CHAPTER 149

FULL SCALE MEASUREMENTS OF WAVE ATTENUATION INSIDE A RUBBLE MOUND BREAKWATER

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ABSTRACT

At the Zeebrugge harbour (Belgium) a cross-section of the N.W.-breakwater has been instrumented for the study of physical processes related to the behaviour of a prototype rubble mound breakwater in random wave conditions. Within the EC MAST programme (project MAS02-CT92-0023) this monitoring system has been re-engineered and extended to a high-quality full scale data acquisition centre (Troch et al., 1995).

The development of the prototype monitoring system to a world-wide unique system with respect to the infrastructure available at Zeebrugge, the instrumentation installed on site, and the data management developed, is briefly summarised.

Filed measurements of wave attack in front of the breakwater, and pore pressure response inside the breakwater core, have been analysed in order to determine the hydraulic response of the full scale breakwater. Analysis results on wave run-up/run-down measurements, phreatic set-up calculations, and pore pressure wave attenuation are presented here in more detail.

INFRASTRUCTURE

The port of Zeebrugge is situated on the eastern part of the Belgian coastline, and is protected by two main breakwaters (Fig. 1). The Zeebrugge breakwater constitutes of a conventional rubble-mound breakwater with a low superstructure and an armour layer consisting of grooved cubes (25 ton). The breakwater core consists of quarry run 2-300 kg, the filter layer is made of rock 1-3 ton. On the breakwater crest, a service road enables easy access of the breakwater. The tidal range at spring tide is 4.3 m.

A measurement jetty of 60 m length supported by a steel tube pile at the breakwater toe and by concrete columns on top of the breakwater is situated on the NW-breakwater, Fig. 2. Six boreholes have been drilled in the core: four vertical boreholes and two oblique boreholes. Galvanised steel casings are placed in these boreholes. These casings are perforated in order not to disturb the overall permeability. Pressure sensors are mounted in these casings. Each pressure sensor cable is protected by a high density polyethylene tube provided with a perforated nylon head at the sensor end.

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In an air-conditioned container, placed on the landside of the breakwater, signal conditioning apparatuses and a data acquisition system (DAS) are installed. All electric cables from the measuring sensors are lead towards the container.

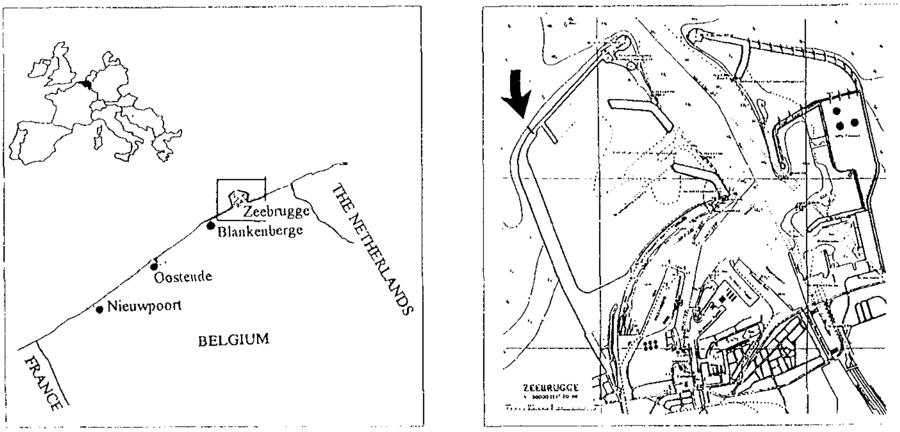

Fig. 1. Location of the prototype monitoring system on the NW breakwater at Zeebrugge harbour (Belgium).

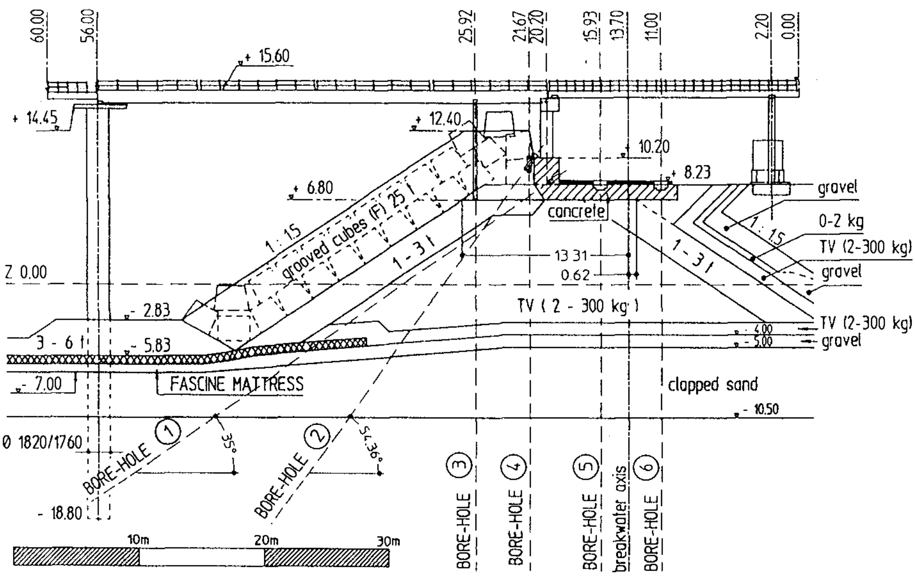

Fig. 2. Cross-section of the Zeebrugge rubble mound breakwater.

INSTRUMENTATION

An overview of instrumentation installed on the measurement jetty is given in Fig. 3, and summarised in Table 1.

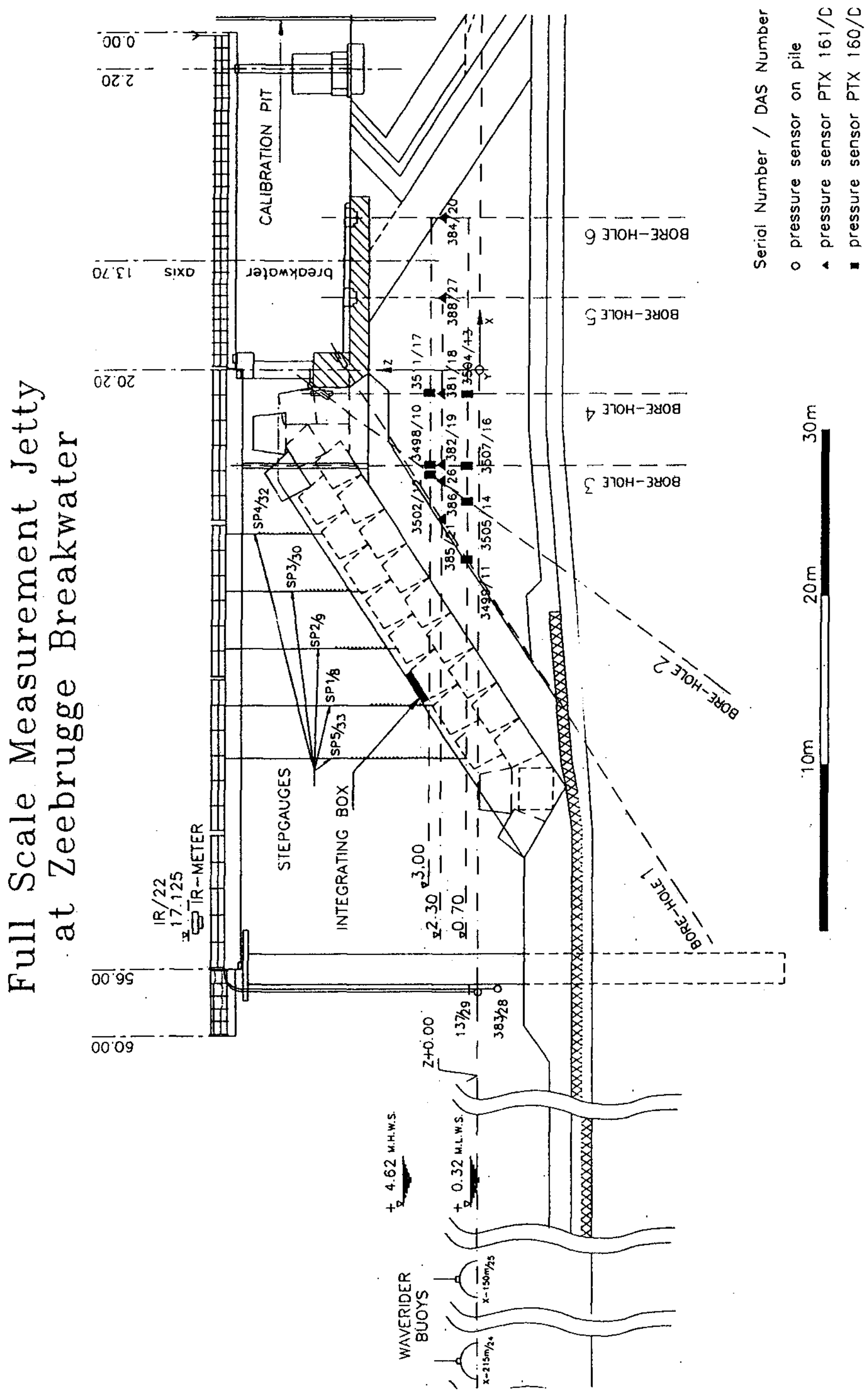

Fig. 3. Zeebrugge breakwater: Location of instrumentation since 1994.

Wave rider buoys are located in front of the breakwater and measure the incident waves. The water level at the toe of the breakwater is measured by an infra-red wave height meter and by two submerged pressure sensors located in front of the steel tube pile.

A set of vertically placed stepgauges between the measurement jetty and the armour layer is able to registrate the wave run-up and run-down on the armour layer.

Inside the core 13 pressure sensors are installed in the six boreholes for the measurement of the internal pore pressures induced by the waves. Assembly of the sensors is conceived to allow flexible placement in the boreholes, ease of maintenance and ease of calibration.

A device called "integrating box" is designed for the measurement of wave forces, i.e. the integrated pressures on (a part of) an armour unit face. The solid steel box is filled with water under pressure. A pressure sensor mounted in the box measures the pressure variation in the enclosed box volume. The device has been calibrated extensively in the laboratory. The pressure inside the box is related to the wave force applied on the box membrane.

INSTRUMENTATION	LOCATION	AIM OF MEASUREMENT
2 waverider buoys	150 m and 215 m from breakwater	wave records
infra-red wave height meter	near steel tube pile	wave records, tide level
2 submerged pressure sensors	near steel tube pile	wave records, tide level
5 vertically placed stepgauges	between armour layer and jetty beam	run-up/run-down wave profiles
'integrating box'	on one face of armour unit	wave forces on an armour unit
13 pressure sensors	inside rubble core	pressure records

Table 1. Instrumentation of Zeebrugge breakwater.

DATA MANAGEMENT

A strategic data management plan is developed for acquisition, processing and distribution of all full scale data.

Each sensor has its dedicated signal conditioner providing individual isolated power supply and converting the current of the pressure sensor circuit to a voltage output -proportional to the physical input- which is galvanically isolated from the sensor circuit. The voltage outputs of all sensors are separately connected to an isolation amplifier followed by a lowpass-filter before being connected to the data acquisition system. Special care with regard to groundings and shielding of the electronic devices has been taken in order to prevent ground loops.

A scanner samples all analog data at 10 Hz. After analog-to-digital conversion raw data are stored as binary files on a hard disk of the computer. The data acquisition computer manages the hard disk as a ringbuffer.

An software package, written in ANSI C and portable on several systems, is included for presentation, evaluation and signal analysis. Software tools were written to read data from file, display data on screen, select pieces of data and analyse the selected data. Routines for input and output have been implemented allowing the user to output measurements values and computational results and to input signals having a format different from the one of Zeebrugge data. Procedures have been developed for the quality control of raw data. This way an interface today is available for the input of external data from scale models and numerical models for comparison and research activities.

The acquired time series are edited for the elimination of errors in the signals. A very valuable technique, developed in speech analysis, and applied here to the signals, is the calculation of a spectrogram of a signal. The spectrogram (Fig. 4) consists of consecutive spectra, calculated for consecutive windows on the time series and placed vertically. The spectral energy information per frequency interval is converted towards a proportional line length. This way, errors in the signal, such as high frequency noise or sample gaps, are easily detected and edited.

Up to date about 16 storms with significant wave heights ranging between 1.00 m and 3.50 m, and wind directions from N.W. -allowing almost perpendicular wave attack- have been registrated. Continuously improvements and new instruments are prepared and installed, keeping the prototype monitoring system highly operational. For more detailed information and technical details of instrumentation and data acquisition, the reader is referred to (Troch et al., 1996-a; Troch et al., 1996-b).

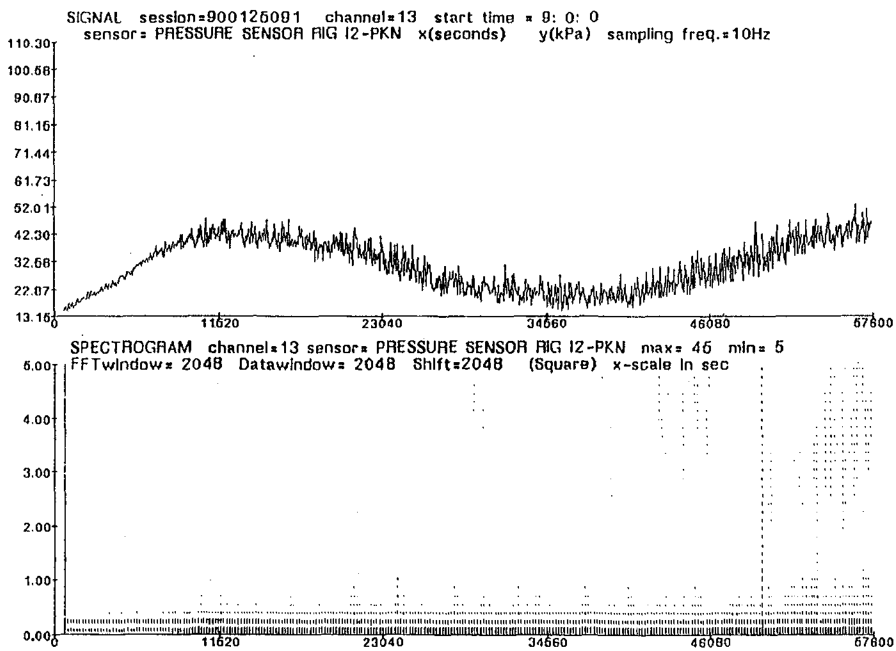

Fig. 4. Typical acquired time series and spectrogram of pressure sensor signal.

FULL SCALE DATA ANALYSIS RESULTS

Wave run-up measurements

The wave run-up (run-down) derived from full scale measurements is defined as the vertical distance between Mean Water Level (MWL) and the highest (lowest) point attained by the wave up-rush (down-rush) on the slope. The run-up height R_u is one of the most important parameters associated with wave loading on rubble mound structures and their stability. It, along with the highest still water level, determines the necessary crest level of the breakwater.

At each stepgauge mounted along the breakwater slope, the instantaneous water surface level is measured. From these time series, the instantaneous wave profile on the slope is derived. At one instant, the wave profile is taken as the polygon connecting the water surface levels measured at all stepgauges. The instantaneous wave run-up height is defined here as the vertical distance between MWL, and the intersection of the breakwater slope with an extrapolated line, constructed trough the two highest water surface levels measured at that instant. MWL is calculated as the mean of a water surface measurement using a submerged pressure sensor mounted in front of the steel tube pile. When the intersection is positioned below MWL, instantaneous wave run-down is identified.

The relevant run-up level of an irregular wave signal is taken as $R_{u,2\%}$. This is the level exceeded by only 2% of the waves running up the armour slope. The run-down level considered is the $R_{d,98\%}$ which is the level exceeded by 98% of the waves. The reference number of waves used for the computation of the exceedence level, is taken here as the total number of run-up waves on the slope. The non-dimensional run-up ratio $R_{u,2\%}/H_{mo}$, where H_{mo} is the significant incident wave height, and run-down ratio $Rd_{,98\%}/H_{mo}$, are plotted versus the deep water Irribarren number of the wave form:

$$\xi = \frac{tan\,\alpha}{\sqrt{\dfrac{H_{mo}\;2\pi}{gT_{(0,2)}{}^2}}} \qquad (1)$$

where α = slope angle (°); significant wave height $H_{mo} = 4(m_0)^{1/2}$ (m); wave period $T_{(0,2)} = (m_0 / m_2)^{1/2}$ (s), where m_n is the n^{th} moment of spectral density.

Allsop et al. (1985) presented model test run-up results on a 1:1.5 Antifer cube slope with irregular waves, using Losada et al.'s (1982) relationship:

$$\frac{R}{H} = A\left[1 - exp(B\xi)\right] \qquad (2)$$

where R = run-up/run-down level, defined as: $R_{u,2\%}$ or $R_{d,98\%}$; $H = H_{mo}$; A, B = experimental coefficients; ξ = Irribarren number. Allsop reported:

run-up:	A=1.52	B= -0.34

From physical model tests on the identical Zeebrugge breakwater cross-section, carried out in the MAST II project (Kingston et al., 1996), it is found that for irregular waves the coefficients are:

run-up:	A= 1.76	B= -0.28
run-down:	A=-1.05	B= -0.43

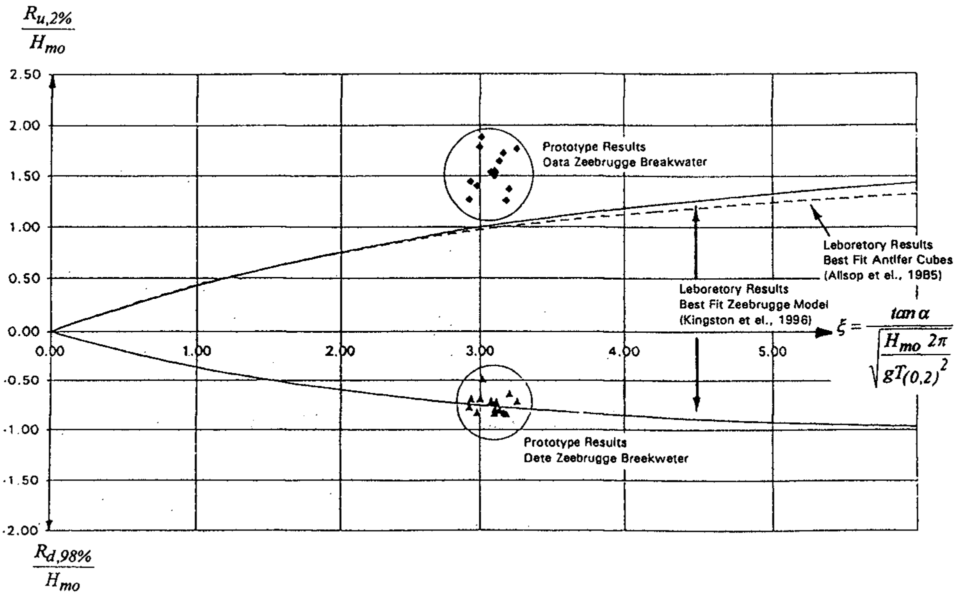

Fig. 5. Comparison between prototype and scale model wave run-up results.

In Fig. 5 non-dimensional wave run-up heights and wave run-down heights obtained from Zeebrugge prototype measurements using the vertically placed stepgauges are plotted as a function of the Irribarren number ξ. Note that the range for ξ is small due to the occurred wave and breakwater characteristics. As a result, for $\xi \approx 3$ the wave run-up $R_{u,2\%}/H_{mo}$ has order of magnitude 1.50, and the wave run-down $R_{d,98\%}/H_{mo}$ has order of magnitude -0.75.

Comparing with the laboratory wave run-up measurements (Allsop et al., 1985; Kingston et al., 1996), it appears that from this analysis of prototype data the wave run-up is about 50% higher than the well-accepted wave run-up, based on laboratory results. Wave run-down agrees well with laboratory data. Due to the measurement technique, mainly the remaining distance between the gauge and the slope, the run-up measured in laboratory tests might be underestimated by up to 20 %.

This high run-up $R_{u,\,2\%}$ confirms the practical experience in Sines (Portugal) and Zeebrugge harbours that wave overtopping is higher than expected during design. As prototype results are only available for one storm within a narrow range of Irribarren numbers, they have to be confirmed by future measurements. It is clear that a better measurement accuracy is needed before final conclusions are drawn. However, this remarkable result, if confirmed, would have large impact on social, economic and environmental aspects of coastal protection works. As a conclusion, more detailed research and measurements on wave run-up are necessary in order to acquire a better understanding of prototype wave run-up.

Internal phreatic set-up

Due to the geometric non-linear effect on the slope subjected to waves, swell or tides, an internal phreatic set-up in a breakwater may occur. As the inflow section along the slope at the moment of a high water level is larger than the outflow section at the moment of a low water level, and as the average inflow path is shorter than the outflow path, more water will enter the breakwater than can leave during cyclic water level changes. Consequently, an average internal set-up of the water level inside will occur.

This internal maximum average set-up occurring after several cycles may be described by a simple theoretical formula (Barends, 1988):

$$\frac{s}{D} = \sqrt{1+\xi.F} - 1 \tag{3}$$

with:

$$\xi = \frac{0.1cH^2}{n\lambda D\tan\alpha}\ , \ \lambda = 0.5\sqrt{\frac{DKt}{n}} \tag{4}$$

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 = porosity (-); λ = penetration length of the cyclic water level into the porous structure (m); α = slope angle (°); K = permeability coefficient (m/s); t = period of cyclic loading (s); F = function related to two cases (open or closed lee-side). For a closed lee-side the maximum set-up s is found at the lee-side.

As a practical example of the theoretical formula the Zeebrugge case is worked out. Barends (1983) assumed that the permeability $K \approx 0.50$ m/s and the porosity n=0.40. The incident wave is characterised by H=6.50 m and T=9 s. The waterdepth D is 11 m. The penetration length is approximated by λ=5.56 m. The situation corresponds to a closed lee-side and s occurs at lee-side. With $L \approx 85$m and L/λ=85/5.56=15.3 m, F(closed)=1. With $\tan\alpha$=0.67, c=1.25 (according to Barends, 1988) and H=6.50 m, ξ becomes 0.32. The maximum average set-up s derived from the theoretical formula (*3*) finally is 1.65 m. This theoretical result is in between the rule of thumb values proposed by Barends. With D=11 m, s=0.15D: the magnitude of the internal set-up is within 10-20 % of the waterdepth at the toe of the breakwater.

A permeability coefficient K with the same order of magnitude is found by (Gudehus, 1974) from tests on rock samples characterised by d_{10}=60 mm, d_{50}=140 mm, d_{90}=190 mm. Dimensions of the test specimen are: height=1.20 m, diameter=1.22 m. The rock dimensions are clearly lower than the 2-300 kg rock used in the core. From this consideration, it may be assumed that the permeability of the core is higher than 0.50 m/s, however no measured values are available. Assuming that the permeability is 10 times higher, K=10x0.50=5 m/s, the set-up reduces to 0.50 m.

Thus, set-up is very sensitive to some of the parameters included, such as the permeability and the air entrainment factor. These are difficult to estimate. The results need to be treated with much care.

In order to study the stability of the filter layers of the Zeebrugge breakwater, a calculation with the mathematical model HADEER (Barends, 1983) was part of the study. Results from these calculations show similar set-up values as calculated from (*3*).

From full scale measurements, maximum average set-up values s, calculated at the position of the pressure sensor located closest to the leeside, are reported in Table 2. The Mean Water Level (MWL) is derived by calculating the mean of a wave record registrated by the infra-red meter. Inside the breakwater core pressure data (Pa) are converted into water level data (datum Z) by assuming a hydrostatic section where the density is constant. Those assumptions do not apply in the hydrodynamic area near the armour layer, but become more reasonable moving in the leeward direction. Internal phreatic set-up is calculated as the difference between mean values over a time interval of 180 s of the infra-red wave meter outside and the pressure sensors inside the core.

H_i [m]	s [m]	s/D [%]	s/Hi [%]
3.50	0.29	2.6	8.3
3.05	0.33	3.0	10.8
2.25	0.37	3.4	16.4
2.00	0.36	3.3	18.0

Table 2. Maximum average set-up s and rule of thumb ratios.

Comparison is made of calculated prototype set-up values with orders of magnitude proposed in literature: the results from prototype data are not within the range proposed by (Barends, 1988): $0.10D \leq s \leq 0.20D$, but the results have the same order of magnitude, as found by (Bürger et al., 1988) on large scale models: $0.10H_i \leq s \leq 0.20H_i$, where H_i = incident wave height $H_{1/3}$ (m). Ratios s/D and s/H_i used in the rules of thumb are summarised in Table 2.

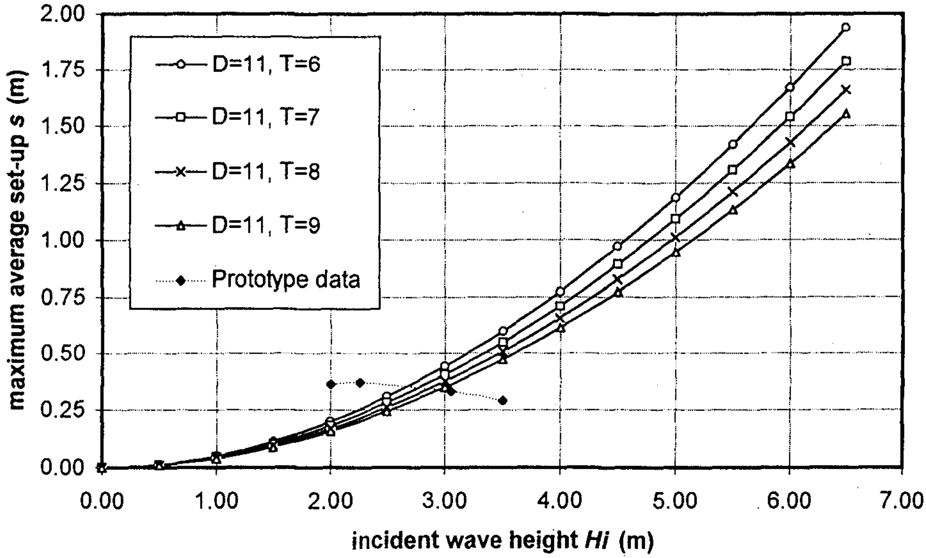

Fig. 6. Comparison between theoretical and prototype set-up calculations.

Barends' theoretical formula (*3*), where depth at toe D=11m, wave height H=3m, wave period T=7.2 s gives a setup value of s=0.41 m at the landward side. This result is of the same order of magnitude of the full scale data, as extrapolating the pressure gradient towards the landward side yields s=0.39. Using K=5 m/s, set-up reduces to 0.12 m.

Fig. 6 shows the set-up for the Zeebrugge case as a function of wave height and period for D=11 m. Further to Barends' theoretical formula the setup increases with increasing wave height. Plotting prototype setup as a function of H_i on Fig. 6 shows no agreement with this suggested non-linear relationship between setup and H_i.

The set-up is evaluated as a function of the horizontal distance from the interface between the filter and the core. It is concluded from prototype measurements that the set-up increases with the distance to the outer face of the breakwater. This is in agreement with the Barends' formula, because the breakwater is backfilled.

Pore pressure wave attenuation

The main objective of a breakwater is to dissipate incident wave energy when waves propagate through the porous breakwater core. This points out the relevance of investigating the breakwater's capability in this respect. In this section, results of full scale wave attenuation are presented. It is not the wave heights which are measured in the core, but the pore pressures.

Theoretical work dealing with oscillatory flow in porous media was carried out by Biesel (1950) who identified the form of the spatial and temporal relationships which describe a linearly damped oscillatory flow. Le Méhauté (1957) applied this relationship to rubble mound breakwaters by introducing parameters accounting for porosity and inertia effects of the porous material. Oumeraci (1990) summarises Le Méhauté's theory on this topic and concludes that the height of the pore pressure oscillation $p(x)$ of a propagating pressure wave decreases exponentially with the distance to the breakwater interface according to the linear damping model:

$$p(x) = p(0)\exp(-\beta \frac{2\pi}{L'} x) \tag{5}$$

where x = co-ordinate across core (m); $p(x)$ = pore pressure at x (N/m²); $p(0)$ = pore pressure at x = 0 (N/m²), i.e. the first pressure sensor location, L' = wave length within breakwater (m), β = damping factor (-).

The wave length within breakwater L' is calculated by $L'=L/\sqrt{1.4}$, (Oumeraci, 1990). The wavelength L is calculated by the dispersion relationship using wave period $T_{(0,2)}$, and waterdepth in front of the breakwater toe D. The pore pressure height $p(x) = 4(m_0)^{1/2}$, is calculated from the energy content of the signal.

The magnitude of the damping factor β depends on the distance y_s of the horizontal level of the sensors to the MWL: the lower the location under MWL, the smaller the friction losses will become as the degree of turbulence decreases, and the smaller the damping factor β becomes.

In Fig. 7, a number of wave records of 15 minutes are considered for the horizontal level of sensors at Z+2.30. The ratio $p(x)/p(0)$ is plotted versus the relative distance x/L'. An exponential curve is fitted through the data, the resulting damping factor β=0.68. In Fig. 8 curves with damping factors 0.81 (Z+3.00), 0.53 (Z+0.70) are plotted as well. As seen from this Fig. 8 the damping factor β decreases with increasing distance

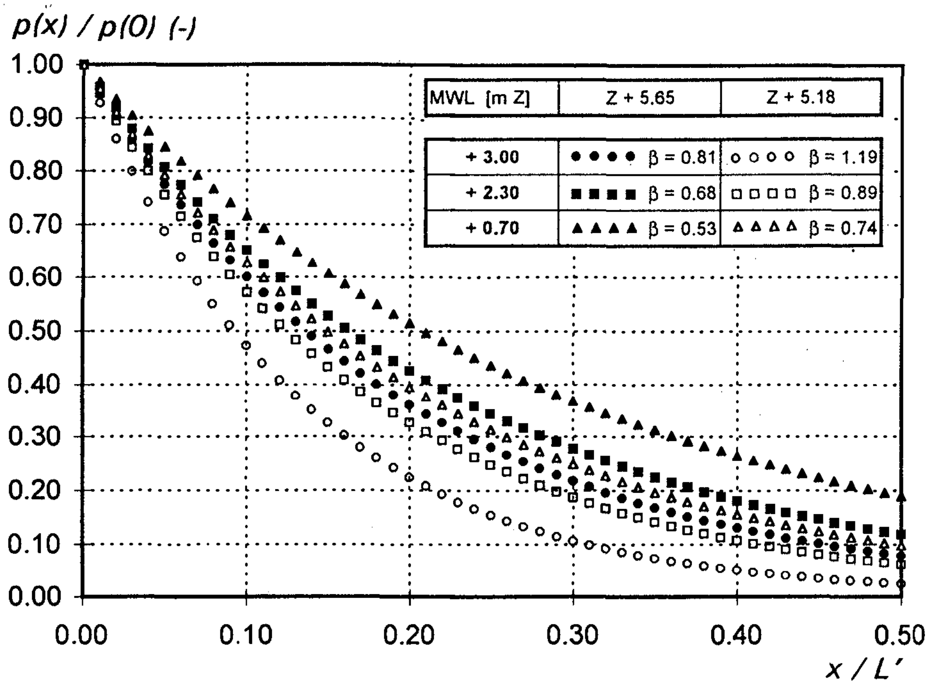

Fig. 7. Damping factor β from the three sensor levels for two MWL's.

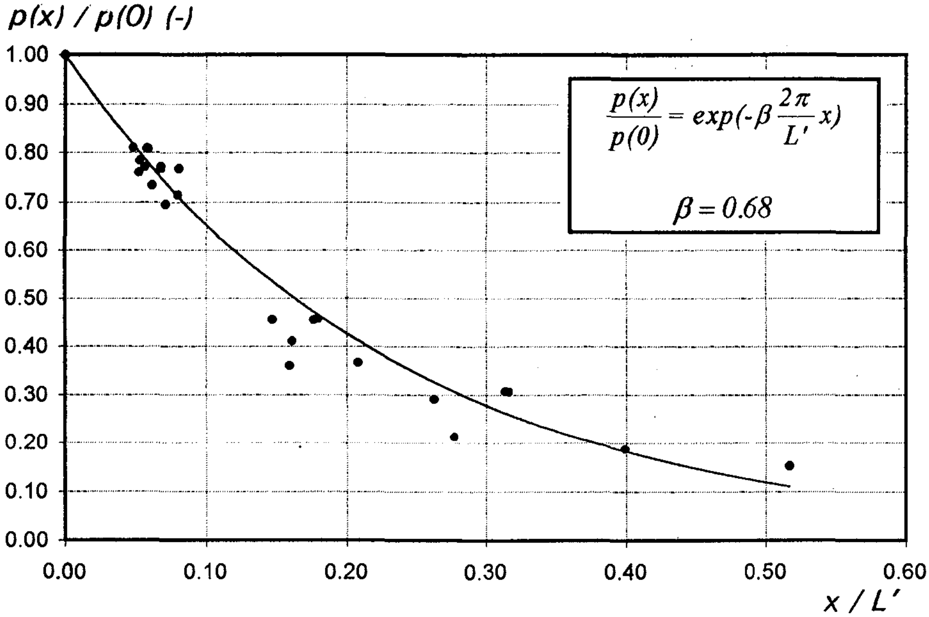

Fig. 8. Prototype pore pressure wave attenuation inside breakwater core: prototype data and fitted linear damping model.

from the MWL (MWL at Z+5.65). In the same Fig. 8 the blank dotted lines have resulting damping factors β for a situation with lower MWL (at Z+5.18). Compared to the former β values at each level, the latter values are higher (e.g. 0.89 versus 0.68 at sensor level Z+2.30). Indeed there is more turbulence due to the lower MWL, and the damping factor β depends on the distance y_s.

Analysis of a comprehensive set of data (Troch et al., 1996-c) shows that the energy content of steeper waves decays much nearer to the front of the core, where x=0. In other words, the rate of attenuation within the core for a given wave period increases with increasing wave height. In contrast the opposite trend is evident for a constant incident wave height. Increasing the wave period resulted in a decrease in the rate of attenuation in the mound. Summarised, the damping factor β also depends on the incoming wave height and period.

It is assumed (Burcharth et al., 1995) that the extended Forchheimer equation models the flow resistance in porous media:

$$i = au + bu^2 + c\,\frac{\partial u}{\partial t} \tag{6}$$

where i = hydraulic gradient (-); u = bulk (or filter) velocity (m/s); a, b, and c = constants. However, the turbulent term (u^2) in equation (*6*) is normally predominant when considering internal flow in breakwaters, and this turbulent term does not occur in (*5*). Hence there is no evident purpose in relating the β-value in equation (*5*) to the coefficients in the Forchheimer equation. From the above it is suggested that the model derived by (Oumeraci et al., 1990) is too simple being a linear model compared with the non-linear Forchheimer model. Moreover a straight comparison between calculated β values, based on prototype results, and the value reported by (Oumeraci et al. 1990), β=2 is not possible because of the different geometrical shape factors, the different breakwater cross section and scaled core materials affecting the core permeability, and different position of the first pressure sensor (the first reference pressure sensor in Oumeraci's analysis is positioned in the filter layer between armour and core).

CONCLUSIONS

This paper briefly outlines the prototype monitoring system installed at the Zeebrugge rubble mound breakwater. To date a comprehensive set of full scale data have been acquired at this unique monitoring system during storm conditions with significant wave heights ranging between 1.00 m and 3.50 m, and are available for validation of physical and numerical models. Prototype measurements consisting of incident wave data, wave run-up data, and pore pressure data have been analysed in order to determine the hydraulic response of the full scale Zeebrugge breakwater. As hydraulic responses the following phenomena are included: wave run-up/run-down, internal phreatic set-up and pressure wave attenuation.

Comparing prototype wave run-up measurements with well-accepted laboratory results, it appears that the prototype wave run-up is about 50% higher than the wave run-up on armoured slopes of scale models. Wave run-down agrees well with laboratory data. This remarkable result, if confirmed, would have large impact on social, economic and environmental aspects of coastal protection works. As a conclusion, more detailed research and measurements on wave run-up are necessary in order to acquire a better understanding of prototype wave run-up.

Prototype set-up measurements are compared with a simple theoretical formulation. For the range of wave heights available, there is good agreement between prototype and theoretical set-up. However no clear dependence of set-up on wave height, as suggested by the theoretical formulation (*3*), is found. The order of magnitude of calculated set-up from prototype measurements is between 10 and 20 % of the incident wave height.

Results show that the pore pressure height $p(x)$ of a wave travelling through a rubble mound breakwater in the horizontal direction decreases exponentially, and the magnitude of β depends on the distance from the horizontal plane to the mean water level. Finally some remarks on the applicability of the linear damping model, based on the Oumeraci formulation are made. There is a clear dependence of the damping factor on the incident wave height and period, which is neglected by the linear damping model. There is no non-linear Forchheimer term present in the linear damping model, although this term is predominant for internal porous flow.

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