

WAVE RUN-UP AND OVERTOPPING: PROTOTYPE VERSUS SCALE MODELS

Julien DE ROUCK¹, Raf VERDONCK¹, Peter TROCH¹,
Luc VAN DAMME², Flemming SCHLÜTTER³, John DE RONDE⁴

ABSTRACT

The determination of the crest level is one of the most important points in the design of sloping coastal structures. The crest level of sloping coastal structures is governed by wave run-up and overtopping. Recent measurement results of run-up on prototype have indicated that wave run-up may be underpredicted by scale model tests. So full scale measurements of wave run-up is necessary. At the same time the overtopping discharge on a prototype breakwater will be measured. Within the MAST III - OPTICREST project, these measurements will be carried out at coastal structures at two locations: Zeebrugge (Belgium) and Petten (The Netherlands). The results obtained from these prototype measurements will be compared with those from scale model tests and will be used to calibrate numerical models. This paper also presents the contents of the OPTICREST project.

1 INTRODUCTION

In the past few decades, the dimensions of seagoing vessels have increased strongly. As a result, many harbours have been constructed in open sea. Harbours need to facilitate smooth and unhindered transfer of passengers and cargo between vessels and land. In order to prevent storms from impeding these harbour activities, the harbour area has to be sheltered.

Sloping structures, such as rubble mound breakwaters are often used for such harbour sheltering purposes. Yet some aspects of their design still remain unsolved. One of these aspects is the crest level. The level to which breakwaters should be built is governed by the phenomena of wave run-up and overtopping. A wave run-up level is

¹ University of Ghent, Dept. of Civil Eng., Technologiepark 9, B-9052 Ghent, Belgium

² Ministry of the Flemish Community, Coastal Division, Vrijhavenstraat 3, B-8400 Oostende, Belgium

³ Aalborg University, Hydraulics and Coastal Eng. Lab., Sohngaardsholmvej 57, DK-9000 Aalborg, Denmark

⁴ Rijkswaterstaat, Kortenaarkade 1, PO Box 20907, NL-2500 ex The Hague, The Netherlands

defined as the vertical distance between the Still Water Level (SWL) and the highest point reached by the wave up-rush on the slope.

Recent research (De Rouck et al., 1996; Troch et al., 1996b) showed that wave run-up may be underpredicted by model tests. Since the design of the crest level of sloping coastal structures is based on such tests, or on formulae derived from such tests, this has serious implications.

In order to establish better design rules for the crest level of sloping coastal structures, prototype measurements of wave run-up and overtopping are essential.

Section 2 describes the discrepancies that have been observed between prototype and scale model measurements of wave run-up.

Section 3 provides a theoretical background on the subjects of run-up and overtopping.

Section 4 describes prototype wave run-up and overtopping measurements on a rubble mound breakwater and prototype measurements of run-up on a sea dike.

Finally section 5 presents a project description of OPTICREST.

2 DISCREPANCIES BETWEEN PROTOTYPE AND SCALE MODEL WAVE RUN-UP

During the MAST II project 'Full scale dynamic load monitoring of rubble mound breakwaters' (MAS2-CT92-0023), wave run-up was measured on the NW breakwater of the Zeebrugge harbour (Belgium). The prototype measurements were compared with run-up levels obtained from scale model tests. The breakwater was modelled in three different laboratories: Aalborg University, Denmark; Flanders Hydraulics, Belgium; University College Cork, Ireland. The run-up levels observed in the scale model tests fit well with experimental design curves. However, as shown in figure 1 the prototype measurements, were approximately 50% higher (De Rouck et al., 1996). This difference confirms the experience in many harbours that wave overtopping is much higher during exploitation, than was expected during design.

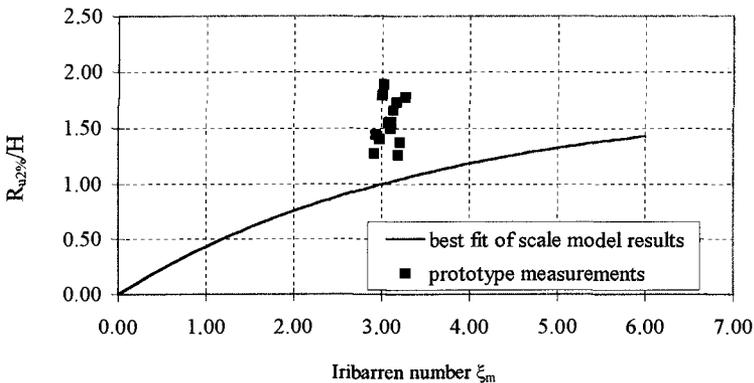


Figure 1. Wave run-up levels with 2% exceedance probability, obtained from prototype and scale model measurements.

All prototype results were obtained during one storm event. Due to the observed wave characteristics and the breakwater geometry, all data are concentrated in an area of Iribarren numbers around 3. Future measurements in a wider range of Iribarren numbers are necessary to confirm this trend for higher full scale run-up levels.

Nowadays, the design of the crest level of sloping coastal structures is based on scale model tests, or on design rules based on such tests. So the aforementioned discrepancy between prototype and scale model run-up levels has very serious consequences: 'the discharge outside will be higher than expected for a given crest level' or 'to limit the discharge to a given value, the crest has to be built higher'. Therefore, further research is necessary.

3 THEORETICAL BACKGROUND

To get a better insight in the problem first a short theoretical background both on wave run-up and wave overtopping is given. So the reader easily can find out which parameters are important with regard to run-up and overtopping.

3.1 Wave run-up

Extensive laboratory testing on wave run-up has been performed by Losada, Van der Meer and others, leading to design rules used today.

Losada and Gimenez - Curto (1981) present a formula to calculate run-up on rough slopes:

$$\frac{R_u}{H} = A \cdot (1 - \exp(-B \cdot \xi)) \quad (1)$$

where R_u = run-up level (m), H = wave height (m), ξ = Iribarren number (-), A and B = experimental coefficients (-) ($A = 1.322$; $B = -0.966$).

This formula is based on tests using **regular** waves. The slopes that were tested, were rip-rap slopes.

The formula for wave run-up on rough slopes, proposed by Van der Meer and Stam (1992), takes the form:

$$\frac{R_{ux\%}}{H_s} = a \cdot \xi_m \quad \text{for } \xi_m \leq 1.5 \quad (2)$$

$$\frac{R_{ux\%}}{H_s} = b \cdot \xi_m^c \quad \text{for } \xi_m \leq 1.5 \quad (3)$$

$$\frac{R_{ux\%}}{H_s} \leq d \quad \text{for permeable structures} \quad (4)$$

where: $R_{ux\%}$ = the run-up level (m) with an exceedance probability of $x\%$, H_s = the significant wave height (m), ξ_m = the surf similarity parameter or Iribarren number (-), based on the mean period, a , b , c and d = experimental coefficients (-) depending on the exceedance probability x .

The coefficients for different probabilities of exceedance are given in table 1.

Probability of exceedance (%)	coefficient a (-)	coefficient b (-)	coefficient c (-)	coefficient d (-)
0.13	1.12	1.34	0.55	2.58
1	1.01	1.24	0.48	2.15
2	0.96	1.17	0.46	1.97
5	0.86	1.05	0.44	1.68
10	0.77	0.94	0.42	1.45
significant	0.72	0.88	0.41	1.35
mean	0.47	0.60	0.34	0.82

Table 1. Coefficients to be used in the formulae (2)-(4) by Van der Meer & Stam (1992) to calculate run-up on rough slopes.

The formula by Van der Meer is based on tests using **irregular** waves. The slopes that were tested, were rip-rap slopes, ranging from 1:2 to 1:6. Wave steepness ranged from 0.004 to 0.06.

Recently a general expression of wave run-up for **irregular** waves has been published (Burcharth, 1998; Van der Meer et al., 1998).

$$\frac{R_{ux\%}}{H_s} = (A\xi + C)\gamma_r \gamma_b \gamma_h \gamma_\beta \quad (5)$$

where

- $R_{ux\%}$ run-up level exceeded by x% of the incident waves
- ξ surf similarity parameter, e.g. ξ_{op}
- A, C coefficients dependent on ξ and x but related to the reference case of a smooth, straight impermeable slope, long-crested head-on waves and Rayleigh distributed wave heights
- γ_r reduction factor for influence of surface roughness; $\gamma_r = 1$ for smooth slopes
- γ_b reduction factor for influence of a berm; $\gamma_b = 1$ for non-bermed profiles
- γ_h reduction factor for influence of shallow water conditions where the wave height distribution deviates from the Rayleigh distribution; $\gamma_h = 1$ for Rayleigh distributed waves
- γ_β factor for influence of angle of incidence β of the waves; $\gamma_\beta = 1$ for head on long-crested waves, i.e. $\beta = 0^\circ$. The influence of directional spreading in short-crested waves is included in γ_β as well

The coefficients A and C, together with estimates of the coefficient of variations for R_u , are given in Table 2. The symbol R_{us} means the significant R_u .

ξ	R_u	ξ -limits	A	C	$\frac{\sigma_{Ru}}{R_u}$
ξ_{op}	$R_{u2\%}$	$\xi_p \leq 2.5$	1.6	0	app. 0.15
		$2.5 < \xi_p < 9$	-0.2	4.5	
	R_{us}	$\xi_p \leq 2.0$	1.35	0	app. 0.1
		$2.0 < \xi_p < 9$	-0.25	3.0	

Table 2. Coefficients in eq (5) for run-up of long-crested irregular waves on smooth impermeable slopes (Burcharth, 1998).

Within the MASTIII-project MAS3-CT97-0116 ‘The optimisation of crest level design of sloping coastal structures through prototype monitoring and modelling’ (OPTICREST) measurements on site will be carried out on the NW rubble mound breakwater in Zeebrugge (rough permeable) and the sea dike in Petten (smooth impermeable). The main difference is the roughness and the void ratio of the armour layer. So only γ_r will be discussed in more detail here. For variations of γ_b , γ_h and γ_β reference is made to De Waal and Van der Meer (1992) and Van der Meer et al. (1998).

The original factor γ_r given in TAW (1974) and in the SPM (1984) has been updated based on experiments including large scale tests with random waves. The new γ_r values taken from de Waal and Van der Meer (1992) are valid for $1 < \xi_{op} < 3-4$. For larger ξ_{op} values the γ_r factor will slowly increase to 1.

Type of slope surface	γ_r
Smooth, concrete, asfalt	1.0
Smooth block revetment	1.0
Grass (3 cm length)	0.90 – 1.0
1 layer of rock, diameter D, ($H_s/D = 1.5 - 3.0$)	0.55 – 0.6
2 or more layers of rock, ($H_s/D = 1.5 - 6.0$)	0.50 – 0.55

Table 3. Surface roughness reduction factor γ_r in eq. (5), valid for $1 < \xi_{op} < 3-4$. (de Waal and van der Meer, 1992)

3.2 Overtopping

The experimental study of overtopping also has led to a number of empirical formulae. Several formulae which relate a dimensionless average discharge to a dimensionless freeboard have been published. The freeboard of a structure is defined as the vertical distance between the Still Water Level (SWL) and the crest of the structure. This type of formulae are called ‘simple regression models’.

Another approach to calculate overtopping discharges is the use of ‘weir models’. These models are based on a theoretical derivation which considers the crest of a structure to be a weir.

It has to be emphasised that the formulae give average overtopping discharges. These values should be used very carefully when thinking about admissible overtopping discharges. The intensity of water hitting a specific location is very much dependent on

the geometry of the structure and the distance from the front of the structure. The maximum intensities might locally be up to two orders of magnitude larger than the average discharge.

Moreover, what is regarded as acceptable conditions is to a large extent a matter of local traditions and individual opinions.

Some background on both models for calculating the average discharge are presented.

3.2.1 Simple regression models

The advantage of simple regression models over weir models is that they are very easy to use. The disadvantage is that they do not fulfil the boundary conditions:

- When the crest becomes very high, the overtopping discharge should be zero.
- When the freeboard is zero, the overtopping discharge should remain finite.

Owen (1980) relates a dimensionless freeboard to a dimensionless discharge by an exponential relationship:

$$Q_* = A \cdot \exp\left(\frac{-B \cdot R_*}{r}\right) \quad (6)$$

The dimensionless variables are defined as:

$$Q_* = \frac{Q}{T_m g H_s} = \frac{Q}{\sqrt{g H_s^3}} \cdot \sqrt{\frac{s}{2\pi}} \quad (7)$$

and

$$R_* = \frac{R_c}{T_m \cdot \sqrt{g H_s}} = \frac{R_c}{H_s} \cdot \sqrt{\frac{s}{2\pi}} \quad (8)$$

where: A and B = experimental coefficients (-), r = coefficient (-) ranging from 0 to 1 to account for the roughness of the slope, Q = mean overtopping discharge per meter crest length (m³/s.m), T_m = mean wave period (s), g = gravitational acceleration (m/s²), s = wave steepness (-), R_c = freeboard (m).

The parameter ranges tested by Owen were: R* ranging from 0.05 to 0.30, Q* ranging from 10⁻⁶ to 10⁻², slope ranging from 1:1 to 1:4, d/H_s (where d = water depth (m)) ranging from 1.5 to 5.5 and H_s/L_{0,mean} (where L_{0,mean} = mean deepwater wave length) ranging from 0.035 to 0.055.

Allsop & Bradbury (1988) propose a different relationship between a dimensionless freeboard and a dimensionless discharge:

$$Q_* = A \cdot F_*^{-B} \quad (9)$$

The dimensionless variables are defined as:

$$Q_* = \frac{Q}{T_m g H_s} = \frac{Q}{\sqrt{g H_s^3}} \cdot \sqrt{\frac{s}{2\pi}} \quad (10)$$

and

$$F_{\star} = \frac{R_c}{H_s} \frac{R_c}{T_m \sqrt{gH_s}} = \frac{R_c^2}{H_s^2} \cdot \sqrt{\frac{s}{2\pi}} \quad (11)$$

where A and B = experimental coefficients (-).

Other formulae are discussed by Burcharth (1998).

A more recent regression model is given by Van der Meer et al. (1998):

$$\frac{q}{\sqrt{gH_s^3}} = \frac{0.06}{\tan \alpha} \cdot \gamma_b \cdot \xi_{op} \cdot \exp\left(-4.7 \cdot \frac{R_c}{H_s} \cdot \frac{1}{\xi_{op} \cdot \gamma_b \gamma_f \gamma_\beta \gamma_v}\right) \quad (12)$$

With as a maximum:

$$\frac{q}{\sqrt{gH_s^3}} \leq 0.2 \cdot \exp\left(-2.3 \cdot \frac{R_c}{H_s} \cdot \frac{1}{\gamma_f \gamma_\beta}\right) \quad (13)$$

where: q = mean overtopping discharge per meter crest length ($m^3/s.m$), α = slope angle (rad), ξ_{op} = the Iribarren number (-) calculated with the peak period and the deepwater wave length, γ_b = a reduction factor (-) to account for the effect of a berm, γ_f = a reduction factor (-) to account for the roughness of the slope, γ_β = a reduction factor (-) to account for the effect of oblique wave attack, γ_v = a reduction factor (-) to account for the effect of a vertical wall.

Full discussion of the influence of a berm, roughness, oblique wave attack, etc... can be found in van der Meer et al. (1998).

3.2.2 Weir models

As an example the first weir model, developed by Kikkawa et al. (1968) is presented. Based on theoretical considerations, the following formula is derived:

$$\frac{Q}{\sqrt{2g} \cdot H_0^{3/2}} = \frac{2}{15} \cdot M \cdot (1 - K_0)^{5/2} \quad (14)$$

Where: H_0 = deep water wave height (m), K_0 = a dimensionless freeboard (-), M = experimental coefficient (-).

The left hand side of equation (14) can be considered as a dimensionless discharge. Thus this equation gives a relationship between a dimensionless discharge and a dimensionless freeboard. It holds the advantage over a simple regression model that it contains more physics and satisfies both boundary conditions:

- When the freeboard equals the highest run-up level, K_0 becomes 1 and the discharge becomes 0.
- When the freeboard is zero, K_0 becomes zero and the discharge remains finite.

4 PROTOTYPE MEASUREMENTS OF RUN-UP AND OVERTOPPING

In order to investigate whether there are systematic differences between run-up levels and overtopping discharges measured in (small) scale models and on full scale, in-situ measurements in Zeebrugge will proceed within OPTICREST. Also in-situ measurements of run-up will be carried out on a smooth sea dike in Petten (The Netherlands). At both locations tide and wind data are available through a nationally funded project.

The main objectives of OPTICREST are:

- Provide designer with improved design rules for the crest level design of sloping coastal structures
- Verify and calibrate scale models for run-up with full scale data
- Calibrate numerical models with full scale data and model test data

The partners of the OPTICREST project are

- University of Gent (Coordinator)
- Flemish Community (Coastal Division; Flanders Hydraulics)
- Aalborg University
- Leichtweiss Institut für Wasserbau
- University College Cork
- Delft Hydraulics
- Rijksinstituut voor Kust en Zeeën
- Universidad Politecnica de Valencia
- Instituto Hidrografico

The project runs from 1 March 1998 till 28 February 2001.

4.1 Zeebrugge (Belgium)

The port of Zeebrugge is situated on the eastern part of the Belgian coastline and is protected by two main breakwaters (Fig. 2). The Zeebrugge breakwater constitutes a conventional rubble mound breakwater with an armour layer of 25 ton grooved cubes.

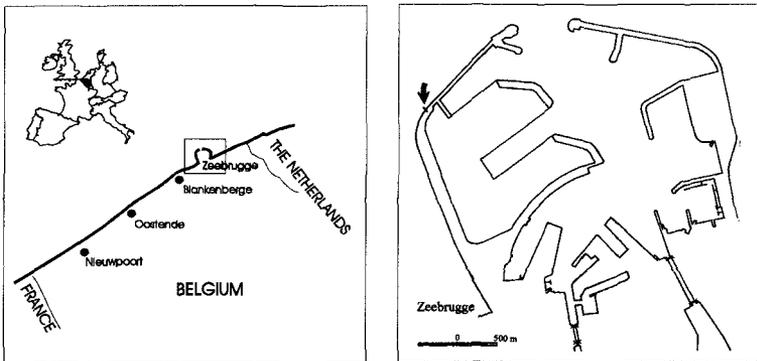


Figure 2. Location of the prototype run-up measurement system on the NW breakwater at the Zeebrugge harbour (Belgium).

Measurement Jetty

Location of sensors (December 1997)

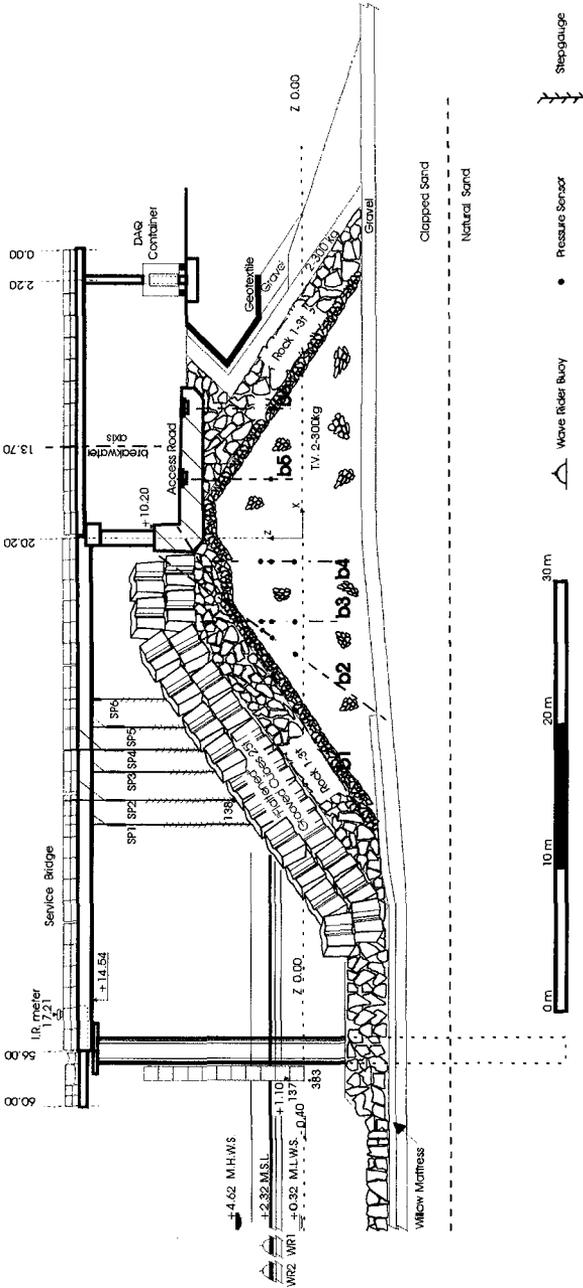


Figure 3. Crosssection of the Zeebrugge rubble mound breakwater with the instrumentation.

A measurement jetty of 60 m length is constructed on the NW breakwater (Fig. 3). It is supported by a steel tube pile at the breakwater toe and by two concrete columns on top of the breakwater.

Run-up is measured using six vertical stepgauges, placed along the breakwater slope. At the bottom, the stepgauges are attached to an armour unit. At the top they are supported by the jetty. Each stepgauge has a number of electrodes, spaced vertically at 20 cm intervals. Each submerged electrode produces an output voltage. The summation of these voltages leads to an output signal that contains the information about the water level with a 20 cm resolution. Using the six stepgauges, this information is available at six positions along the breakwater slope. Specially developed software computes the water surface profile on the slope as the polygon connecting the water surface levels measured by the six stepgauges. The instantaneous run-up level is taken as the intersection of the slope with an extrapolated line through the highest two water surface levels measured at that instant. (Figure 4)

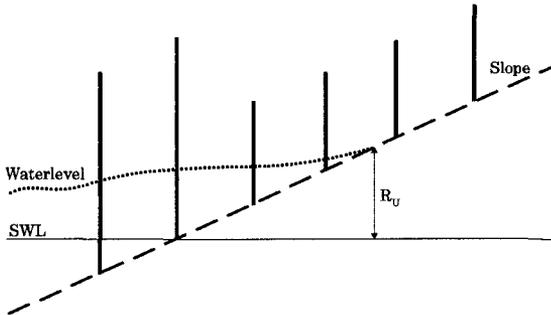


Figure 4. Definition of wave run-up R_u as detected by the stepgauges.

A device to measure overtopping will be installed on the breakwater in autumn 1998. Overtopping will be measured by a screen, catching the overtopping water which will then be collected in a container (Fig. 5). The container will be continuously emptied by a weir. The volume of water in the container, as well as the discharge over the weir will be calculated from the water surface level in the container.

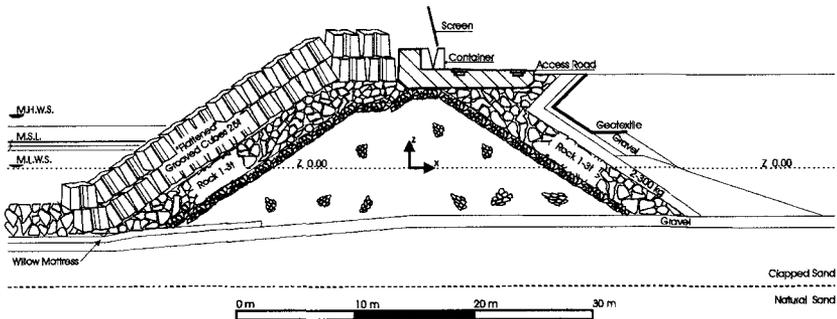


Figure 5. Prototype measurement of overtopping.

Wave data are obtained from two wave rider buoys close to the breakwater, at 165 m and 215 m in front of the breakwater, and from three wave rider buoys located at 5 km, 30 km and 45 km from the structure.

4.2 Petten (The Netherlands)

The Petten sea dike is situated in the northern part of The Netherlands (Fig. 6). The Petten coastal region is very suitable for a measurement campaign as the iso-baths are approximately parallel to the coastline. The bathymetry is characterised by a bar and a dike onshore.

The Petten sea dike consists of a lower slope, made of basalt stones on approximately a 1:3.5 slope. Above the lower slope an asphalt berm (1:13.5) and an asphalt slope (1:2.75) form the higher protection. Figure 7 shows the crosssection of the dike.

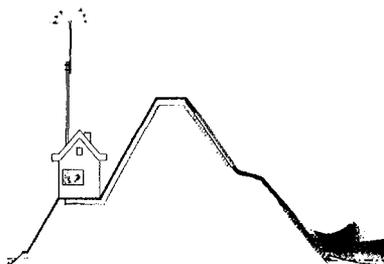
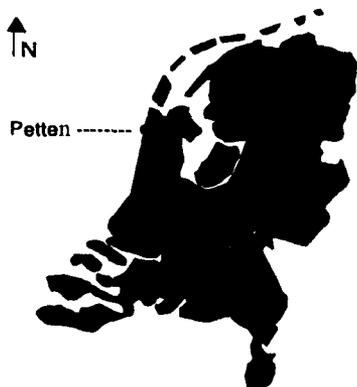


Figure 6. Location of the prototype run-up measurement system on the Petten sea dike (The Netherlands).

Figure 7. Cross section of the Petten sea dike (distorted scale).

Figures 8 and 9 show the bathymetry and the instrumentation.

Data concerning the wave climate are collected by wave rider buoys (Mp2 and Mp4) and by directional wave riders (Mp1 and Mp5). Just in front of the bar, a wave staff and a water level meter (Mp3) measure the incoming waves. In front of the dike the waves are measured by a wave staff and a pressure sensor (Mp6 and Mb6).

The wave run-up is measured, using a staff gauge placed along the slope of the sea dike (Mp7). The staff gauge consists of a number of electrodes spaced at 10 cm intervals. The vertical distance between the electrodes is approximately 3.5 cm

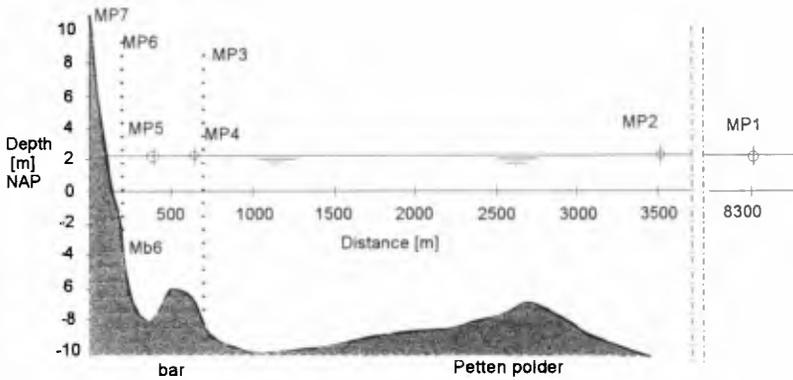


Figure 8. Bathymetry and instrumentation at the Petten sea dike.

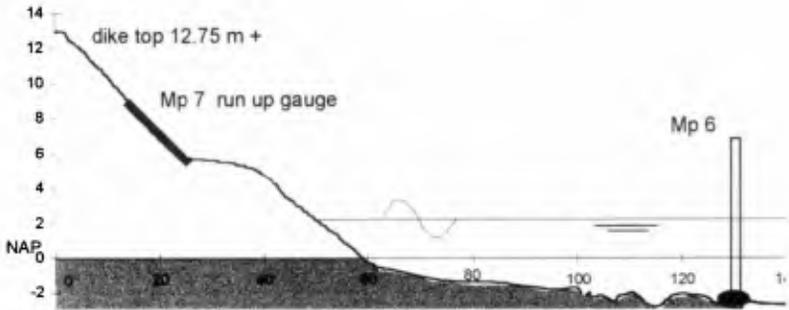


Figure 9. Bathymetry and instrumentation in front of the Petten sea dike

5 FURTHER PROJECT DESCRIPTION

5.1 Laboratory investigations

Both the Zeebrugge and the Petten site will be modelled in scale model tests. Run-up levels and overtopping discharges will be measured. In the scale model tests, both in-situ measured spectra and standard spectra will be used. Special attention will be paid to the influence of the foreshore bathymetry.

Two dimensional and three dimensional tests will be carried out.

5.1.1 Two dimensional testing

Deviations between the scale model results and the prototype measurements will be determined and the influence of different scale effects will be investigated. This may lead to changes in building models for the two sites. The models will then be re-built and the basic tests re-run.

5.1.2 Three dimensional tests

Three dimensional tests will be carried out for models of the Zeebrugge and the Petten site. The scaling will be as close as possible to the scales used in the two dimensional tests.

These tests will attempt to validate the prototype results and continue to investigate the influence of such parameters as wave height, period, water depth, angle of wave attack, directional spreading, currents, foreshore bathymetry and structure geometry.

5.1.3 Link between prototype and laboratory results

One of the conclusions of the MAST II project 'Full scale dynamic load monitoring of rubble mound breakwaters', was that wave run-up is underpredicted by model tests. This may be due to several factors, such as: measuring systems, wind, roughness, randomness of sea conditions, etc... The purpose of the comparison between prototype and laboratory results is to quantify these effects in order to allow better predictions of run-up levels and overtopping quantities from laboratory tests.

5.2 Numerical modelling

It has to be expected that numerical models will play a very important role in future research and design of coastal structures. Therefore it is of paramount importance that these models are as reliable as can be achieved. Up to now, these models have been calibrated against scale model test results only. In the OPTICREST project, they will also be calibrated against full scale data.

5.3 Outlook

Existing design techniques for sloping coastal structures, which include the use of mathematical and physical models have not prevented severe failures. Such failures have disastrous consequences, such as loss of life, loss of business, damage to property, economic ruin, ... Reliable design methodologies are essential.

Full scale measurements of run-up and overtopping will provide better insights in the phenomena and will lead to better mathematical and physical models. This will lead to a better design method of the crest of sloping coastal structures. With our coast being used more than ever before, these results will be very valuable.

6 ACKNOWLEDGEMENTS

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