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WAVE LOADING, OVERTOPPING AND TRANSMISSION OF LOW CREST CAISSON BREAKWATERS

Giovanni Cuomo[1], Andrea Polidoro[1,2], Elettra Pasquini[3], John Alderson[1], David Robinson[1], William Allsop[1] and Scott Dunn[1]

[1] *HR Wallingford – Howbery Park, Wallingford, Oxfordshire OX10 8BA, UK*
[2] *Dipartimento di Ingegneria Idraulica, Geotecnica ed Ambientale (DIGA) Facoltà di Ingegneria - Università degli Studi di Napoli Federico II*
[3] *Dipartimento di Ingegneria Civile, Ambientale e dei Materiali (DICAM) Facolta di Ingegneria - Alma Mater Studiorum Università di Bologna*

Key Words
Coastal Structures, Caisson Breakwaters, Low Crested Structures Wave-structure interaction, Physical model testing, Computational Fluid Dynamics, Wave loading, Wave transmission, Wave overtopping

Abstract
Recent LNG projects have pointed out the efficiency of low crested caissons breakwaters in providing shelter from waves to LNG terminals during frequent conditions whilst allowing for wave overtopping / transmission during most severe storms, reducing the design loads and therefore the overall cost of the breakwater. Unfortunately, little is known on hydrodynamic behaviour and loading of low crested structures heavily overtopped by long waves and use of existing design codes generally leads to over-design and excessive construction costs. Bearing this in mind, both physical and numerical models have been successfully applied to investigate wave-structure interaction problems and in particular to improve accuracy and increase confidence in the estimations of wave loading, wave overtopping and transmission over low crested structures. Tentative semi-empirical formulas are presented for wave overtopping and wave transmission based on new experimental data.

1 INTRODUCTION

Many recent LNG projects have required dedicated terminals, often sheltered by a dedicated breakwater. As shelter for loading / offloading operations is only required over relatively non-extreme wave conditions, the breakwater can be tuned to limit wave transmission during frequent conditions whilst allowing for wave overtopping / transmission during most severe storms. This reduces design loads on the caisson, and therefore the overall cost of the breakwater. Given the scarcity of details provided in design codes, a series of numerical and physical model studies have been completed with the aim of developing new guidance on:

- Influence of crest width on wave transmission
- Reflection coefficients for submerged crest structures
- Vertical uplift / downward pressures during significant overtopping events
- Effect of reducing crest level and caisson width on:
    - Loading / stability

- Wave transmission

1.1 Previous work

Wave transmission over caisson breakwater was firstly investigated by Goda (1969) who derived a prediction method based on measurements from physical model tests performed using regular waves. Goda's original work was later extended by Takahashi (1996) who suggests that the above formulation is still applicable to random waves when using H_s as input. For the irregular wave case, calculated from:

$$C_{tr} = \begin{cases} \left\{ \left\{ 0.25\left[1-\sin(\pi/2\alpha)\left(\frac{R_c}{H_s}+\beta\right)\right]^2 + 0.01\left(1-\frac{d}{h}\right)^2 \right\}^{0.5} & \beta-\alpha < \frac{R_c}{H_s} < \alpha-\beta \\ 0.1\left(1-\frac{d}{h}\right) & \frac{R_c}{H_s} > \alpha-\beta \end{cases} \quad (1)$$

where R_c is the freeboard, H_s is the incident significant wave height, and d and h are the water depth respectively at the toe of the structure and in front of the wall. α= 2.2 and β is a function of d/h.

The ranges of parameters investigated were relatively wide:

- Relative depth and mound height, d/h = 0, 0.3, 0.5 & 0.7.
- Relative crest elevation, R_c/H_s = -2.5 to +2.0

Successive studies aimed to assess the relative effect of reshaping the leading edge profile of the caisson including the effect of a parapet wall (reviewed by Takahashi, 1996) results are generally provided in terms of transmission coefficients plots as a function of the dimensionless freeboard: R_c/H_S.

A review of previous HR Wallingford studies, literature and recent breakwaters was performed in order to asses what geometries should be tested in the flume. Information of caisson height (H), width (W) and length (L) was collected and the following ranges of dimensionless ratios were identified:

- Width-to-Height ratio (W/H) - Min = 0.23, Max = 1.61 & Mean = 0.93
- Width-to-Length ratio (W/L) - Min = 0.56, Max = 0.94 & Mean = 0.68
- Height-to-Length ratio (H/L) - Min = 0.37, Max = 0.75 & Mean = 0.59

The review of geometries also highlighted that most caissons fall in the range of L = 20 to 70m, and are generally installed in depths between h = 10m to 60m.

1.2 PHYSICAL MODEL TESTS

1.2.1 Model setup

A set of 2D physical model tests were carried out in one of the wave flumes of HR Wallingford (45 m long, 1.2 m wide and 1.6 m deep). Both regular and irregular waves were produced by an absorbing wave generator at the offshore end of the flume. Transmitted waves were absorbed by a dissipating beach. Waves were measured using twin wire wave gauges at the generation point and in front and behind the model structure. An overall view of the flume setup is given in Figure 1 showing location of model structure and wave probes.

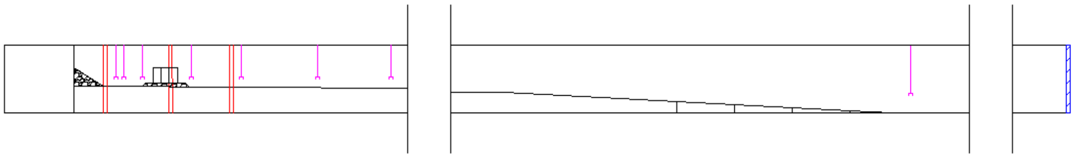

Figure 1 Section of physical model setup showing bathymetry and location of wave probes and model structure.

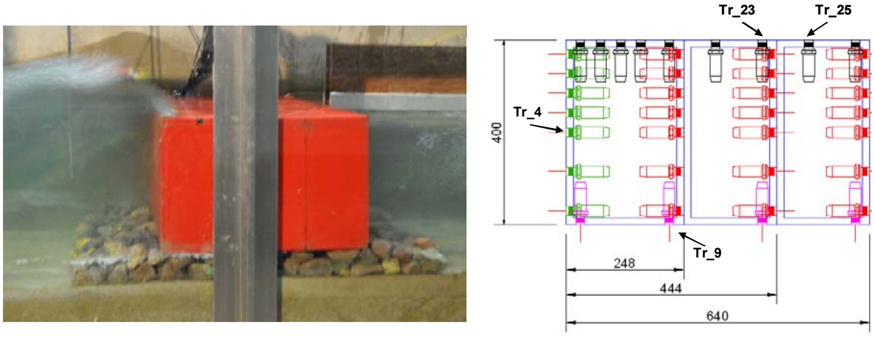

Figure 2 Picture (left) and cross section (right) of the modular model structure used in the model tests, showing location of pressure sensors (dimensions: mm).

1.2.2 Model structure

The model structures (Figure 2) were formed in timber and designed to provide the required stiffness of the setup to avoid corruption of recorded signal due to dynamic response of the model structure whilst allowing for optimal installation of sensors. Based on results of the literature review, 3 caisson geometries where selected with W/H = 0.62, 1.11 and 1.6. An additional screen structure was tested with W/H = 0.125. Wave loads were measured using pressure transducers, wave overtopping and wave transmission were also monitored using respectively collecting tanks and wave probes.

1.2.3 Wave conditions

Wave conditions used in the tests are summarised in Table 1. The range of variation of the key parameters investigated is as follows: 0.025m< H_s <0.3m , 1s< T_p <4.35s and -0.063m< R_c <0.05m.

1.2.4 Initial results

Wave load time histories recorded during testing using both regular and random waves were analysed to gain results of quasi-static and impulsive loading. The following parameters are present in this paper:

- $Fh_{qs+,1/250}$, $Fh_{qs-,1/250}$: quasi-static horizontal positive (shoreward) and negative (seaward) force at significance level 1/250
- $Fv_{qs+,,1/250}$: quasi-static vertical positive (upward) force at significance level 1/250

Horizontal and vertical loads on the caissons are compared in Figure 3 to predictions using Goda (2010, $Fh_{qs+,1/250}$ and $Fv_{qs+,1/250}$) and Sainflou (1929, $Fh_{qs-,1/250}$). For completeness, estimate of seaward loads using empirical correction suggested by McConnell et al (1999) is also plotted (dash-dotted line in the central panel of Figure 3).

Table 1 Water level and incident wave conditions used during physical model tests, MD being to the bottom level at the toe of the model structure.

Caisson toe level	Caisson crest level	Caisson width	SWL	H_s	T_p
mMD	mMD	M	mMD	m	s
0.075	0.475	0.05-0.640	0.425-0.538	0.025	1.27-1.3
0.075	0.475	0.05-0.640	0.425-0.538	0.066-0.075	1.0
0.075	0.475	0.05-0.640	0.425-0.538	0.075-0.078	2.2
0.075	0.475	0.05-0.640	0.425-0.538	0.108-0.125	1.3
0.075	0.475	0.05-0.640	0.425-0.538	0.122-0.125	2.8
0.075	0.475	0.05-0.640	0.425-0.538	0.174-0.296	1.5-1.6
0.075	0.475	0.05-0.640	0.425-0.538	0.200-0.296	1.7-1.9
0.075	0.475	0.05-0.640	0.425-0.538	0.180-0.296	3.9-4.4

Initial results suggest that most established prediction methods in literature tend to overestimate actual wave loading on low crested caisson. Best-fit (bold solid) lines in Figure 3 give estimates 20%, 30% and 10% lower than predictions (dashed lines) by most established methods for respectively for quasi-static shoreward, seaward and uplift loads.

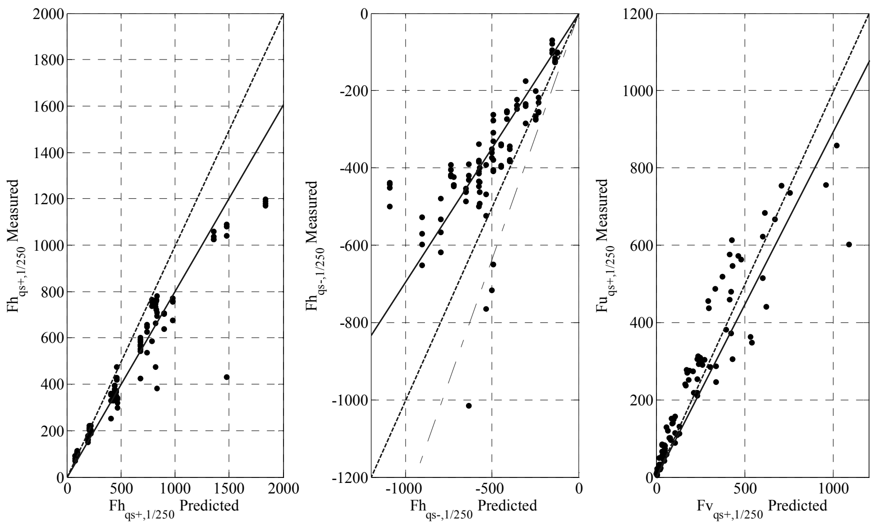

Figure 3 From left to right: comparison of measured and predicted values of quasi-static horizontal (shoreward and seaward) and vertical (uplift) loads.

1.3 Wave overtopping

The Overtopping Manual (EurOtop, Pullen et al. 2007) prescripts prediction methods to assess wave overtopping at sea walls depending on the geometry of the structure and wave conditions at the toe. Wave conditions at vertical walls are said to be non-impulsive (or pulsating) when the

EurOtop coefficient, h* > 0.3; and impulsive when h* < 0.2 (defined in the EurOtop manual Equation 7.1.A similar “impulsiveness” parameter d* is defined in EurOtop Equation 7.2 for vertically composite walls.

Impulsive waves overtopping at composite vertical walls can be estimated using EurOtop Equation 7.12:

$$Q^* = \frac{q}{d_*^2\sqrt{gh_s^{\,3}}} = 4.1\times10^{-4}\left(d_*\frac{R_c}{H_{m0}}\right)^{-2.9} = 4.1\times10^{-4}R^{*-2.9} \tag{2}$$

where q is the mean overtopping discharge and R_c is the freeboard. Equation 2 is valid for $0.05 < d_*\frac{R_c}{H_{m0}} < 1$ and $h_* < 0.3$.

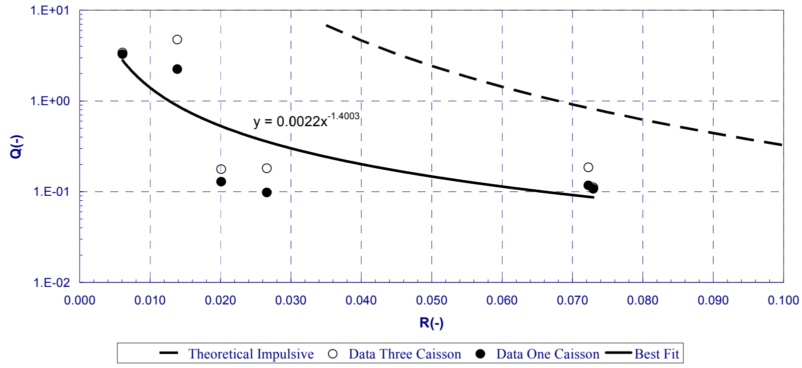

Figure 4 Comparison of measured overtopping and prediction by EurOtop manual for impulsive conditions using best fit coefficients a=0.0022 and b=-1.4.

Wave overtopping measured during these physical model tests using impulsive random waves are compared to predictions by EurOtop Equation 7.12 in Figure 4 showing rather lower discharges than predicted. A best fit curve in Figure 4 can be described by EurOtop Equation 7.12 with modified coefficients:

$$Q^* = aR^{*b} \tag{3}$$

where Q* and R* are respectively the non dimensional discharge and freeboard (note that the two are defined differently for impulsive and non-impulsive conditions in the EurOtop manual. The revised coefficients derived from these tests are a = 0.0022 and b = -1.4.
According to the EurOtop, non-impulsive waves overtopping at composite vertical walls can be estimated using EurOtop Equation 7.3:

$$Q^* = \frac{q}{\sqrt{gH_{m0}^3}} = 0.04\exp\left(-2.6\frac{R_c}{H_{m0}}\right) = 0.04\exp(-2.6R^*) \tag{4}$$

Overtopping discharges from these physical model tests using non-impulsive random waves are compared to prediction by EurOtop Equation 7.3 in Figure 5. Best fit coefficients in Eq. 4 are a = 0.04 and b = -4.24.

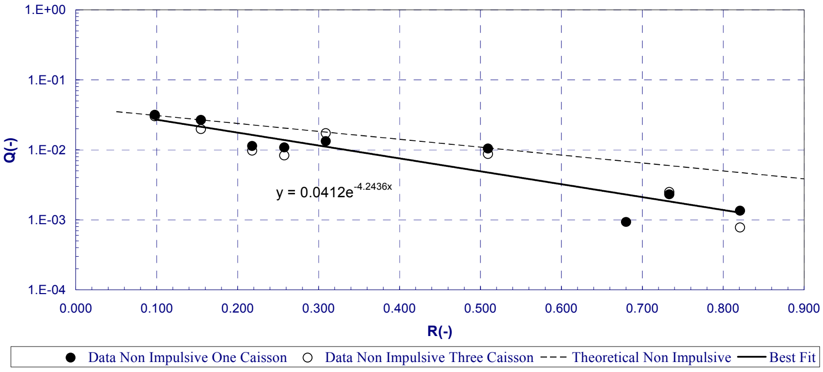

Figure 5 Comparison of measured overtopping and EurOtop prediction for non-impulsive conditions using best fit coefficients a=0.04 and b=-4.24.

For completeness the whole dataset is also presented in Figure 6. It is interesting to note that a single best fit curve for both impulsive and non impulsive wave conditions can be derived to still obey Equation 3, with new coefficients a= 0.0014 and b =-1.52.

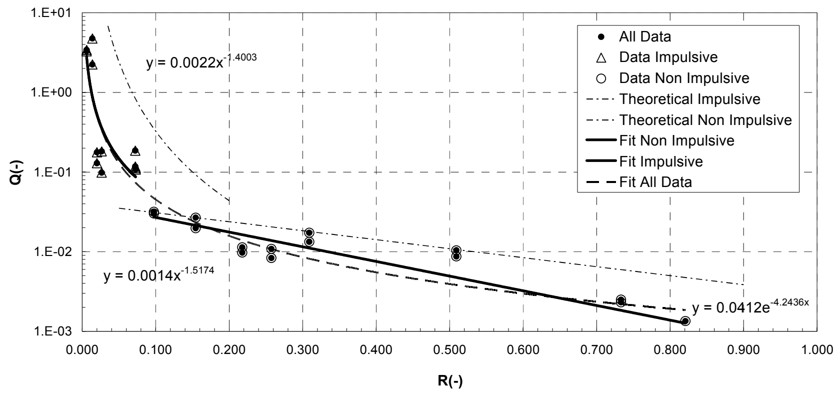

Figure 6 Comparison of measured overtopping and prediction by EurOtop manual for both impulsive and pulsating waves using coefficients a=0.014 and b=-1.52.

1.3.1 Wave transmission

Transmitted waves were recorded at three locations shoreward of the model structure (Figure 1), the effect of caisson width on wave transmission is presented in Figure 7, showing transmission coefficients at location of probe No. 2, as a function of the relative freeboard (R_c /H_s) for different caisson widths. Best fit curves in Figure 7 obey equation:

$$C_{tr} = \left\{ c\left[1 - \sin(\pi/2a)\left(\frac{R_c}{H_{si}} + b\right)\right]^2 + 0.01\left(1 - \frac{d}{h}\right)^2 \right\}^{0.5} \tag{5}$$

where a, b and c are best fit empirical parameters, summarised in Table 2 for different values of the caisson width.

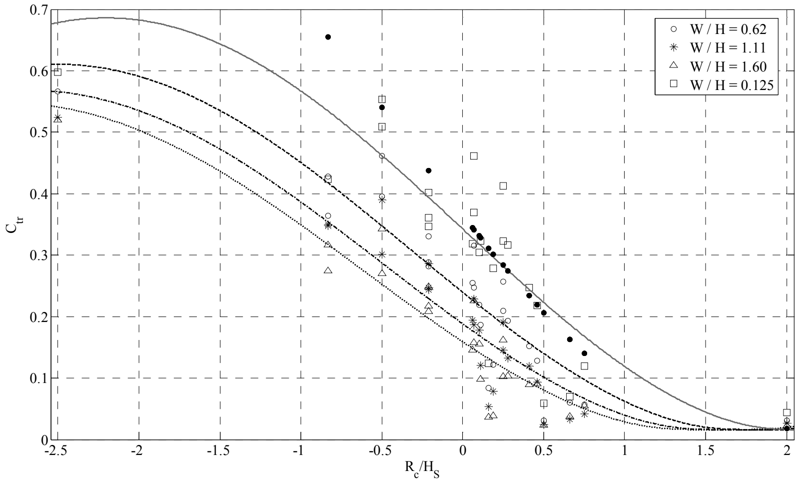

Figure 7 Comparison of wave transmission coefficients measured during random waves tests and prediction by Eq. 5 using best fit coefficients in Table 2. Predictions using Eq. 1 (black dots).

As expected, wave transmission decreases with increasing R_c/H_s. Changing caisson width on wave transmission is also shown in Figure 7, with C_{tr} reducing with increasing caisson width.

Table 2 Best fit parameters in Equation 5

Caisson width (m)	A	b	c	R^2
0.050	2.2	0.0034	0.118	0.73
0.25	2.2	0.31	0.093	0.84
0.44	2.2	0.49	0.081	0.87
0.64	2.2	0.61	0.075	0.87

1.3.2 NUMERICAL MODELLING

Numerical simulations were performed using both commercial (Ansys CFX) and open-source (OpenFOAM) codes solving Navier-Stokes equation for compressible fluids. Ansys CFX is a general purpose computational fluid dynamics software suite, capable of modeling steady-state and transient multi phase (free surface) laminar and turbulent flows. The code solves the partial differential equations using a finite volume approach over an unconstructed three-dimensional mesh. A number of turbulence models are available including RANS, LES and DES. OpenFOAM (Open Field Operation and Manipulation) is an open source CFD able to model both single and multiphase problems including waves' generation, propagation and interaction with structures. The model allows Navier-Stokes equations to be solved over finite element grids through different computational approaches (DNS, RAS or LES) using a wide range of turbulence models. Both models have been modified to allow generation of non-linear waves according to the FTT Theory (Fenton, 1988).

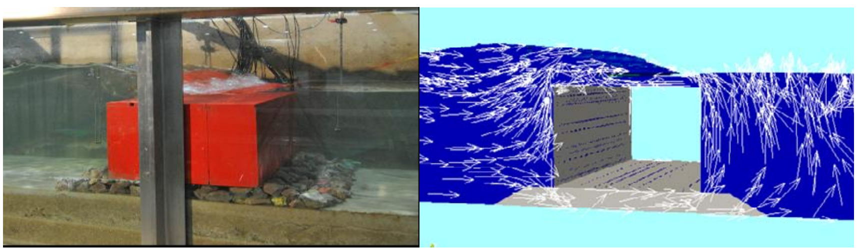

Figure 9 Example interaction of an overtopping wave with a low crested caisson breakwater, comparison between results from physical and numerical model..

1.4 INITIAL RESULTS

Numerical models were set up to replicate the physical model tests, example results from simulation of regular waves interaction with a submerged caisson (H=0.125m T=2.8s d=0.538m R_c= -0.06m) are shown in Figures 8 and 9, comparing pressure time histories recorded during physical and numerical model tests (from top to bottom and left to right respectively for sensors 4, 9, 23 and 25.). Transmission coefficients from the same numerical simulation (C_{tr} = 0.41) also compares well (less than 2.5% difference) with measurements (C_{tr} = 0.42).

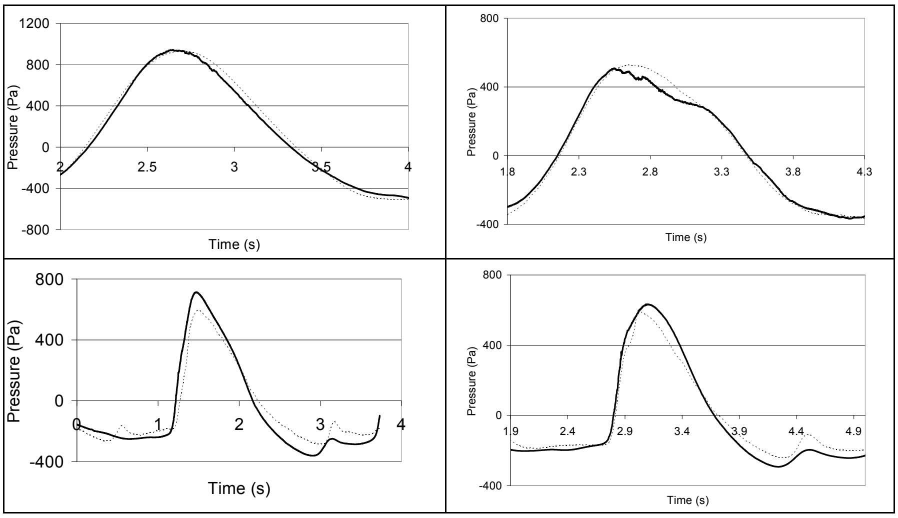

Figure 8. Comparison between pressure time histories recorded during physical (dashed line) and numerical (solid line) model tests.

1.5 CASE STUDIES

HR Wallingford has previously performed physical model tests of the hydraulic performances of a caisson design (Figure 10). At a later stage, a numerical model study was performed to assess the effect of replacing the caisson by a 0.5m thick vertical concrete wall (Figure 11) and to compare performance of the two breakwater designs, with regard to wave-induced loading on and overtopping / wave transmission behind the structure.

The data recorded during this physical model study were first used to validate results achieved using CFD; example results are shown in Figure 12, comparing measurement of wave pressure at selected locations over the caisson under both non-breaking and breaking wave attack.

After calibration, the numerical model has been used to assess, qualitatively, any differences in the hydraulic performances of the two alternative breakwaters.

Example time-histories of surface elevation computed at selected locations along the flume are plotted in Figure 13 respectively in front (top) and behind (bottom) the breakwater. For comparison, data refer to simulations carried out using the same incident wave conditions interacting with the wall (solid thin lines) and the caisson (dots) breakwater.

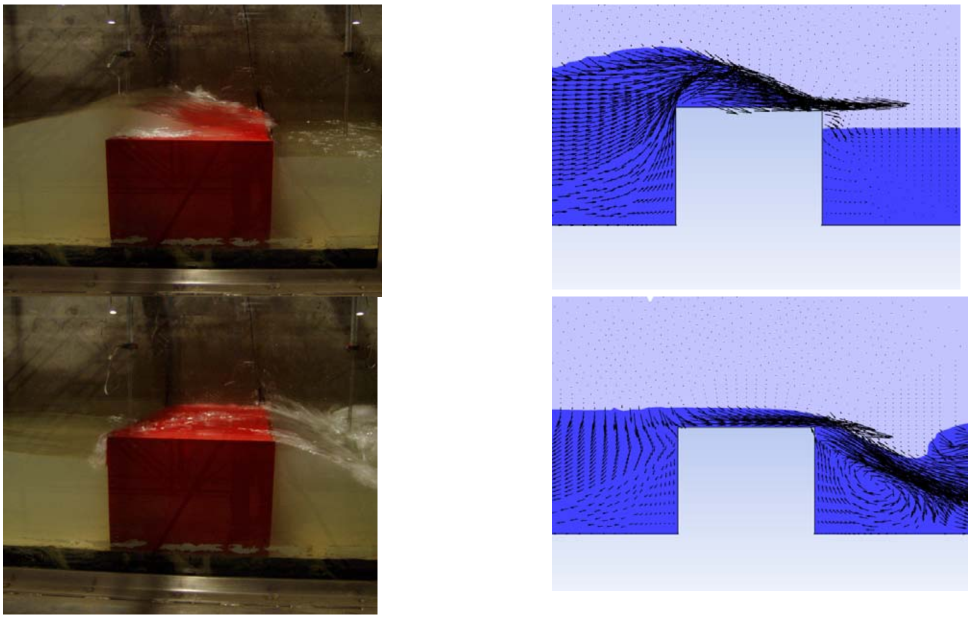

Figure 10. Example interaction of an overtopping wave with a low crested caisson breakwater, comparison between results from physical and numerical models.

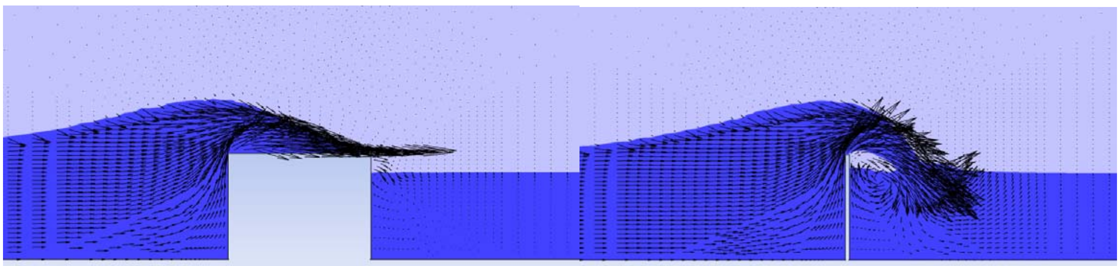

Figure 11. Example results from numerical simulation of wave interaction with a low crested caisson breakwater (top) and wave screen (bottom).

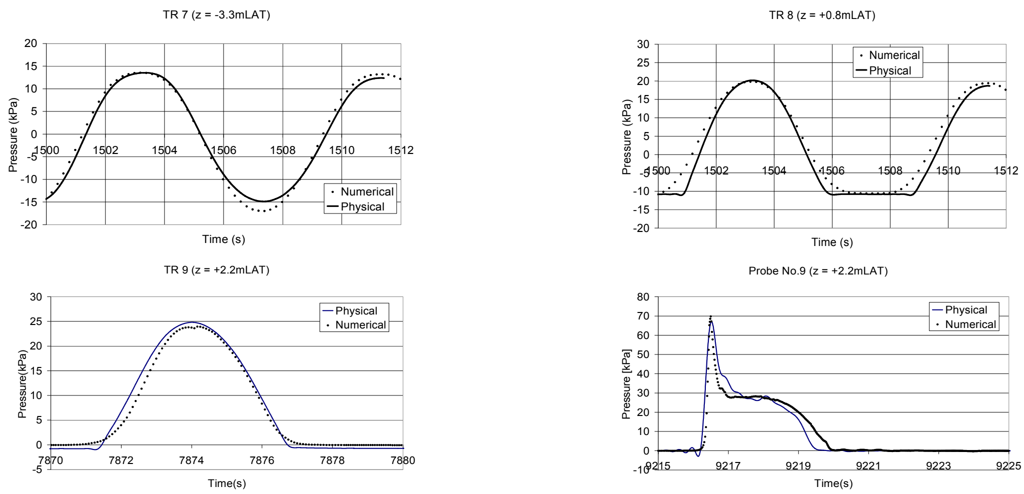

Figure 12. Comparison of wave load time histories recorded during physical model tests (solid line) and numerical results obtained during calibration (dotted line).

Example time-histories of computed horizontal force are plotted in Figure 14. For comparison, data on the top and bottom panels refer to simulations carried out respectively with the wall breakwater and the caisson. Graph show total (bold line), shoreward (thin solid line) and seaward (dashed thin line) horizontal forces.

Due to reduction in the time-lag between shoreward and seaward horizontal forces acting respectively on seaward and shoreward faces of the breakwater, the total horizontal forces acting on the wall appear to be slightly less than those on the caisson. Over the range of wave condition tested in the numerical simulations, this variation appears to be up to a maximum of 10%.

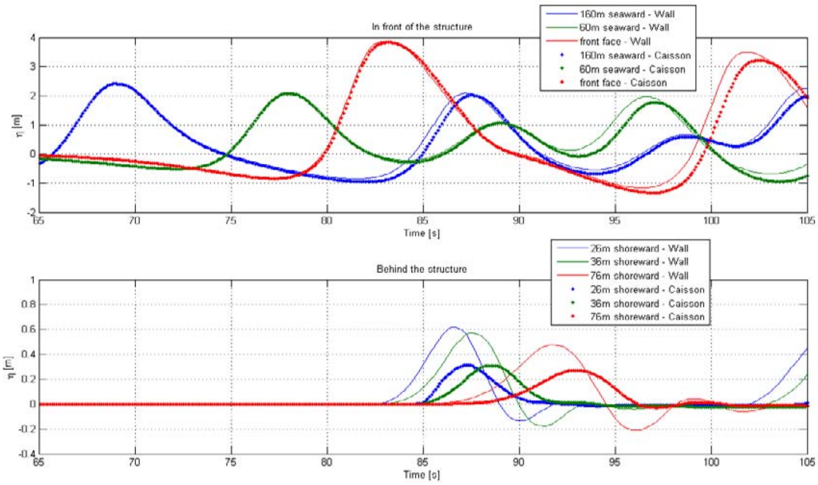

Figure 13. Example time-histories of surface elevation computed at selected locations along the flume in front (top) and behind (bottom) the breakwater.

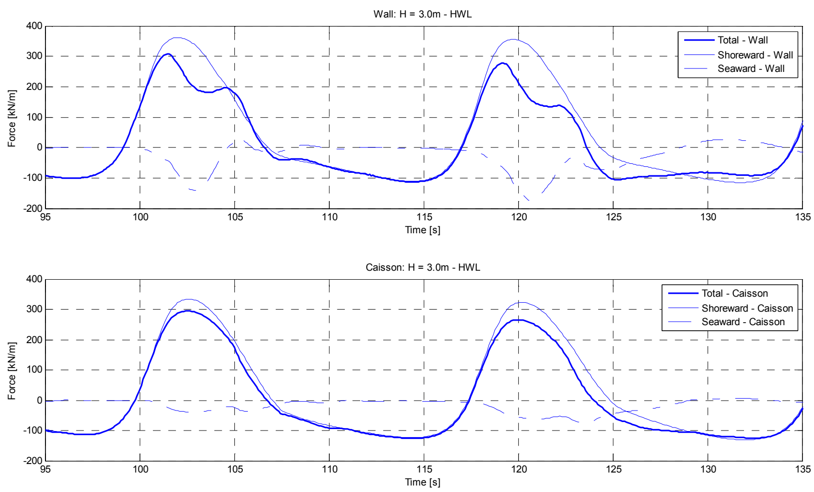

Figure 14. Example time-histories of wave induced seaward (solid line) back (dashed line) and total horizontal forces (solid bold line) acting on the wall (top) and the caisson breakwater (bottom).

1.6 INITIAL CONCLUSIONS AND FURTHER WORK

Both physical and numerical (CFD) models have been successfully applied to investigate performance under wave action, and in particular to improve accuracy and confidence in estimations of wave loading, wave overtopping and transmission over low crested structures.

Physical model tests covered a wide range of incident wave conditions and geometrical characteristics of the breakwater. The performances of different type of breakwaters were analysed in terms of offered protection from wave overtopping, reduction of wave transmission and of wave-induced loads on the structure.

Data from physical model tests were compared to predictions from existing methods. Initial results show that current prediction methods may over-estimate both wave loads and transmission. Wave transmission was found to reduce with increasing width of the crest of the caisson and tentative semi-empirical formulas have been derived for both wave transmission and overtopping.

These experiments also provide high quality benchmarks for evaluation and calibration of numerical (CFD) models. Computational results were compared to measurements from physical model tests to identify capabilities and limitations of the two approaches.

Analysis of the recorded physical model data and numerical model studies are currently ongoing, with expectation to provide additional information and prediction methods for wave loading on all 4 faces of the caissons.

2 ACKNOWLEDGMENTS

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3 REFERENCES

Fenton, J. D. (1988) “The numerical solution of steady water wave problems“, Computers and Geosciences 14, 357–368.

Goda, Y (1969) "Re-analysis of Laboratory Data on Wave Transmission over Breakwaters," Rept. Port and Harbour Res. Inst., Vol. 8, No. 3, pp. 3-18.

Goda, Y. (2010) “Random Seas and Design of Maritime Structures“, 3rd Edition, World Scientific.

McConnell K.J., Allsop, N.W.H. and Flohr H. (1999) “Seaward wave loading on verstical coastal structures” Coastal Structures '99, Santander, Spain.

Pullen, T., Allsop,. N.W.H., Bruce T., Kortenhaus A., Schuttrumpf H. and wan def Meer J.W. (2007). “EurOtop – Wave Overtopping of Sea Defences and Related Structures: Assessment Manual“ www.overtopping-manual.com

Sainflou, M. (1928) "Essai sur les Digues Maritimes Verticales" Annales des Ponts et Chaussees. Vol. 98, Pt. 1, Tome 11.

Takahashi, S. (1996) “Design of Vertical Breakwaters“, Port and Airport Research Institute, Japan (Revised in July, 2002 Version 2.1) Revised Version of Reference Document No.34, PHRI.