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Flow in and on the Zeebrugge breakwater

- a comparison between prototype measurements and physical model tests

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1 Introduction

This paper is based on data and results obtained during the MAST II project entitled "Full Scale Dynamic Load Monitoring of Rubble Mound Breakwaters". The main part of this project has been exploitation of a fully instrumented rubble mound breakwater located in the outer harbour of Zeebrugge, Belgium and conduction of attaining physical model tests at three different scales. This approach of looking into the hydraulic response of a rubble mound breakwater makes the project unique world-wide. Thorough description of the instrumentation of the prototype breakwater can be found in (Damme et al. 1992) and (Troch et al. 1996). This paper emphasizes on a comparison between prototype measurements and measurements obtained by physical model testing as well as an inter-comparison of the model tests. The aim is to present achieved results, sections 2 and 3, and subsequently draw conclusions concerning the hydraulic response of the Zeebrugge breakwater, model precision and scale effects.

1.1 The Zeebrugge breakwater

The instrumented part of the Zeebrugge breakwater is constituted by a conventional rubble mound breakwater as shown on figure 1. Due to transverse currents outside the breakwater a protection of willow mattresses have been laid out in order to avoid scour. Some years back the current conditions resulted in considerable amounts of fine sands being washed into the core which combined with the widely graded core material makes the breakwater relatively impermeable. This is further emphasized by the back-filling of the breakwater. As armour layer antifer blocks of 25 t is used

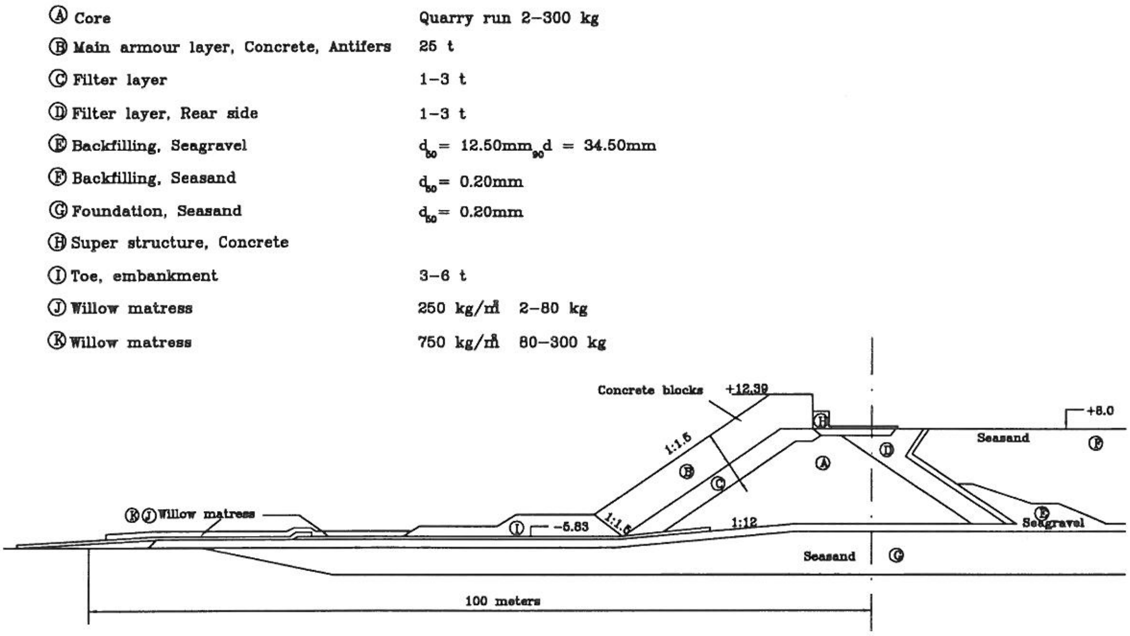

Figure 1: Cross-section of the Zeebrugge breakwater.

randomly placed in two layers, but all with the top of the block facing outwards. The foreshore has depths varying from app. 5 m at the tip of the breakwater downto app. 16 m depth 300 - 500 meters from the breakwater. The tidal range is 0.32 m at mean low water level and 4.62 m at mean high water level. These constraints allows for wave heights till about $H_s = 6m$, wave conditions which the breakwater easily can resist although considerable wave overtopping seems to start already at lower wave heights.

Instrumentation of the prototype breakwater consists of the list shown in table 1. After initial

Instrument	No.	Location	Function
Wave rider buoys	2	150,250 m from breakwater	Wave conditions
Infra-red meter	1	42 m from breakwater axis	Incoming wave conditions and setup
Ext. pressure gauges	2	App. 42 m from breakwater	Waves and setup measured as pressures
"Spider web"	5	Stepgauges placed vertically attached to the face of the breakwater	Run-up and run-down
Integrating box	1	On the top face of one of the antifer cubes	Wave impacts
Pressure gauges	21	See figure 2	Pressures inside the core

Table 1: Instrumentation of prototype breakwater

setup, testing and calibration of the prototype instrumentation all significant storm events have been stored successfully. This paramount task has been coordinated and carried out mainly by the University of Ghent. Figure 2 shows the location of most of the pressure gauges inside the core of the breakwater.

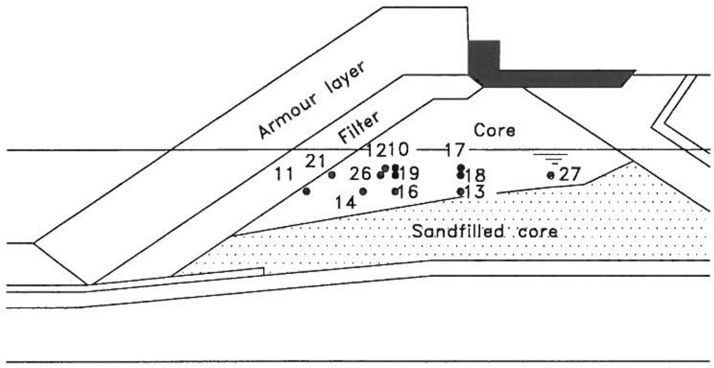

Figure 2: Location of pressure gauges used for analysis in this paper.

1.2 The physical models

Physical model testing has been carried out at three different hydraulic laboratories, namely at Flanders Hydraulics (Belgium) (HRLB), The Hydraulic & Maritime Research Centre (Cork, Ireland) (HMRC) and at The Hydraulics & Coastal Engineering Laboratory (Aalborg, Denmark) (AAU). As tests have not been setup and carried out at the same time and with identical preconditions and aims they differ in a number of aspects besides the scales. However, this does not imply that they are not comparable. Table 2 holds principal information on the tests carried out. As seen in the table six different physical model have been constructed and tested and furthermore

Laboratory	Scale	Scaling law	Note
HRLB	1:20	Froude	With and without sand layer in core. No jetty pile. Scaled bathymetry.
HMRC	1:30	Core material 1:10	Sand layer in core. With/without jetty pile. No bathymetry modelled.
AAU	1:65	Froude, Core material 1:20, Core material 1:40	Sand layer in core. No jetty pile. No bathymetry modelled.

Table 2: Overview of constructed models.

the influence from the measurement jetty pile has been investigated by HMRC. The geometry of all models are scaled using Froude's scaling law whereas distorted scaling of the core material has been used in some models in order to obtain similitude in reynolds numbers for the flow inside the mound. Both tests with regular and irregular waves have been carried out on all models.

1.2.1 Scaling of materials

As one of the main objectives of the project has been to determine the internal hydraulic response of the breakwater it is imperative to give the scaling of materials in the physical models some consideration (Jensen and Klinting 1983), (Oumeraci 1984). In order to apply the Froude scaling law it is assumed that flow is fully turbulent within the breakwater. In small scale models this may not always be true and thus scale effects occur if no measures are taken. Different methods exist proposing alternate scaling of the grain size of the materials used in the core of the models (Méhauté 1965), (Keulegan 1973) and (Jensen and Klinting 1983). After evaluation of the different methods applied on the physical model setup the method of Le Méhauté was chosen for the model at HMRC and one of the models at AAU. It is assumed that scale effects are negligible in the armour layer and that identical gradations are applied in prototype and model. A scale correction factor K is given by

$$\frac{\lambda_p}{\lambda_m} = K\frac{d_p}{d_m} \tag{1}$$

where λ_p and λ_m is the scale of the prototype and model respectively and d_p and d_m are stone diameters. The factor K is determined by

$$K = \frac{H_i}{\Delta L} d_p^3 e_p^5 \tag{2}$$

where H_i is the incoming wave height, ΔL the average width of the core and e_p the porosity of the core in prototype.

1.2.2 Description of physical models

At HRLB two models were tested, one model with sand infiltration in the core and one without. Scaling was done according to Froude scaling law as the flow was expected to be fully turbulent even under laboratory conditions. Stone weights W_m and their sizes were deducted by applying

$$W_m = \frac{W_p}{\lambda_m^3 \frac{\rho_{sea}}{\rho_f}} = \frac{W_p}{\lambda_m^3 \cdot 1.03} \tag{3}$$

and

$$d_m = \sqrt[3]{\frac{W_m}{0.7 \cdot \rho_s}} = \sqrt[3]{\frac{W_m}{0.7 \cdot 2.65 \frac{kg}{m^3}}} \tag{4}$$

where ρ_{sea} is the density of seawater, ρ_f the density of fresh water and ρ_s the stone density.

Tests were conducted in a wave flume with a length of 70 m and a width of 4 m and a depth of 1.45 m. Instrumentation consisted of five wave gauges, a run-up gauge and ten pressure sensors. Tests were carried out variating tide level and wave conditions. Furthermore, tests have been conducted measuring wave impact forces on a specially constructed model armour unit, but results from that study will though not be rendered in this paper.

At HMRC comparable tests were conducted at a general scale of 1:30 in a 25 m by 3 m wave flume. Opposed to the HRLB model distortion of the core material in order to achieve similarity of the flow regime within the core was needed. Compensated stone sizes in the core was determined using the method of Le Méhauté, applying formula 1 and 2. Analogous instrumentation and test programme was applied in order to be able to compare results from HRLB and HMRC.

At Aalborg University tests were conducted in a 12 m by 18 m wave basin. It was decided to construct three models at the scale of 1:65. One model using Froude scaling of the core material as well and one with a compensated scaling according to the method of Le Méhauté (core material scaled 1:20) and finally a model with a distorted scaling of the core material determined on the basis of results from the prior two models (core material scaled 1:40).

In this paper results shown will mainly stem from tests conducted at a tide level of $Z = +4.62m$ in prototype. All results are scaled to prototype before comparisons are made. Instrumentation of all the physical models as well as the prototype breakwater ensures measurements of the wave conditions in front of the breakwater, run-up and run-down levels and pressures at a number of different locations within the core.

2 External hydraulic response

As external hydraulic responses the following phenomena are included: external setup, reflection, up-rush, down-rush and overtopping. These responses is a result of the wave and current conditions which again depends on e.g. wind, tide etc. Within this project almost all the different subjects have been treated. The most apparent entities to investigate is the run-up and the run-down levels as these have been measured in all the physical models as well as on the prototype.

2.1 Run-up and run-down

In the physical models tests have been carried out with both regular and irregular waves, but only the results from the analyses performed on tests with irregular waves are presented here as comparable measurements are available from the prototype, which is obvious not the case for regular waves. Results from the Froude scaled model at scale 1:65 are lacking due to erroneous measurements.

A number of well-known semiempirical or empirical formulae for the determination of run-up and run-down levels exists. Common for these formulae is that they have all been derived on the basis of model tests with rock slopes and not for a breakwater with antifer cubes as armour layer like the Zeebrugge breakwater. However, in order to get an impression of the response measurements are compared with two selected formulae.

When one considers run-up and run-down levels due to irregular wave attack it is necessary to apply a statistical estimate of the levels. Allsop et al. (1985) have suggested an empirical formula for the determination of the run-up level exceeded by 2% of the run-up's $R_{u2\%}$

$$R_{u2\%} = 1.52(1 - \exp(-0.34\xi)) \quad , \qquad \xi = \frac{\tan\alpha}{\sqrt{\frac{H_s}{L_0}}} \tag{5}$$

where α is the slope angle, H_s the incoming wave height determined by zero-downcrossing analysis, L_0 the wave length in deep water and ξ the surf similarity parameter. Likewise for run-down a formula has been proposed by Battjes and Roos (1975) where 5 is implemented in analysis.

$$R_{d98\%} = R_{u2\%}(1 - 0.4\xi) \tag{6}$$

Figure 3 shows the result of the analysis. As indicated above the two curves shown are not a result of a fit to the measured data. As seen on figure 3 the proposed formulae fits the measured data reasonably well. Prototype measurements indicates that the laboratory testing underestimates the run-up and run-down levels assuming that no systematic error has occurred during the prototype measurement campaign or analysis. The difference can arise from the placement of the run-up gauge under laboratory conditions. Scale effects is expected as phenomena such as flow velocities, turbulence, air entrainment and surface roughness are similar in prototype and physical models. These phenomena does not necessarily influence results in the same direction. If an intercomparison of laboratory results are made no significant conclusions can be drawn with respect to scale effects.

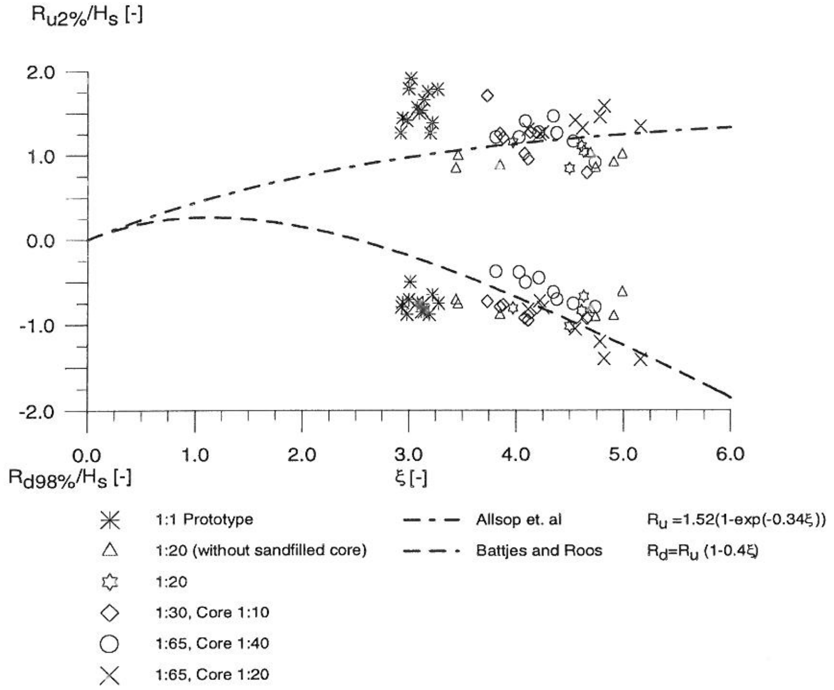

Figure 3: Run-up and run-down analysis for irregular waves.

3 Internal hydraulic response

The Zeebrugge project offers unique possibilities to evaluate different hydraulic responses inside the breakwater. I the following paragraphs internal phreatic setup, pressure wave attenuation and hydraulic gradients will be treated.

3.1 Internal phreatic setup

In this Section the internal set-up is considered. Only results obtained from irregular waves are shown. For the model tests a Pierson-Moskowitz spectrum has been applied. The water level corresponds to $z = +4.62$ m. A simple theoretical formulation of internal set-up is presented (Barends, 1988), and secondly, the measured set-up is shown both as a function of the position in the core and the incident wave height. The set-up is determined as the mean of the pore pressure records within the core of the breakwater (all pressures are equal to zero at SWL).

When considering a conventional breakwater under wave attack, an internal set-up is likely to occur. This internal set-up can be explained by considering the length and cross-section of a flow tube during inflow and outflow. During inflow, the cross-section is relatively large and the penetration length is relatively short. Outflow takes place in the lower part of the breakwater when the phreatic surface is low i.e., the flow lines are long. In order to obtain an outflow which equals the inflow, the outflow velocity must be higher than the inflow velocity, which requires a high pressure gradient. Eventually, this leads to an internal set-up, because this set-up increase the pressure gradient during outflow and decreases the inflow.

3.1.1 Set-up as a function of distance from filter-core interface

The set-up for both prototype and model tests are analysed and evaluated as a function of the horizontal distance from the interface between the filter and the core. From that analysis it can be concluded that the set-up is more or less constant through the core of the breakwater for a given wave condition. Only near the filter layer, the set-up seems to be slightly smaller than further inside the core. However, this is only seen for some of the tests. Because the breakwater is backfilled, it was expected that the set-up would increase with the distance from the outer face of the breakwater. This is only seen for the prototype.

3.1.2 Set-up as a function of incident wave height

The maximum average internal set-up can be estimated by the simple theoretical expression presented by Barends (1988):

$$\frac{s}{D} = \sqrt{1 + \xi F} - 1 \tag{7}$$

where

$$\xi = \frac{0.1cH^2}{n\lambda Dtan\alpha} \quad ; \quad \lambda = 0.5\sqrt{\frac{DKt}{n}} \tag{8}$$

where

s : maximum average set-up [m]
D: depth at toe of slope [m]
c : constant depending on effects of air entrainment and run-up (c>1)
H: wave height at slope [m]
n : effective porosity [-]
λ : penetration length of the cyclic water level into the porous structure [m]
α : slope angle [°]
K: permeability [m/s]
t : period of cyclic loading [S]
F: function depending on the rear side of the breakwater (open or lee-side) [-]

If the Barends formula is simplified where D and C^* are coefficients to be fitted, then the expression can be written as

$$s = D(1 + C^* H^2)^{\frac{1}{2}} - D \tag{9}$$

where

$$C^* = \frac{0.1 \cdot c}{n \cdot 0.5\sqrt{\frac{DKt}{n}} D \tan\alpha}$$

Normally, the magnitude of the internal set-up is within 10-20 % of the water depth at the toe of the breakwater D (Barends 1988), or within 10-20 % of the incident wave height (Bürger et al. 1988).

In figure 4, the set-up at the position of pressure transducer 27 is shown as a function of the incident wave height. The curves attaining the data are fitted to the expression of the Barends type shown in equation 9.

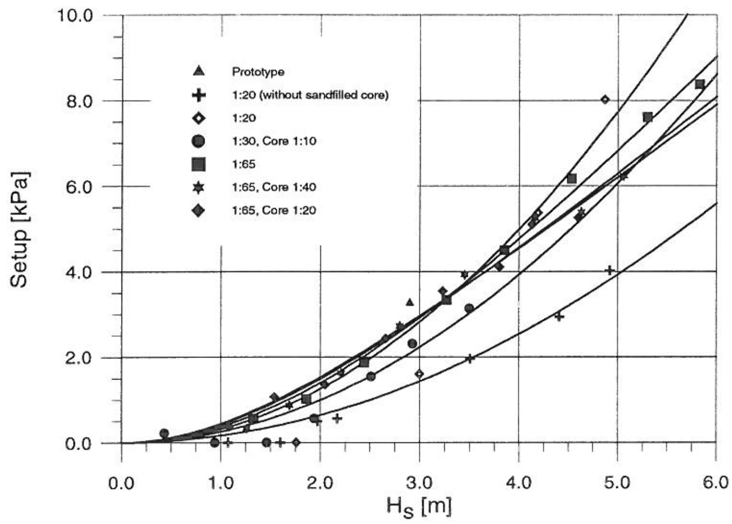

Figure 4: Set-up (pressure transducer 27) as a function of the incident significant wave height.

In general it is seen that there is relatively good similarity between the set-up in the prototype and the different scale models. Only the model in scale 1:20 without sandfilled core yields too small set-up, hence the porosity of this model is too large compared to the other models where the pores are filled with sand.

Furthermore, figure 4 shows that there is no apparent dependency between the set-up and the incident wave height, but that the wave height must exceed 1.0-2.0 m before any notably set-up is developed. The set-up varies between 0 for small wave heights and 15-20 % of the wave height for the largest wave heights, which corresponds very well to the rule of thumb by Bürger et al. (1988).

For the models at scale 1:65 with different core scaling the set-up is expected to be highest in the model with the finest core material. This is seen for the largest wave heights, but the differences in set-up for the three models are not very pronounced.

From these considerations it is seen that modelling the sandfill in the core is more important than modelling the grain size of the original core material.

In figure 4, the set-up determined by the theoretical expression already mentioned is shown for five different periods. Furthermore, the measured set-up is indicated for the different tests. Several parameters are included in the theoretical formulation and the following parameters have been applied in figure 5:

D=14 m
c=1.25 (Barends 1988)
n=0.4
α=33.7°
K=1.0 m/s

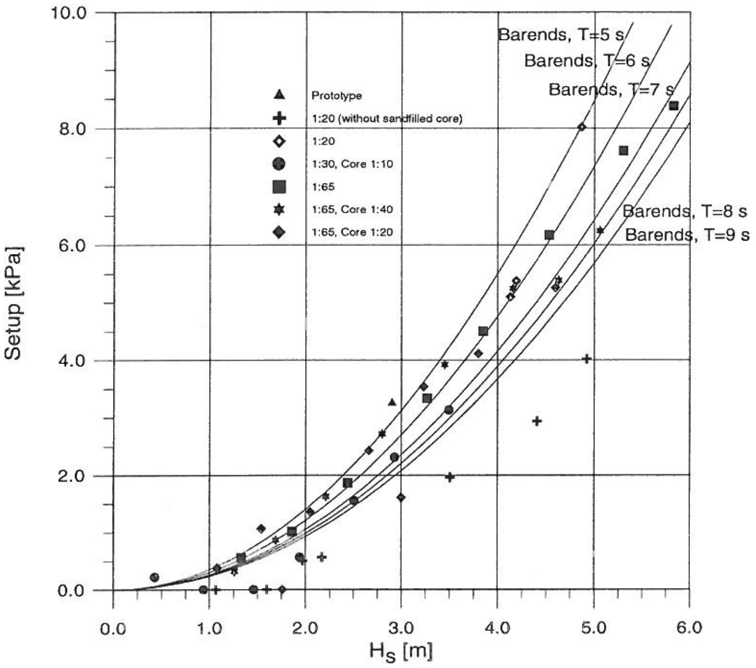

Figure 5: Theoretical maximum average set-up and measured set-up at transducer 27.

The theoretical set-up seems to correspond quite well to the measured set-up. However, the theoretical set-up is a power function of the wave height (for a given period), while the measured set-up increase linearly with increasing wave height as shown in figure 4. This is due to the fact that a Pierson-Moskowitz spectrum has been applied. When the wave height increase, the period increase as well, and thus the set-up will increase because the wave height increase, but it will not increase that fast because the period increase as well. It is seen that also theoretically, the wave height must be of a certain magnitude before any significant set-up is developed

As mentioned above, several parameters have to be determined in order to apply the theoretical expression. The set-up is very sensitive to some of these parameters and e.g. the permeability and the air entrainment factor are very difficult to estimate. Therefore it is possible to manipulate these parameters to fit the measured set-up, and thus the very good similarity between experimental and theoretical set-up as shown in figure 5 should not be taken for more than it is.

3.2 Pressure wave attenuation

The objective of a breakwater is to protect inland areas against the wave forces exerted on the coastline. This is done by dissipation of wave energy by letting the waves propagate through the breakwater. This points out the relevance of investigating the breakwaters capability in this respect. In this paragraph the result of the wave damping analysis is presented. As it is not the wave heights which is measured inside the core but excess pressures related to the hydrostatic pressure. Research has earlier been carried out by e.g (Bürger et al. 1988) and (Oumeraci and Partenscky 1990) where it is suggested that the height of the pore pressure oscillation $P(x')$ of a propagating pressure wave decreases exponentially with the distance to the breakwater interface

$$P(x') = P_0 \exp(-\beta \frac{2\pi}{L'} x') \tag{10}$$

where the following parameters have been applied in the analysis presented in this paper

- x' is the horizontal distance to pressure gauge 21, i.e $P(0) = P_0$
- L' is the wavelength inside the breakwater calculated by $L' = \frac{L}{\sqrt{D}}$ where a value of $D = 1.4$ has been applied. The wavelength L is calculated by the dispersion relationship with T_s, which is the wave period corresponding to H_s, and the waterdepth in front of the breakwater used as input. H_s equals the significant wave height found by zero down-crossing analysis, namely $H_{dn,\frac{1}{3}}$.
- β is the damping factor.
- $P(x')$ is calculated by zero.downcrossing analysis of the pore pressure wave height.

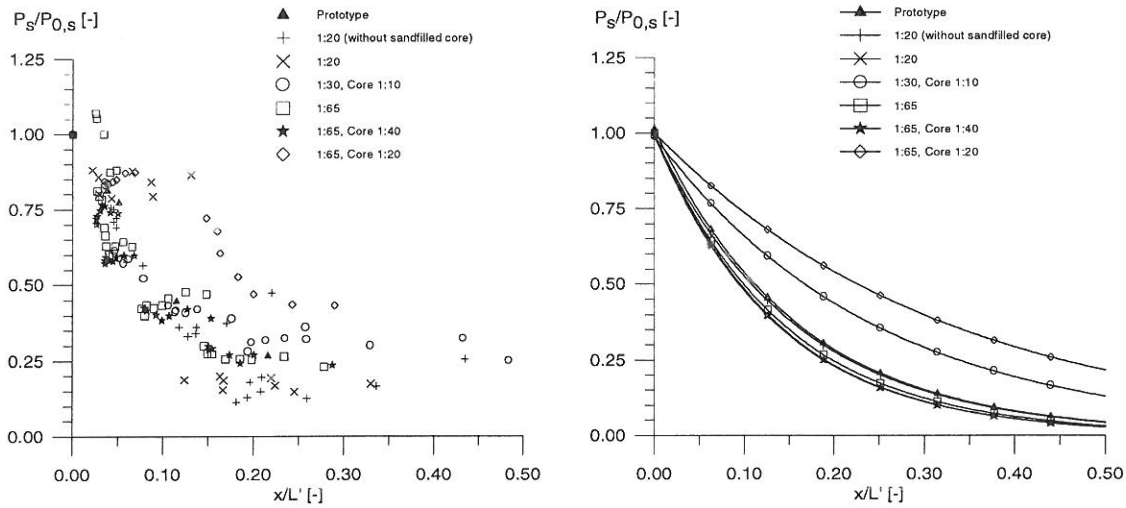

Figure 6: Attenuation related to the excess pressure near the core-filter interface.

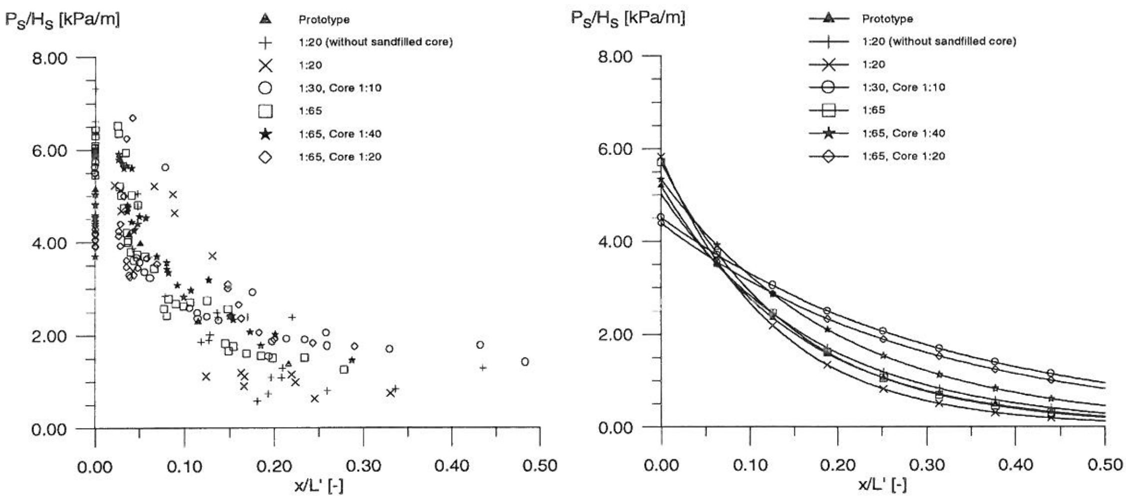

Figure 7: Attenuation related to the incident wave height.

Normally it is assumed that the extended Forchheimer equation (Forchheimer 1901) models the flow resistance in a porous media. Applying the analysis described by equation 10 corresponds to assuming a Forchheimer equation with constant coefficients i.e. a linear relation.

Figure 6 shows the result of the above described analysis. The left-hand side graph shows the measurement points, and the right-hand side graph the curves where β is fitted to the measurements. The points on the right-hand side graph only serves as identification of the curves. If one looks at the measurement points a considerable scatter is observed, but mainly results from the models 1:30 and 1:65 (core 1:20) generates points differing from other tests and the prototype. This observation is also easily seen on the curves. One of the problems with this specific analysis is that fits are forced through the point $(0.0, 1.0)$ leaving out the influence of the wave conditions on the breakwater. If instead the pressure wave height is related to the incoming wave height the results shown in figure 7 appears. This analysis e.g. shows that if an inter-comparison of the models at scale 1:65 is made the model with the finest core material has a large impact pressure but also large damping. In the latter analysis the curves follows the following expression

$$P(x') = a\ H_s \exp(-b\frac{2\pi}{L'}x') \tag{11}$$

Table 3 shows obtained β-values and coefficients a and b.

Model	β	a	b
Prototype	1.01	5.20	1.01
1:20 (without sandfilled core)	1.01	5.02	0.91
1:20	1.16	5.82	1.24
1:30, Core 1:10	0.65	4.51	0.50
1:65	1.11	5.71	1.07
1:65, Core 1:40	1.16	5.34	0.79
1:65, Core 1:20	0.49	4.39	0.53

Table 3: *Achieved damping coefficients*

3.3 Hydraulic gradients

In order to understand the complex phenomena occurring within the core of a breakwater, the hydraulic gradients are very important. In this paragraph hydraulic gradients within the core of the Zeebrugge breakwater are presented for both model scale tests (1:65) and prototype measurements. The gradients are determined for different incident wave heights.

The hydraulic gradients is calculated at one location in the mound using three pressure cells, (PR14-PR16-PR19). The positions of the cells are shown in figure 2. According to the definition in figure

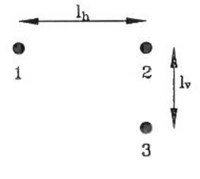

Figure 8: *Mutual location of pressure cells.*

8, the horizontal and vertical component of the resulting hydraulic gradient can be calculated as

$$i_{horiz}(t) = \frac{PR2(t) - PR1(t)}{l_h} \qquad i_{vert}(t) = \frac{PR2(t) - PR3(t)}{l_v} \tag{12}$$

whereas the resulting gradient and its direction are determined by

$$i_{res}(t) = \sqrt{i_{horiz}(t)^2 + i_{vert}(t)^2} \qquad \tan\Theta(t) = \frac{i_{vert}(t)}{i_{horiz}(t)} \tag{13}$$

Figure 9 shows a plot of the different hydraulic gradients for some of the tests. The plots are generated by calculation of the resulting gradient and its direction for each timestep in each time series and subsequently averaging the results within every 10 degrees. A closed path is obtained from each of the sea states tested in the laboratory i.e., the plots obtained from model tests at scale 1:65 each contain eight closed paths originating from the eight different tests with different wave heights (and periods). The model test data is scaled to prototype and axes are shown as [kPa/m].

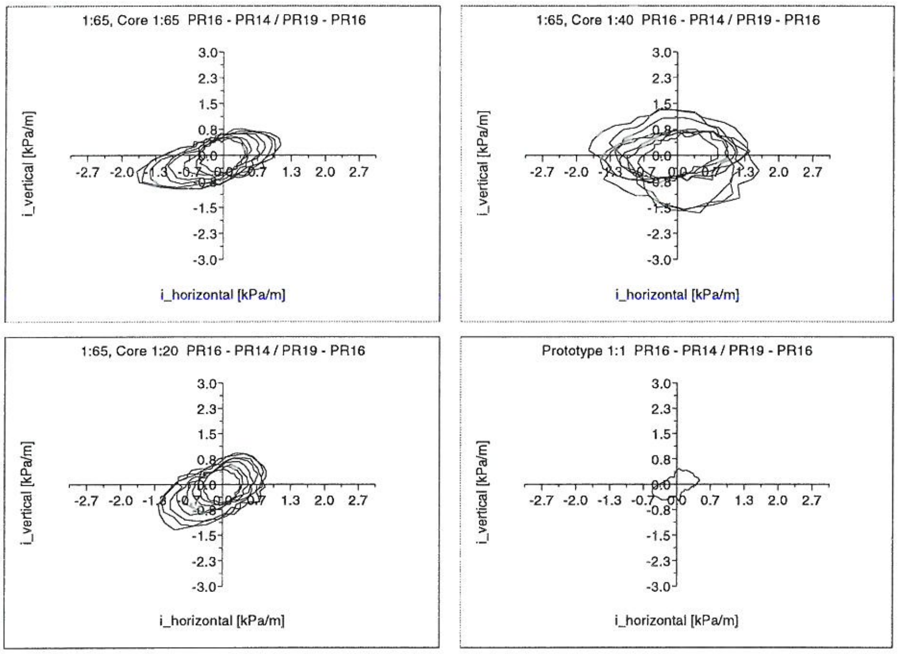

Figure 9: *Pressure gradients, PR14-PR16-PR19.*

The relevance of these kinds of plots is dependent of the placement of the pressure cells as well as the desired information to be extracted. If measurements were made within or just below the main armour layer, it could be of interest to investigate the relation between the size and direction of the prevailing gradients and the stability of the armour in terms of a critical gradient. In this case, though, the measurements are only made within the core of the breakwater and the plots can therefore only be interpreted by mutual comparisons and comparisons with phenomena related to the core such as internal setup and damping.

It is seen in figure 9 that the hydraulic gradient increase for increasing wave height (the "diameter" of the closed paths increase, and the innermost curve of course correspond to the smallest wave height), and in most cases the shape of the path is independent of the wave height.

In general, the magnitude of the gradients from the model tests are much larger than the the corresponding gradient in the prototype. This leads to the conclusion that the models constitute more dense structures than expected (also for the models with distorted core material) or that the prototype may be more permeable than expected. Perhaps the sand infiltration is not as pronounced as expected. These observations agree with the fact that a linear scaling of the grain material size leads to a too dense structure in the smaller scale model where the Reynolds number is considerably smaller. However, the shape of the gradients are relatively similar in prototype and model tests.

It is seen that an assumption of no vertical and only horizontal hydraulic gradients i.e., a purely hydrostatic pressure distribution, which is implemented in a number of numerical models cannot be justified on the basis of the results. The results show that the flow inside breakwaters must be considered as a complex flow in at least two dimensions.

4 Conclusion

In order to avoid any misunderstandings, diverse habits, etc. by different researcher all presented results are the outcome of analyses made at one institution using identical calculation procedures on the raw time series. This only rules out a few possibilities for errors and uncertainness of that specific nature - but unfortunately opens up for other kinds e.g. systematic calculation errors on all data.

In prototype it is not necessary to worry about scale effects. Instead other problems appear when dealing with large installations. The following items may lead to uncertainness and errors:

- Characterisation and quantification of construction materials used for the breakwater.
- Sensors placement being representative for the phenomenon it is supposed to measure.
- Spatial location of the various instruments.
- Calibration and validation of measurements.
- Analysis and interpretation of data.

During this project very thorough work has been done in order to minimise problems arising in connection with the above list. Some of the instrumentation e.g. the spider web, has not been in function for a very long period yet and thus may still be connected with undiscovered problems. In the physical models the same problems with respect to uncertainty, errors and systematic errors occur. Used materials in the models may not exactly equal desired materials and specific porosities and geometries may be difficult to reproduce in the laboratory as well as validate afterwards.

In addition in the physical models scale effects can influence results. These scale effects may affect the results in different directions dependent on the entity concerned. In this project the physical model testing has been conducted at three different laboratories and this may furthermore entail problems due to different measuring techniques etc.

If presented results in this paper are evaluated with respect to scale effects unambiguous conclusions cannot be drawn. The results does not show sign of scale effects or rather the scale effects occurring in the physical models downto the scale of 1:65 appears to be of smaller magnitude than the variations observed in the data. These variations stem from the natural variations due to the stochasticity of the measured phenomena and the variations originating from problems with errors and uncertainness.

In order to quantify the attaining uncertainty and scale effects it can be concluded that it is necessary to perform measurements under optimally controlled conditions. Within this project alone the fact that different teams of researcher at different institutions and laboratories have performed the measurements is tantamount with the impossibility of this quantification. This is on the other hand a very positive conclusion as results from the different models and prototype as shown in the paper demonstrate comparably good coherence which suggest that scaled models of breakwaters in general are well representing the prototype conditions, in this case at least downto the scale of 1:65.

Acknowledgement

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