



Aalborg Universitet

AALBORG UNIVERSITY  
DENMARK

## Wave Run-Up on a Rubble Mound Breakwater

*prototype measurements versus scale model tests*

Rouck, J. De; Troch, P.; Walle, B. Van de; Meer, J. van der; Damme, L. Van; Medina, J. R.; Willems, M.; Frigaard, Peter; Kofoed, Jens Peter

*Published in:*

Proceedings of the 5th Int. Conf. on the Mediteranean Coastal Environment

*Publication date:*

2001

*Document Version*

Early version, also known as pre-print

[Link to publication from Aalborg University](#)

*Citation for published version (APA):*

Rouck, J. D., Troch, P., Walle, B. V. D., Meer, J. V. D., Damme, L. V., Medina, J. R., Willems, M., Frigaard, P., & Kofoed, J. P. (2001). Wave Run-Up on a Rubble Mound Breakwater: prototype measurements versus scale model tests. In E. Özhan (Ed.), *Proceedings of the 5th Int. Conf. on the Mediteranean Coastal Environment: Oct. 2001, MEDCOAST-01, Hammamet, Tunisia* (Vol. 3). National Institute for Marine Science and Technology Coastal Management and Protection Agency.

### General rights

Copyright and moral rights for the publications made accessible in the public portal are retained by the authors and/or other copyright owners and it is a condition of accessing publications that users recognise and abide by the legal requirements associated with these rights.

- Users may download and print one copy of any publication from the public portal for the purpose of private study or research.
- You may not further distribute the material or use it for any profit-making activity or commercial gain
- You may freely distribute the URL identifying the publication in the public portal -

### Take down policy

If you believe that this document breaches copyright please contact us at [vbn@aub.aau.dk](mailto:vbn@aub.aau.dk) providing details, and we will remove access to the work immediately and investigate your claim.

# Wave run-up on a rubble mound breakwater: prototype measurements versus scale model tests

De Rouck J.<sup>1</sup>, Troch P.<sup>1</sup>, Van de Walle B.<sup>1</sup>, van der Meer J.<sup>2</sup>, Van Damme L.<sup>3</sup>, Medina J.R.<sup>4</sup>,  
Willems M.<sup>5</sup>, Frigaard P.<sup>6</sup>

## Abstract

Wave run-up on a rubble mound breakwater is investigated. Prototype measurements are carried out on the Zeebrugge breakwater. Two 2D (1:30) and one 3D (1:40) scale models are tested. Measured prototype storms are simulated and parametric tests are performed. Wave run-up is detected by a novel step gauge allowing to detect wave run-up more accurately. In prototype, wave run-up is measured by two different measuring devices: a so-called 'spiderweb system' and a run-up gauge. Both instruments yield comparable results. A clear difference between prototype measurement and physical modelling results is noticed. Also differences between the results of the different laboratories are seen. Various factors leading to these differences are highlighted.

## 1 INTRODUCTION

Wave run-up is one of the main physical processes which are taken into account in the design of the crest level of sloping coastal structures. The crest level design of these structures is mainly based on physical scale model results. However, prototype measurements have indicated that small scale models may underestimate wave run-up for rubble mound structures (Troch et al. (1996)). Therefore wave run-up has been studied in detail comparing prototype measurements and physical modelling results. Wave run-up is also investigated using numerical modelling.

Detailed research on wave run-up is carried out within the European MAST III OPTICREST project ('The optimisation of crest level design of sloping coastal structures through prototype

---

<sup>1</sup> Ghent University, Department of Civil Engineering, Technologiepark 9, B-9052 Zwijnaarde (Ghent)

<sup>2</sup> Infram, PO Box 81, Patroonsweg 29, NL-3892 AB Zeewolde

<sup>3</sup> Ministry of the Flemish Community, Coastal Division, Vrijhavenstraat 3, B-8400 Oostende

<sup>4</sup> Universidad Politécnica de Valencia, Departamento Transportes / ETSI Caminos, Camino de Vera s/n, E-46022 Valencia

<sup>5</sup> Ministry of the Flemish Community, Flanders Hydraulics, Berchemlei 115, B-2140 Borgerhout

<sup>6</sup> Aalborg University, Department of Civil Engineering, Sohngaardsholmsvej 57, DK-9000 Aalborg

monitoring and modelling' - MAS3-CT97-0116) (De Rouck et al. (2000)). The four main objectives are (1) to provide improved design rules for the crest level of sloping coastal structures, (2) to verify physical scale model data with prototype wave run-up data, (3) to calibrate numerical models with prototype and small scale wave run-up data, and (4) to improve existing wave run-up monitoring devices.

Prototype measurements and scale model tests have been performed on a rubble mound breakwater. The breakwater located in the outer harbour of Zeebrugge has been chosen as prototype measuring site. Prototype measurements have been carried out and the data have been analysed by the Flemish Community (Belgium) and Ghent University (Belgium). A 3D scale model of the Zeebrugge breakwater is tested in Aalborg University (Denmark) and 2D scale model tests have been carried out by Flanders Hydraulics (Belgium) and Universidad Politécnic de Valencia (Spain).

In this paper a summary is given of the main results and conclusions concerning the prototype measurements and the physical model tests at the Zeebrugge breakwater. For comparisons with numerical models reference is made to Troch et al. (2001).

## **PROTOTYPE MEASUREMENTS AT ZEEBRUGGE BREAKWATER**

The outer harbour of Zeebrugge (Belgium) is protected by two rubble mound breakwaters armoured with 25 ton grooved cubes (figure 1) (Troch et al. (1998)). Prototype measurements are carried out on the northern part of the western breakwater. The breakwater core consists of quarry run 2-300 kg and the filter layer is made of 1-3 ton rock. The design conditions are: significant wave height  $H_{s,d} = 6.20$  m, maximum peak period  $T_p = 9$  s, MLWS  $Z+0.32$ , MHWS  $Z+4.62$  and design water level  $Z+6.76$ . The maximum tidal current velocity is about 1.80 m/s at approximately 30 minutes before high tide.

A measuring jetty of 60 m length is constructed on the breakwater. It is supported by a steel tube pile at the breakwater toe and by two concrete columns on top of the breakwater.

Two wave rider buoys, located at a distance of 150 and 215 m from the breakwater slope, are used to measure the wave characteristics in front of the breakwater. The water level at the toe of the breakwater is measured by a pressure sensor at the pile and an infra-red wave height meter. Two different measuring devices are used for the measurement of wave run-up: a "spiderweb system" and a run-up gauge. The "spiderweb system" (SP) is a set of 7 step gauges installed vertically between the armour units and the jetty bridge. At their upper end these are attached to the jetty by means of a heavy spring and at their lower end these are fixed to an armour unit. Each step gauge measures the water surface elevations on the breakwater slope. The wave run-up and wave run-down level is extrapolated from these measurements. A five-part run-up gauge (RU) is mounted on top of the armour units. Wave run-down is measured by the most seaward placed vertical step gauge.

Between 1995 and 2000, 13 storms (with significant wave heights  $H_{mo}$  between 2.40 m and 3.13 m, mean wave periods  $T_{0,1}$  on average 6.24 s, peak periods  $T_p$  around 7.93 s and wind ( $\geq 7$  Beaufort) blowing direction almost perpendicular to the breakwater) have been measured. The value of the wave height  $H_{mo}$  is close to the dimensions of the flattened grooved cubes ( $2.40 \times 2.40 \times 2.00$  m<sup>3</sup>). During all storms wave run-up has been measured by the SP and during the last 9 storms also the RU was operational.

Wave run-up  $Ru$  is defined as the difference between the wave run-up level and  $MWL$ . The 2% exceedence level of the expected wave run-up  $Ru$  is used for comparison. Also other exceedence probabilities  $x$  are considered. The point of time of high water is noted down as  $t_{HW}$ . The  $i^{th}$  hour before and the  $i^{th}$  hour after this point of time  $t_{HW}$  are  $t_{HW-i}$  and  $t_{HW+i}$  respectively.

Only during a period of time of 2 hours at high tide, the mean water level in front of the Zeebrugge breakwater is nearly constant. Because of the changing water level (due to tide) in front of the structure, the length of the time series is important when half a tide cycle (symmetric in time with regard to  $t_{HW}$ ) is analysed as the wave run-up value is calculated relative to a *constant* water level: thirty minutes time series are used.

When time series with a period of time of 2 hours at high tide are analysed in their entirety, mean dimensionless wave run-up values of respectively  $\frac{Ru_{2\%}}{H_{mo}} = 1.76$  (RU data of 9 storms) and 1.75 (SP data of 13 storms) are processed. Both wave run-up measuring devices yield comparable results. Due to the absence of the two upper parts of the five-part run-up gauge, the highest wave run-up levels could not be measured during the three February 1999 storms. However, it was possible to determine the 2% wave run-up level theoretically because wave run-up is Rayleigh distributed. For the mentioned three February 1999 storms, these 'Rayleigh equivalent' wave run-up values (based on  $Ru_s$ ) have been taken into account further on.

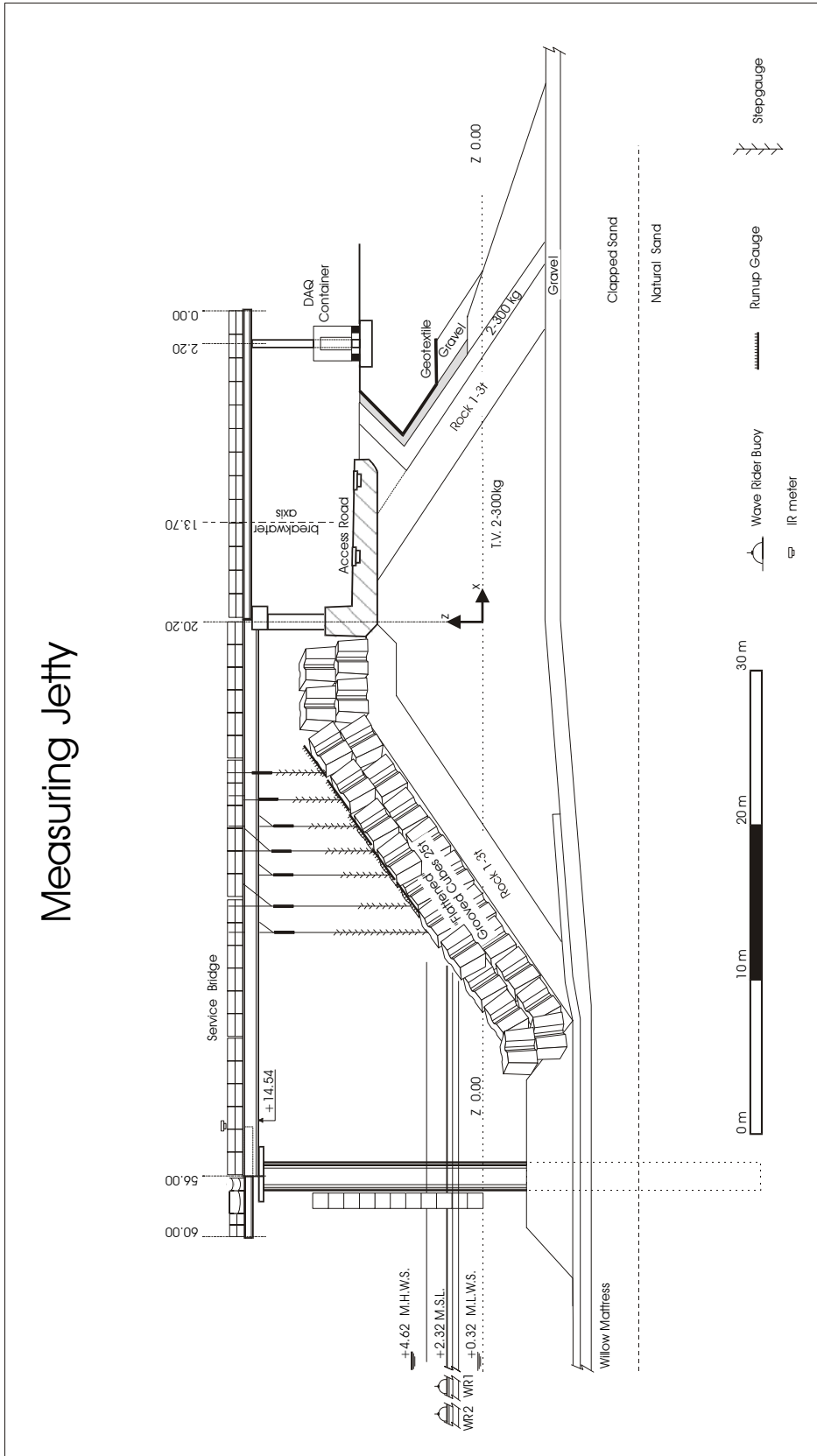
The dimensionless 2% wave run-down value  $\frac{Rd_{2\%}}{H_{mo}}$  (calculated using the same 2 hours time series) equals -0.86.

When 30 minutes time series are used in the analysis of the 2 hours period at high tide (from  $t_{HW-1}$  to  $t_{HW+1}$ ),  $\frac{Ru_{2\%}}{H_{mo}} = 1.77$  for the RU data and  $\frac{Ru_{2\%}}{H_{mo}} = 1.78$  for the SP measurements.

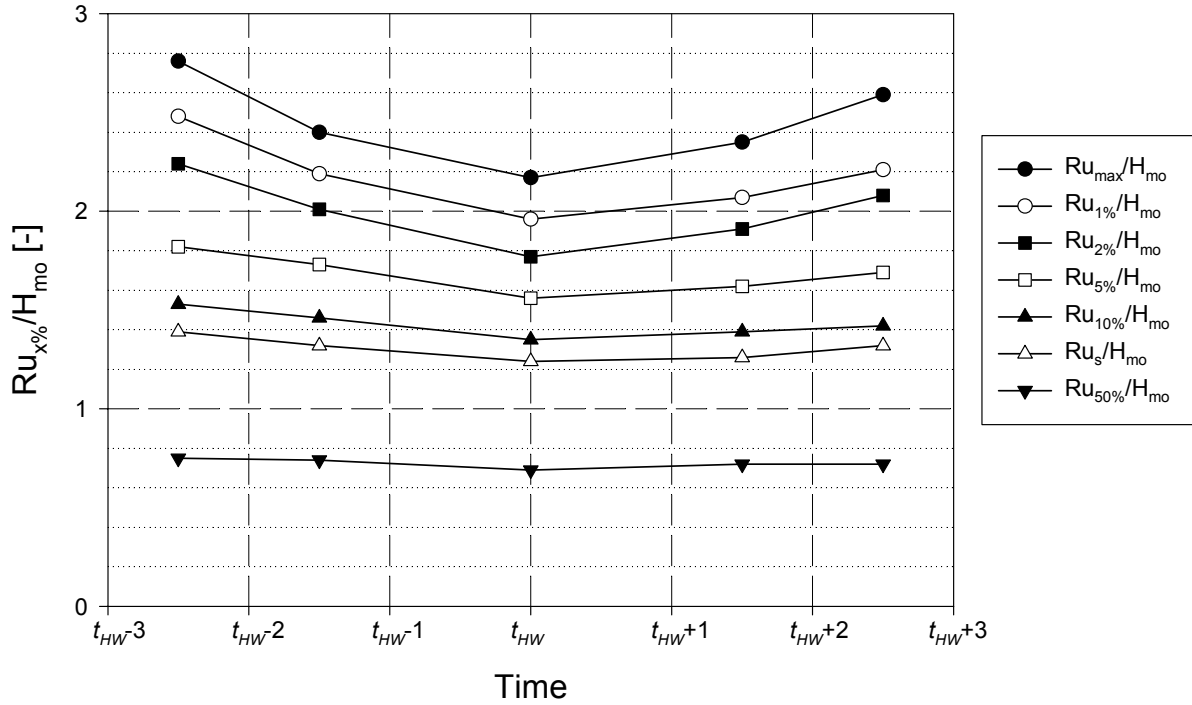
The length of the time series at high water does not affect the results.

**Table 1: Dimensionless prototype wave run-up results (RU, 9 storms, 30 minutes time series).**

	$t_{HW-3} t_{HW-2}$	$t_{HW-2} t_{HW-1}$	$t_{HW-1} t_{HW+1}$	$t_{HW+1} t_{HW+2}$	$t_{HW+2} t_{HW+3}$	van der Meer and Stam (1992)
$\frac{Ru_{max}}{H_{mo}}$	2.76	2.40	2.17	2.35	2.59	2.58
$\frac{Ru_{1\%}}{H_{mo}}$	2.48	2.19	1.96	2.07	2.21	2.15
$\frac{Ru_{2\%}}{H_{mo}}$	2.24	2.01	<b>1.77</b>	1.91	2.08	1.97
$\frac{Ru_{5\%}}{H_{mo}}$	1.82	1.73	1.56	1.62	1.69	1.68
$\frac{Ru_{10\%}}{H_{mo}}$	1.53	1.46	1.35	1.39	1.42	1.45
$\frac{Ru_s}{H_{mo}}$	1.39	1.32	1.24	1.26	1.32	1.35
$\frac{Ru_{50\%}}{H_{mo}}$	0.75	0.74	0.69	0.72	0.72	0.82



**Figure 1: Cross section of the Zeebrugge breakwater with the prototype measuring jetty.**



**Figure 2: Dimensionless prototype wave run-up  $\frac{Ru_{x\%}}{H_{mo}}$  vs. time.**

The results of an analysis of the data of half a tide cycle (using time series of 30 minutes) are mentioned in table 1 and plotted in figure 2. Different values of the exceedence probability  $x$  (1%, 2%, 5%, 10%, 50% as well as the maximum and the significant  $Ru$ ) for dimensionless wave run-up  $\frac{Ru_{x\%}}{H_{mo}}$  have been used.

An interesting aspect from table 1 is that dimensionless wave run-up values increase when the mean water level decreases. The lower the exceedence probability  $x$ , the more the dimensionless wave run-up values increase (figure 2).

Wave run-up levels are slightly higher during rising tide than during receding tide. This may be due to tidal currents and/or the asymmetric tide.

A part of the explanation why dimensionless wave run-up values depend on the water level in front of the structure can be found within the fact that wave heights are lower when lower water depths are considered, so for constant  $Ru$  the ratio  $\frac{Ru}{H}$  becomes larger when  $H$  decreases. However, when looking at the  $Ru$  values themselves, these increase also when water depth decreases. This phenomenon could be explained by the fact that at lower water levels wave run-up takes place at a lower part of the slope. The lower porosity of the armour layer at lower levels (due to the settlement of the armour units during the lifetime of the breakwater (built in 1983)) may cause larger wave run-up. Moreover, at lower water levels, the water depth is less, leading to breaking waves with higher wave run-up.

## PHYSICAL MODEL TESTS ON ZEEBRUGGE BREAKWATER

The Zeebrugge breakwater has been modelled in 3 laboratories: 2D-models (1:30) at Flanders Hydraulics (FH) and at Universidad Politécnica de Valencia (UPV) and a 3D-model (1:40) at Aalborg University (AAU). The influence of wind is investigated in the combined wave flume and wind tunnel facility at UPV. The armour units in the top layer are placed according to the actual position in full scale. The core material has been scaled in such way that the hydraulic gradients are reproduced properly (Burcharth et al.(1999)).

Seven measured storms (of which two cover half a tide cycle) have been reproduced and parametric tests have been carried out. Various measuring devices have been employed to determine the wave run-up: several wire gauges placed at different heights above the surface of the breakwater slope and a novel step gauge, designed and constructed at Ghent University (figure 3). The step gauge is a comb of which each individual needle can be adjusted to the slope of the breakwater. So the distance between the armour units and the gauge is less than 2 mm. In the case of a traditional run-up gauge the distance between the armour units and the gauge can mount to much higher values because of the craggy slope surface.

In table 2, the  $\frac{Ru_{2\%}}{H_{mo}}$  values obtained by small scale model tests are mentioned. In all

laboratories, the same storm sessions have been reproduced. Perpendicular incident waves are generated in the AAU wave tank. All wave run-up values are measured by the novel step gauge. Simulation of the prototype storms give an average value for  $\frac{Ru_{2\%}}{H_{mo}}$  of 1.46 (FH), 1.79

(UPV) and 1.64 (AAU). For AAU the scatter is large: the minimum value is 1.28 and the maximum value is 1.91. The AAU tests comprise spectra which fit the prototype spectra very well (storm November 6 1999 and November 6-7 1999) as well as spectra which are shifted to lower frequencies (which may lead to higher (too high) wave run-up values).



**Figure 3: A novel step gauge for laboratory wave run-up measurements on a breakwater slope (designed and constructed at Ghent University).**

**Table 2: Laboratory results for Zeebrugge breakwater.**

	length of time series	$\frac{Ru_{2\%}}{H_{mo}}$ [-] prototype measurements	$\xi_{om}$ [-]	$\frac{Ru_{2\%}}{H_{mo}}$ [-] FH	$\frac{Ru_{2\%}}{H_{mo}}$ [-] UPV	$\frac{Ru_{2\%}}{H_{mo}}$ [-] AAU
Aug. 28, 1995 am	1h 15min	1.49	3.51	1.48		1.52
Aug. 28, 1995 pm	2h 15min	1.66	3.76	1.42		1.91
Jan. 19, 1998	2h 30min	1.73	3.70	1.53		1.76
Jan. 20, 1998	2h	1.79	3.64	1.40		1.89
Feb. 7, 1999	2h	1.73	3.55	1.39		1.71
Nov. 6, 1999	2h	1.82	3.45	1.44	1.81	1.36
Nov. 6-7, 1999	2h	1.84	3.64	1.57	1.76	1.28

## DISCUSSION OF RESULTS

Wave run-up was preliminary investigated in the MAST II project 'Full scale dynamic load monitoring of rubble mound breakwaters' (MAS2-CT92-0023): a clear difference between prototype measurement results and small scale modelling results was noticed. In the OPTICREST project, wave run-up was studied into detail and the prototype value  $\frac{Ru_{2\%}}{H_{mo}} =$

1.76 is clearly higher than known and generally accepted before the project. The prototype results are first compared to formulae found in literature (Allsop et al. (1985), van der Meer and Stam (1992) and Ahrens and Heimbaugh (1988)) and next to scale model results.

The formula of Losada and Giménez-Curto (1982) is:

$$\frac{Ru_{2\%}}{H_{mo}} = A[1 - \exp(B\xi)] \quad (1)$$

Allsop et al. (1985) reported  $A = 1.52$  and  $B = -0.34$ , based on small scale model tests on a 1:1.5 Antifer cube slope with irregular waves (geometry very alike the Zeebrugge breakwater). Three remarks have to be made: (i) the basic equation (1) results from tests with regular waves; (ii) the results reported by Allsop et al. (1985) relate to structures with highly permeable mounds; (iii) because all different investigations use different parameters, all surf similarity parameters had to be rescaled using the surf similarity parameter (calculated using  $H_{mo}$ ,  $T_{0.1}$  and  $\tan \alpha = \frac{1}{1.3}$  (for the Zeebrugge breakwater)). For the sea state in front of the

Zeebrugge breakwater the relationship  $\frac{T_p}{T_{0.1}} = 1.26$  is used (Troch P. and De Rouck J. (1996)).

The formula of van der Meer and Stam (1992) for rock armoured slopes, attacked by long-crested head-on waves is:

$$\frac{Ru_{x\%}}{H_s} = A\xi_{om} \quad \text{for } 1.0 < \xi_{om} \leq 1.5 \quad (2a)$$



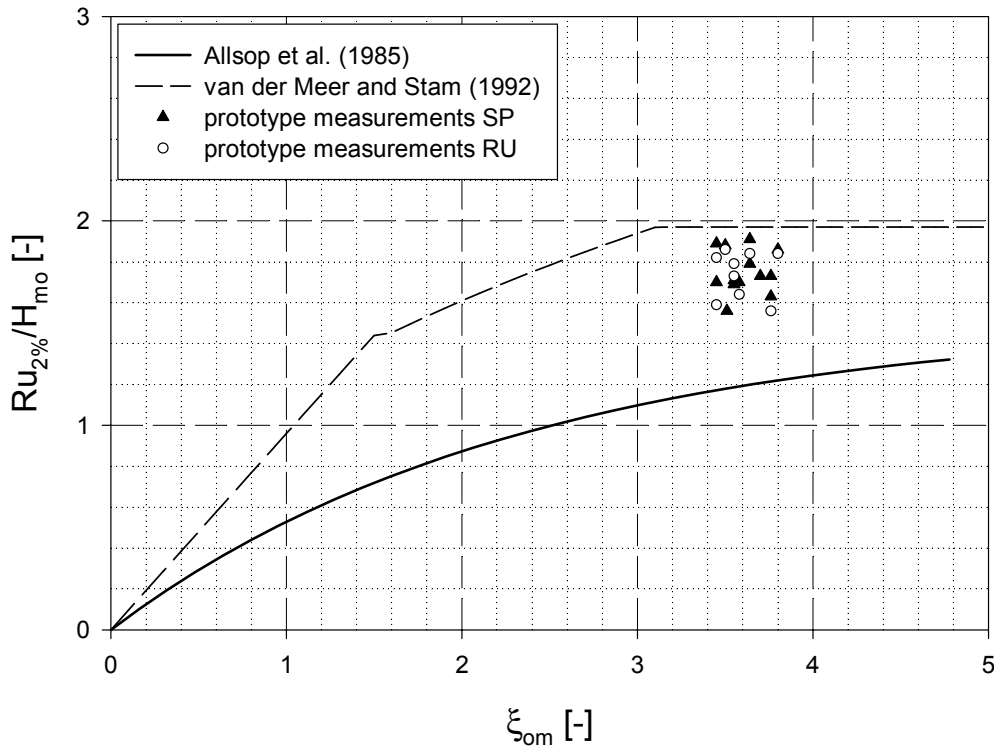
$$\frac{Ru_{x\%}}{H_s} = B\xi_{om}^C \quad \text{for } 1.5 < \xi_{om} \leq \left(\frac{D}{B}\right)^{\frac{1}{C}} \quad (2b)$$

$$\frac{Ru_{x\%}}{H_s} = D \quad \text{for } \left(\frac{D}{B}\right)^{\frac{1}{C}} \leq \xi_{om} < 7.5 \quad (2c)$$

with A, B, C and D depending on the exceedence probability  $x$ . Only formula (2c) is of importance in case of the Zeebrugge breakwater and the respective values of  $\frac{Ru_{x\%}}{H_s}$  are given in table 1.

Equation (2) is valid for relatively deep water in front of the structure where the wave height distribution is close to the Rayleigh distribution. This formula is obtained by tests on rip-rap slopes with rock dimensions which are much smaller than the wave height. In Zeebrugge, wave heights are Rayleigh distributed and the dimensions of the armour units are of the same magnitude as the significant wave height.

Equation (1) and equation (2) (for  $x = 2$ ) are plotted together with the prototype measurement results at high tide (from  $t_{HW}-1$  to  $t_{HW}+1$ ) in figure 4.



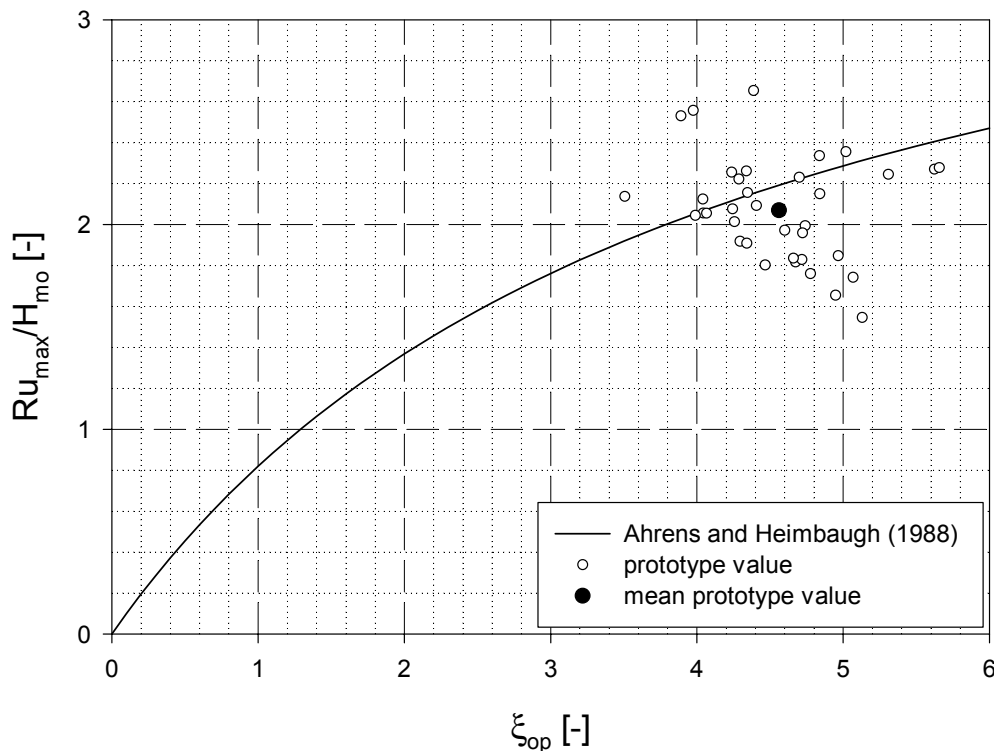
**Figure 4: Comparison between dimensionless wave run-up values from prototype (from  $t_{HW}-1$  to  $t_{HW}+1$ , SP (13 storms) & RU (9 storms), 2 hours time series) and from literature.**

For the prototype value  $\xi_{om} = 3.59$ , equation (1) yields  $\frac{Ru_{2\%}}{H_{mo}} = 1.19$  which is a much lower value than the prototype values. Equation (2) yields  $\frac{Ru_{2\%}}{H_{mo}} = 1.97$  for the average prototype value  $\xi_{om} = 3.59$ . Hence, eq. (2) predicts a slightly higher value than the prototype results. Equation (2) is also compared to the prototype measurement results at the Zeebrugge site for other values of  $x$ . From table 1 it is seen that equation (2) fits the prototype measurements very well during the period from  $t_{HW-2}$  to  $t_{HW-1}$ . During the period of two hours at high tide (from  $t_{HW-1}$  to  $t_{HW+1}$ ), eq. (2) yields higher values than the prototype values.

Ahrens and Heimbaugh (1988) propose another formula:

$$\frac{Ru_{max}}{H_{mo}} = \frac{a\xi}{1 + b\xi} \quad (3)$$

Using the standard surf parameter  $\xi_{op}$  (calculated using  $T_p$  instead of  $T_{0,1}$ ), the run-up coefficients  $a$  and  $b$  equal respectively 1.022 and 0.247. Figure 5 shows the comparison of equation (3) to the maximum measured wave run-up on site. A good agreement is seen, nonetheless equation (3) is also based on tests on rip rap protected slopes.



**Figure 5: Comparison of prototype data to formula (3) ( $t_{HW-1}$   $t_{HW+1}$ , RU, 9 storms, 30 minutes time series).**

From the graph in figure 4, it can be concluded that equation (1) yields a clear underestimation of the prototype wave run-up values. It seems that wave run-up on a rubble mound breakwater armoured with grooved cubes is closer to the run-up given by the

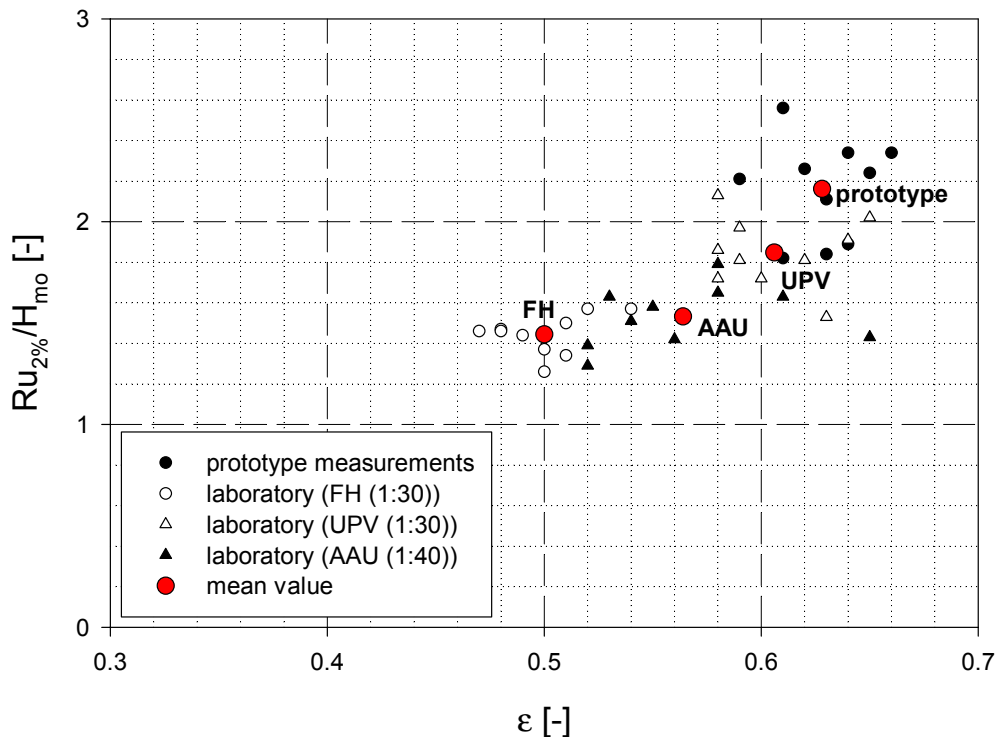
formulae for rip rap slopes as investigated by van der Meer and Stam (1992) (equation (2)) and Ahrens and Heimbaugh (1985) (equation (3)).

Differences between small scale model test results on the one hand and between small scale modelling and prototype measurement results on the other hand are noticed (figure 6). Figure

7 shows the dependency of  $\frac{Ru_{2\%}}{H_{mo}}$  on the MWL. One also notices that prototype results and

AAU results have the same trend: dimensionless 2% wave run-up values increase with decreasing water level. However AAU results are clearly lower than prototype results. UPV results equal prototype results at high water, but diverge from prototype results when the water level becomes lower. A slight increase in dimensionless 2% wave run-up is noticed in the UPV results when the water level is decreasing. The results of FH are almost the same as the AAU results at high water, but remain almost constant when the water level changes. The difference between the result of all laboratories and prototype result becomes smaller and smaller when higher exceedence probabilities  $x$  are considered. Again, AAU results confirm the trend noticed in prototype (dimensionless wave run-up increases with decreasing water level), but the laboratory  $\frac{Ru_{5\%}}{H_{mo}}$  values are smaller than the prototype values. At high tide the

UPV  $\frac{Ru_{5\%}}{H_{mo}}$  values have the same order of magnitude of prototype results, but remain almost constant when the water level changes. At high tide, FH results are slightly higher than AAU results and decrease with decreasing water level. All results become very similar when  $\frac{Ru_{10\%}}{H_{mo}}$  values are considered: at high tide all values have almost the same value.

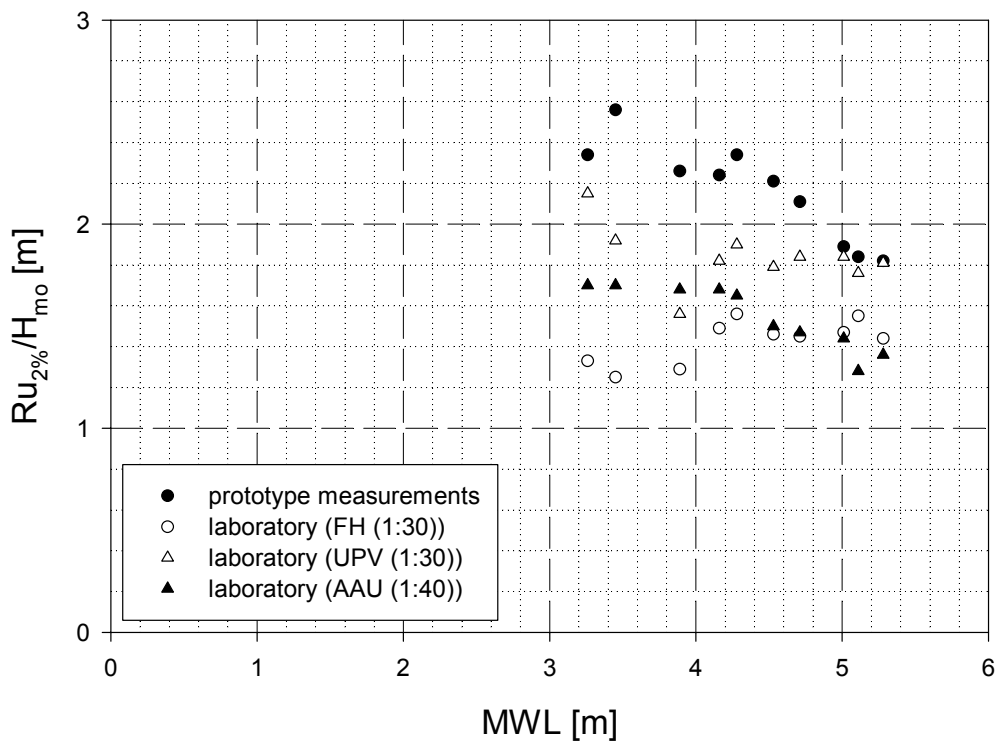


**Figure 6: Comparison between prototype measurements and small scale model test results (cf. data in table 5).**

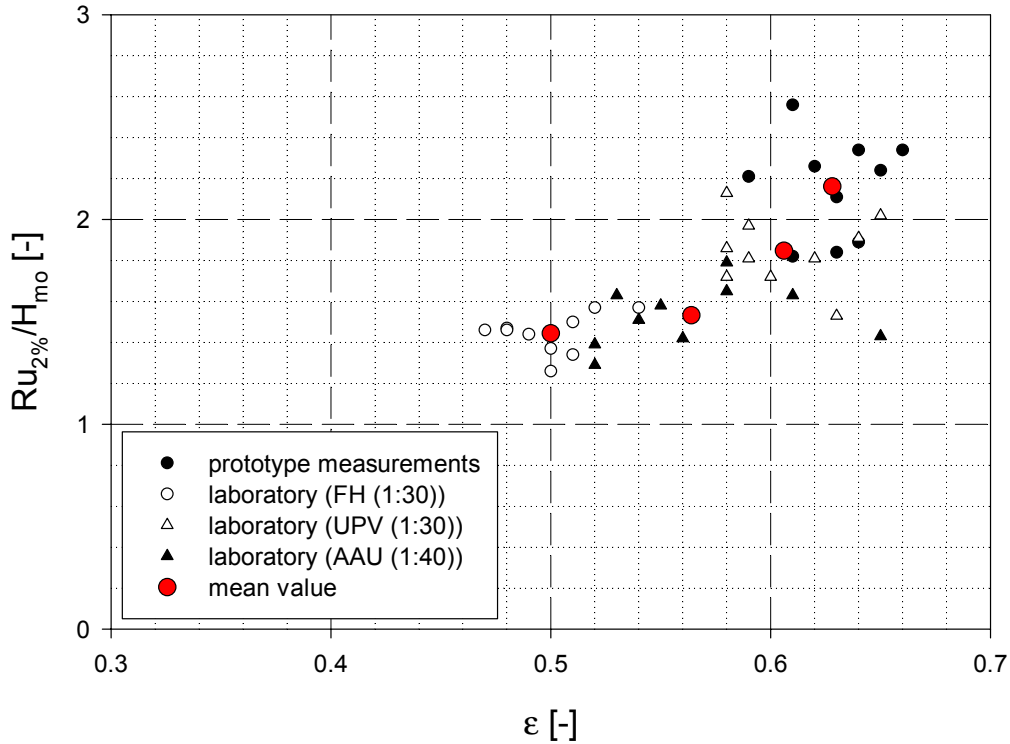
In the simulation of the measured storms, much attention is paid to reproduce the storms as accurate as possible (parameters  $H_{mo}$  and  $T_{0,1}$ ). Nonetheless spectra fit very well, differences in the spectral width parameter  $\varepsilon$  and wave height distributions produced in different laboratories are noticed. Numerous AAU model tests, performed with the same target spectrum, showed that  $\frac{Ru_{2\%}}{H_{mo}}$  is very dependent on the spectral width  $\varepsilon$  which is defined

by  $\varepsilon = \sqrt{\frac{m_0 m_2}{m_1^2} - 1}$ . The same is noticed when all prototype and laboratory  $\frac{Ru_{2\%}}{H_{mo}}$  values are

plotted vs.  $\varepsilon$  on the same graph: the dimensionless 2% wave run-up increases with increasing spectral width parameter value (figure 8). Reproducing prototype spectra in the laboratory only by tuning  $H_{mo}$  and  $T_{0,1}$  to the prototype value is insufficient. Waves, produced in the laboratory, are only defined by their amplitude spectra parameters  $H_{mo}$  and  $T_p$  (or  $T_{0,1}$ ) which is not a complete representation of the kinematics of the waves.



**Figure 7: Comparison prototype measurements (RU) and small scale model test results (Nov. 6 & Nov. 6-7, 1999).**



**Figure 8:**  $\frac{Ru_{2\%}}{H_{mo}}$  versus spectral width parameter  $\epsilon$  (Nov. 6, 1999 & Nov. 6-7, 1999).

At AAU the same target spectrum has been reproduced several times. A quite large scatter is observed in the obtained  $\frac{Ru_{2\%}}{H_{mo}}$  results. The spectral shape (and more specific the spectral width) seems to be of big importance: a small variation of the  $\epsilon$  parameter has a big effect on the  $\frac{Ru_{2\%}}{H_{mo}}$  value.

Finally, it can be mentioned that during the project also following findings are obtained:

In AAU tests it is found that  $\frac{Ru_{2\%}}{H_{mo}}$  increases slightly with increasing tidal current. On site the tidal current is maximum at thigh tide. So this phenomenon has to be taken into account when comparing prototype with laboratory test: especially at the top of high tide when current velocity is maximum. At mean tide, tidal currents are negligible.

The parametric study has shown that wind has only a little influence on wave run-up.

Obliqueness of waves has an influence as well: dimensionless wave run-up decreases when the mean incident wave angle increases.

Laboratory investigation also indicates that the pattern of the armour units and the porosity of the armour layer has a very big influence on the results: values of dimensionless wave run-up values increase with 30% when the porosity of the armour layer decreases !

Although suitable for dikes, the 2% wave run-up level (equivalent to an overtopping discharge of 1 l/m/s) cannot be considered as the key parameter to design the crest level of a rubble mound breakwater. However, wave run-up levels can to some extent be linked to wave overtopping discharges in order to define a crest level height based on an agreed and allowable wave overtopping discharge. The overtopping discharge should be the criterion to determine the crest level of a rubble mound breakwater.

For design purposes it is advised

- to take into account an extra safety when relying on wave run-up levels on permeable slopes in small scale model tests results
- to repeat model tests to study the repeatability of the results
- to use a step gauge for detection of wave run-up

## CONCLUSIONS

The prototype measuring system at the Zeebrugge breakwater is presented and described. It is capable of measuring the wave climate in front of the breakwater and wave run-up on the breakwater slope.

Based on the synthesis of measurements on the Zeebrugge rubble mound breakwater the following conclusions are made:

- The mean prototype dimensionless 2% wave run-up value equals 1.76 ( $\xi_{om} = 3.59$  and valid for mean water level  $z + 5.09$  and for  $H_{mo} \cong D_{n50}$ ) and increases when the water level decreases. The mean prototype dimensionless 2% wave run-down value equals -0.86 which is approximately 50% of wave run-up.

Prototype  $\frac{Ru_{2\%}}{H_{mo}}$  is of the same order of magnitude as found in laboratory tests for rip rap slopes.

- A clear difference between prototype measurements and scale model test results is observed for the Zeebrugge rubble mound breakwater. The difference is the largest at lower water levels. Various factors leading to the difference between prototype measurement and small scale model test results have been highlighted. These reasons can be summarised:
  - model effects (such as the imperfect modelling of porosity and permeability of armour units and core material, no wind is applied in the models, no currents in the model, imperfect modelling of the sea bed topography, imperfect modelling of target spectrum, limitations of some wave generators (stroke of the paddle),...)
  - scale effects (these effects are important for thin water tongues on a rough hard surface and for porous flow).
- Instead of wave run-up allowable wave overtopping should be considered as design parameter for the crest level of rubble mound breakwaters.

## ACKNOWLEDGEMENT

These results are the outcome of the European Community funded MAST III project 'OPTICREST' ('The optimisation of crest level design of sloping coastal structures through prototype monitoring and modelling' - MAS3-CT97-0116). The financial support of the European Community is very much acknowledged.

The project website is <http://awww.rug.ac.be/opticrest/index.shtml>.

## REFERENCES

Ahrens J.P., Heimbaugh M.S., Irregular wave runup on riprap revetments, Journal of Waterway, Port, Coastal, and Ocean Engineering, Vol. 114, No. 4, July, 1988.

Allsop N.W.H., Hawkes P.J., Jackson F.A., Franco L., Wave run-up on steep slopes - model tests under random waves, Report SR2, 1985.

Burcharth H. F., Liu Z. and Troch P., 1999, Scaling of core material in rubble mound breakwater model tests. Proceedings of the 5th International Conference on Coastal and Port Engineering in Developing Countries (COPEDEC V), Cape Town, South Africa, April 1999.

De Rouck J., Troch P., Van de Walle B., Van Damme L., Bal J., The optimisation of crest level design of sloping coastal structures through prototype monitoring and modelling, EurOCEAN 2000 Conference, August-September 2000, Hamburg (Germany), 2000.

Frigaard P., Jensen M.S., Task 3.4: Zeebrugge model: (1) Wave run-up under simulated prototype storms (II) and (2) The influence on wave run-up introducing a current, final report MAST 3, 2001.

Losada M.A., Gimenez-Curto L.A., Flow characteristics on rough, permeable slopes under wave action, coastal Engineering, Vol. 4, pp. 187-206, 1981.

Troch P., Experimental study en numerical modelling of wave interaction with rubble mound breakwaters, PhD thesis (in Dutch), 2000.

Troch P., De Rouck J., Detailed Scientific Report of MAST 2 project: Full scale dynamic load monitoring of rubble mound breakwaters, Ghent University, 1996.

Troch P., De Rouck J., Van Damme L., Instrumentation and prototype measurements at the Zeebrugge rubble mound breakwater, Coastal Engineering 35, pp. 141-166, 1998.

Troch P., De Somer M., De Rouck J., Van Damme L., Vermeir D., Martens J.P., Van Hove C., Full scale measurements of wave attenuation inside a rubble mound breakwater, Proc. 25<sup>th</sup> I.C.C.E., 2-6 September 1996, Orlando (USA), pp. 1916-1929, 1996.

van der Meer J.W., Stam C.-J.M., Wave run-up on smooth and rock slopes of coastal structures, Journal of Waterway, Port, Coastal and Ocean Engineering, Vol. 118, No. 5, 1992.

Willems M., Kofoed J.P., Task 3.3: Laboratory investigations: two dimensional testing, final report MAST 3, 2001.