1991). The damping rate of pore pressure oscillations increases with wave steepness (Buerger et al., 1988; Troch et al., 1996) and decreases with increasing distance from the still water line (Oumeraci & Partenscky,1990; Troch et al., 1996).

Oumeraci & Partenscky (1990) proposed the following model for pore pressure oscillation within core material:

$$P(x) = P_0(\beta \frac{2\pi}{L'} x) \tag{1}$$

with dimensionless damping coefficient β, amplitude of pore pressure oscillations P(x) (at varying position x>0, as being x the x-co-ordinate across the core), initial pressure P0 (at position x =0, assuming x=0 at the interface filter-core) and wavelength L' inside the structure. The previous model was fitted through data tests obtaining one global damping coefficient β=2. Subsequently, based on prototype results from Zeebrugge and large scale model tests, Burcharth et al (1999) developed an expanded expression for the damping coefficient:

$$\beta = a \frac{\sqrt{n} L_p^2}{H_s b} \tag{2}$$

Where *n* is the material porosity, *b* is the horizontal width of the breakwater at the core depth and *a* is a fitting coefficient (L_p and H_s are wavelength and wave height).

Experimental data described in section 2 have been fitted to formula [1] with attenuation coefficients β= 1.2-2, as shown in Figure 15. Best fit is obtained with β=2, as Oumeraci & Partenscky found in their tests.

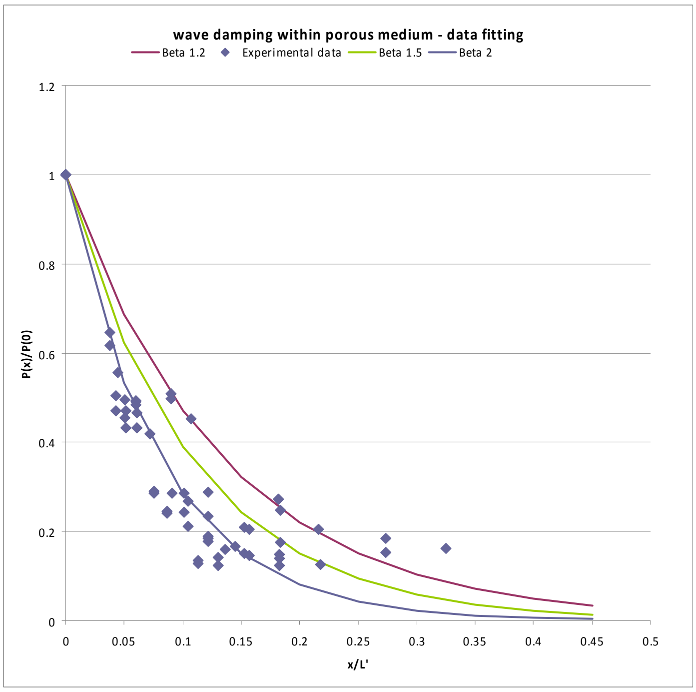

Figure 15 Wave damping calculated with Oumeraci & Partenscky formula and different values of β

The expression for the attenuation coefficient derived from the formula proposed by Burchart [2] was then used to calculate wave damping. A value of a=0.0078 was derived as best fit of formula [2] to the experimental data. A comparison between experimental and theoretical data of wave decay is shown in Figure 16.

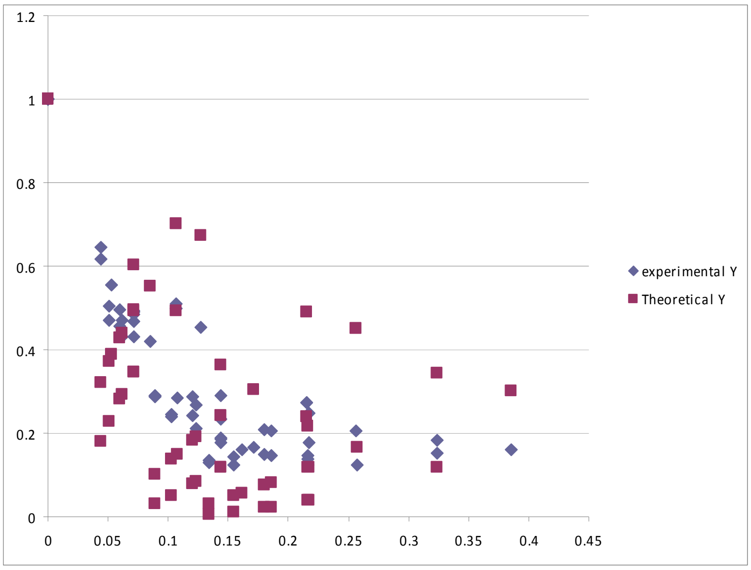

Figure 16 Wave damping calculated with Oumeraci & Partenscky formula, damping coefficient proposed by Burchart

5 Numerical model Validation

Physical model test data have been compared with a numerical scheme able to predict wave dynamic within porous media. The model OTTP-1D, is based on the hydrodynamic model of the swash zone OTT-1D, building on the earlier numerical model by adding a permeable layer. Such a model was previously validated against field data from a gravel beach at Slapton, U.K in 2001 (Clarke et al 2004) and was tested here with data from a rubble mound breakwater.

Wave action in the model uses the Non Linear Shallow Water equations in the free-water flow region, and equations for flow in porous media are based on the Forchheimer equations (the pressure gradient is balanced by friction, inertia and advective terms).

Using NLSW equations, the model is strictly valid only in the swash zone. Two parameters are used to determine whether the model can be used to obtain reliable predictions: the shallow water non-dimensional parameter (h/L) and the Ursell number (Ur) which measures the degree of non-linearity of the wave. The following conditions must be fullfilled:

$h/L \ll 1$ and $Ur \gg 1$ [3]

To test the efficiency of the numerical model to predict wave propagation within porous medium, some of the flume tests were simulated with OTTP-1D. Table 1 shows the laboratory tests where condition [3] is respected:

Table 1 Tests condition run in OTTP-1D

							SHALLOW WATER			URSELL NUMBER		
test	H_s[m]	T_p [s]	L0 [m]	depth [m]	L [m]	k [1/m]	h/l					
PDRD_3	0.15	4	24.96	0.22	5.80	1.08	0.038	<	0.1	12.04	>>	1
PDRD_4	0.15	3.33	17.29	0.22	4.81	1.30	0.045	<	0.1	8.25	>>	1
PDRD_10	0.1	4	24.96	0.22	5.80	1.08	0.038	<	0.1	8.02	>>	1
PDRD_11	0.1	3.33	17.29	0.22	4.83	1.29	0.045	<	0.1	5.56	>>	1

The above tests have been run in OTTP. A simplified reproduction of the real flume geometry was used in the numerical model **shown in Figure 1**7.

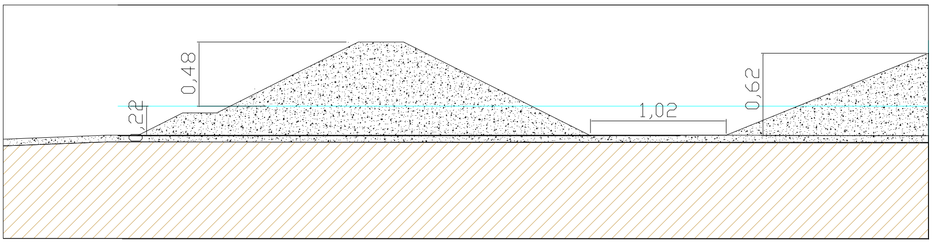

Figure 17 Geometry of numerical flume

OTTP allows the user to choose a single granular material. As the greatest dissipation happens in the core material, the whole structure was assumed to be made of a single granular material with the same properties as the core of the rubble mound in the flume.

Numerical gauges were placed along the flume at the same locations as in the flume tests. As the model is based on NLSW equations, it assumes a depth averaged value of velocity (neglecting vertical acceleration) and therefore is not able to represent the vertical wave damping due to the non linearity of the wave. In shallow water, the assumption of hydrostatic pressure distribution should be reasonably correct. Numerical simulations were conducted using as input the time series recorded during the flume tests, then in order to assess the model validity the time series recorded at each probe during both experimental and numerical tests have been compared.

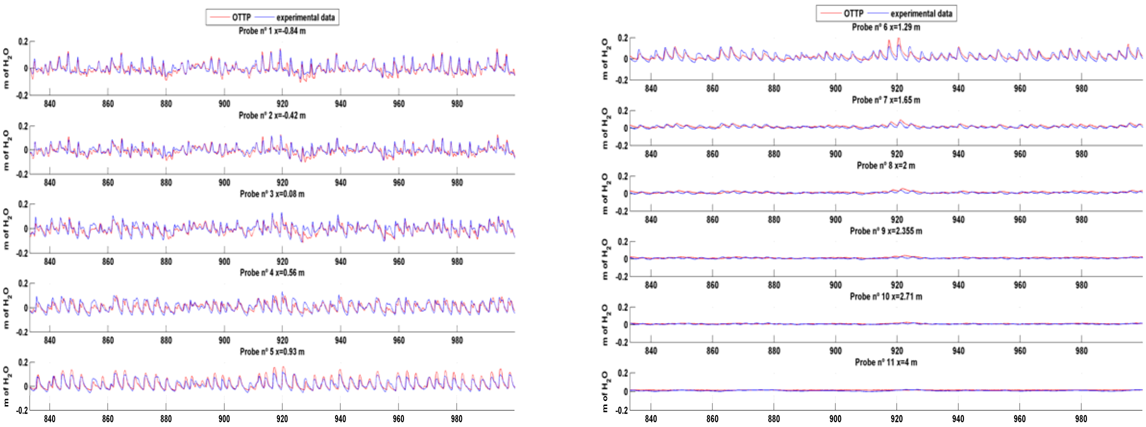

Figure 18 Time series of wave pressure at the bottom of the flume comparing experimental and numerical data

An example of RMS and max dynamic pressure profile at the front and along the bottom of the structure obtained with both experimental and numerical results is plotted in **Figure 19**.

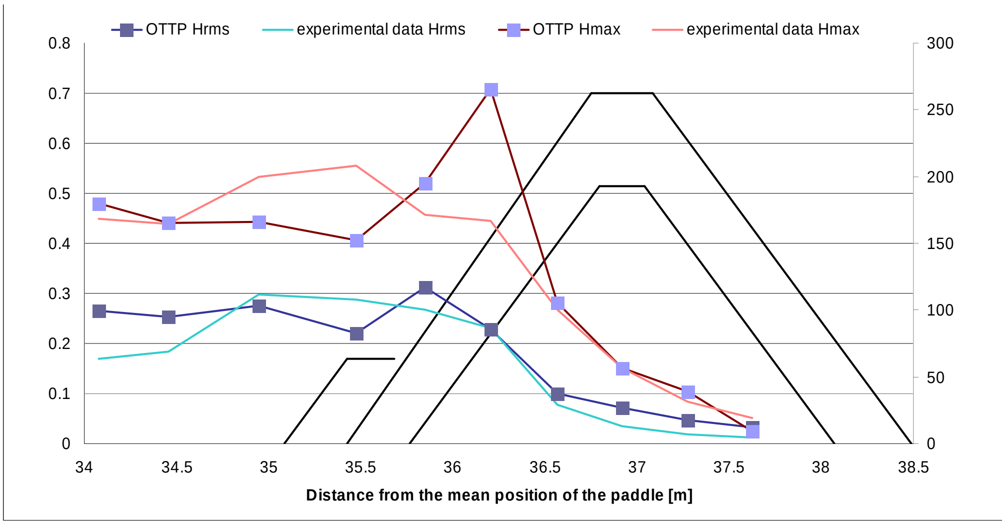

Figure 19 RMS and Max dynamic pressure along the structure, experimental and numerical data

Some differences occur over the structure toe, and on the front slope in the run-up zone, probably because vertical acceleration, non-linearity and impacts are more significant here, and the numerical model is not able to account for them., Within the porous medium, there is however reasonable agreement between numerical and experimental data.

6 Conclusions

Tests were performed to investigate dynamic wave pressure evolution within rubble mound breakwaters. Data were used to validate semi-theoretical approaches describing wave damping in porous media. Results were compared with results from a one-dimensional numerical model, OTTP-1D. The ability of the numerical model to propagate waves within porous medium was tested, comparing numerical against experimental data. Both numerical and semi-theoretical models describing wave motion in porous medium give encouraging results. The research is still in progress and different CFD models are to be tested with the experimental data collected.

The next step in this research initiative to investigate the overall stability of maritime structures and the soil liquefaction potential, will be to couple hydro-dynamic with geotechnical models (particularly SWANDYNE). Such analysis has been successfully applied to a caisson breakwater and is to be extended to rubble mounds.

Acknowledgments

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References

Clarke S, Dodd, N. and Damgaard, J. 2004: Modelling flow In and Above a porous beach. Journal of waterway, Port, Coastal and Ocean Engineering, Vol 130, No. 5 September 1, 2004.

Lara J.L., Losada I.J. & Guanche R. (2008) Wave interaction with low-mound breakwaters using a RANS model, Ocean Engineering, Volume 35, Issue 13, pp 1388-1400

Muttray M. & Oumeraci H.(2005) Theoretical and experimental study on wave damping inside a rubble mound breakwater. Coastal Engineering 52, 709-725

Oumeraci H. & Partenscky H.W., 1990. Wave induced pore pressure in rubble mound breakwaters. Proc 22nd International Conference of Coastal Engineering, pp 1334-1347, Delft, The Netherlands, ASCE

S. Dunn, P L Vun, A H Chan.(2006) Numerical modelling of wave-induced liquefaction around pipelines. Journal of waterway, Port, Coastal and Ocean Engineering, Vol 132 No 4, 276-288

Troch P. (2001) Experimental study and numerical modelling of pore pressure attenuation inside a rubble mound breakwater. PIANC bulletin 108, 5-27.

Vun P-L, 2005. "Experimental and numerical studies on wave induced liquefaction to soil around marine structures founded in the seabed". *PhD Thesis*, University of Birmingham, UK.

Zienkiewicz,O.C., Chan, A.H.C., Pastor, M., Screfler, B.A., and Shiomi T. (1990). "*Computational geomechanics with special reference to earthquake engineering*", Wiley, Chivcester, U.K.

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HR Wallingford
Working with water

HR Wallingford Ltd
Howbery Park
Wallingford
Oxfordshire OX10 8BA
UK

tel +44 (0)1491 835381
fax +44 (0)1491 832233
email info@hrwallingford.co.uk

www.hrwallingford.co.uk