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prototype versus scale model results

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Wave run-up on sloping coastal structures: prototype measurements versus scale model tests

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

Prototype measurements and physical model tests have been performed on the Petten Sea-defence ('Pettemer Zeewering') in the Netherlands and at the Zeebrugge breakwater in Belgium. Prototype measurements have been performed and analysed by Rijkswaterstaat (RIKZ - the Netherlands) for the Petten Sea-defence and the Flemish Community (Belgium) and Ghent University (Belgium) for the Zeebrugge breakwater.

For the Pettemer sea defence, two-dimensional physical model tests have been performed by WL I Delft Hydraulics (the Netherlands) and three-dimensional physical model tests have been performed by University College Cork (Ireland). A 3D model of the Zeebrugge breakwater is tested in Aalborg University (Denmark) and 2D model tests have been carried out by Flanders Hydraulics (Belgium) and Universidad Politécnica de Valencia (Spain).

In this paper a summary is given of the main conclusions concerning the prototype measurements and the two-dimensional physical model tests at both structures. For comparisons with numerical models reference is made to Van Gent and Doorn (2001) (with regard to the Petten Sea-defence) and Troch (2000) (with regard to the Zeebrugge breakwater).

PROTOTYPE MEASUREMENTS AT PETTEN SEA-DEFENCE

Figure 1 shows the foreshore perpendicular to the dike. The depth-contours in prototype are rather parallel to the coast while the mean angle of wave attack was nearly perpendicular. For the 2D model test and 3D model tests the most relevant instrumentation concerns wave buoys/capacitance wires at locations MP3, MP5 and MP6, at respectively 635 m, 300 m and 130 m seaward of the crest of the dike. The dike itself consists of a 1:4.5 slope below the berm with a slope of 1:20 (between NAP+5 m and NAP+5.7 m) and a 1:3 slope above the berm (figure 3). On this upper slope a wave run-up gauge is placed. Figure 2 shows that the measured wave run-up levels ($z_{2\%}$, relative to NAP) correlates strongly with the measured water levels and wave heights. The storms that were reproduced in the 2D model tests are summarised in table 1.

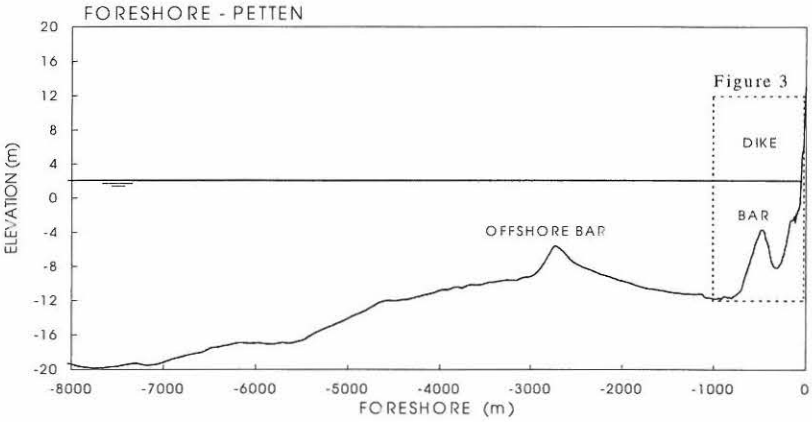

Figure 1: Measured foreshore perpendicular to the Petten Sea-defence.

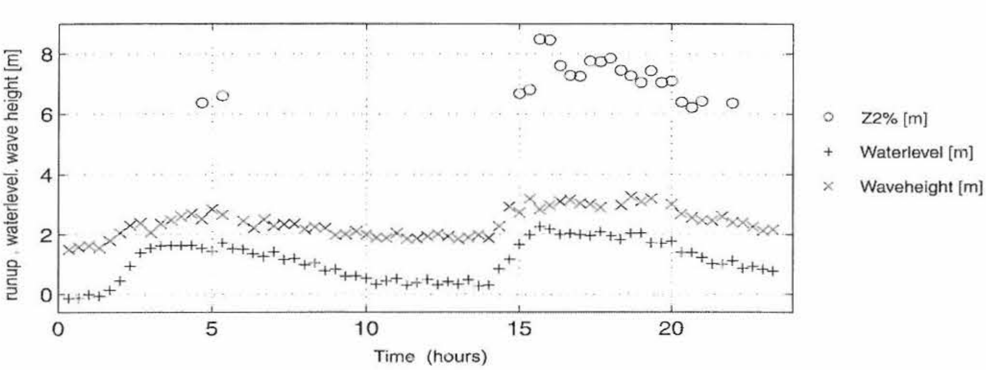

Figure 2: Example of wave run-up measured by run-up gauge at Petten dike; the water levels and wave heights are measured at 635 (MP3) and 130 m (MP6) from the crest of the dike, respectively.

Table 1: Measured storms and wave run-up levels $z_{2\%}$ in prototype (Petten dike).

Measured storms and wave run-up levels in prototype (data from Rijkswaterstaat-RIKZ).												
No	Date	*MWL* (NAP)	$H_{s\text{-}T}$ (MP3)	$T_{-1,0}$ (MP3)	T_p (MP3)	$H_{s\text{-}T}$ (MP5)	$T_{-1,0}$ (MP5)	T_p (MP5)	$H_{s\text{-}T}$ (MP6)	$T_{-1,0}$ (MP6)	T_p (MP6)	$z_{2\%}$ (NAP)
1.01	1-1-1995	2.10	4.24	8.9	11.1	2.61	8.7	11.1	2.94	9.2	12.5	8.3
1.02	1-1-1995	2.01	4.24	8.6	11.1	2.65	8.7	11.1	2.81	9.1	12.5	7.6
1.03	2-1-1995	2.18	3.84	10.2	16.7	2.61	9.9	7.1	2.99	10.8	20.0	8.7
1.04	2-1-1995	1.64	4.24	10.4	16.7	2.39	10.1	16.7	2.64	10.8	12.5	6.9
1.05	2-1-1995	1.60	3.08	9.8	14.3	2.37	9.6	14.3	2.60	9.8	14.3	6.4
1.06	10-1-1995	2.00	3.70	8.8	10.0	2.66	9.0	11.1	2.78	9.4	9.1	7.7
These wave conditions are based on analysis of signals of total waves (index T), including reflected waves, using the energy between the frequencies 0.03 and 0.3 Hz (Δf = 0.01 Hz). NAP: Dutch vertical reference level												

PHYSICAL MODEL TESTS ON PETTEN SEA-DEFENCE

Two-dimensional physical model tests were performed and analysed by WL I Delft Hydraulics (Van Gent, 1999, 2000). Figure 3 shows the last 1000 m of the foreshore as it was modelled in the flume (scale 1:40), while figure 4 shows the structure with the step-gauge to measure wave run-up at the slope above the berm. Three-dimensional physical model tests were performed and analysed by University College Cork. In Murphy (2001) the description of the measurements (scale 1:40) and its analysis are given. The foreshore was similar to the one shown in figure 3, but now the seaward slope of the bar was 1:10 and not 1:30. This was done because of the limited length of the basin (25 m). The dike was positioned over the full width (18 m) of the basin at about 24 m from the wave paddles. Perpendicular to the dike 4 groins were modelled. Wave run-up was measured using a step gauge at the slope above the berm (similar to figure 4).

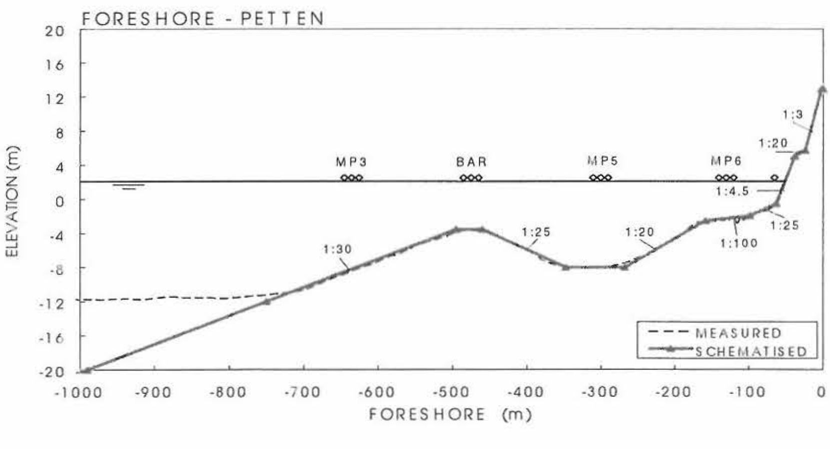

Figure 3: Foreshore as schematised for the model tests.

Since only the most landward bar could be modelled in the tests, the spectral shapes at the corresponding position of the wave board in the prototype situation were affected by wave breaking on the offshore bar. Therefore, for the tests where measured storms were modelled

also the measured wave energy spectra were used, instead of standard spectral shapes such as Pierson-Moskowitz spectra or JONSWAP-spectra. The waves, approximately 1000 waves per wave condition, were generated such that at the location MP3 the wave energy spectra were similar to those measured in prototype.
Because the seaward slope of the bar was 1:10 in the 3D tests, while in prototype a slope close to 1:30 was present, the depth at the position corresponding to MP3 was larger (approximately NAP-15 m compared to NAP-8.2 m in prototype). The spectra measured in prototype at MP3 were reproduced very accurately.

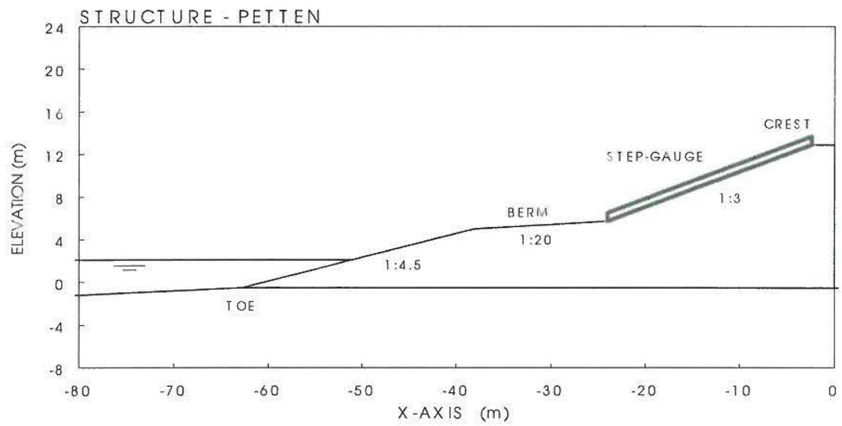

Figure 4: Structure as schematised for the model tests.

At several positions on the foreshore wave conditions were measured in the model tests; at deep water, at the crest of the bar, at the toe of the structure and at three positions where wave conditions have been measured in the prototype situation: MP3, MP5 and MP6. Figure 5 shows an example of the evolution of wave heights over the foreshore while figure 6 shows an example of the evolution of wave energy spectra over the foreshore.

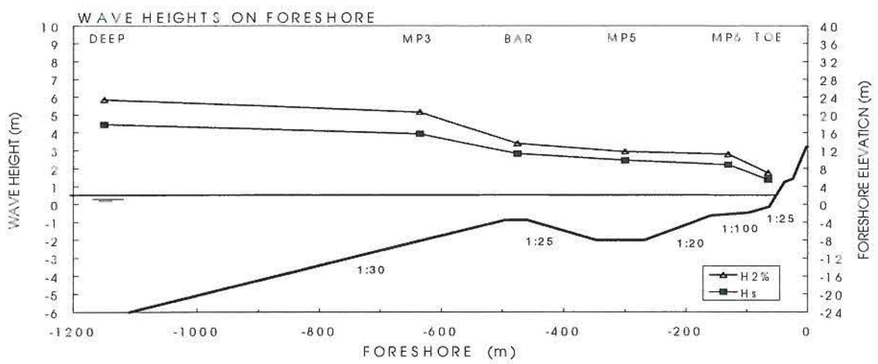

Figure 5: Example of evolution of wave heights over the foreshore (model tests).

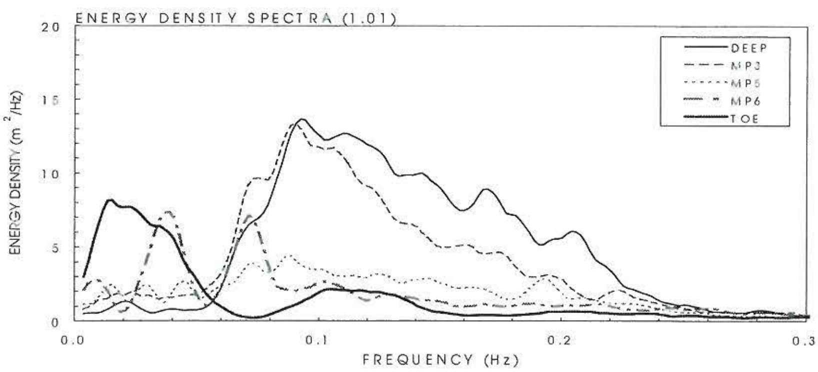

Figure 6: Example of evolution of wave energy spectra over the foreshore (model tests).

Table 2 shows the comparison of the measured wave heights at three locations (MP3, MP5 and MP6) in prototype and in the 2D tests. Table 3 shows the comparison of the measured wave heights at two locations (MP3 and MP6) in prototype and in the 3D tests. The average difference between the measured significant wave heights ($H_s = H_{1/3}$) in prototype and in the 2D model tests is at MP3 1.5 %, at MP5 3.4% and at MP6 8.8%. The average difference between the measured significant wave heights ($H_s=H_{1/3}$) in prototype and in the model tests is at MP3 1.6 % and at MP6 5.6 %. These differences are due to the influence of many factors such as a slightly different foreshore during the actual storms than used in the model tests, 3D effects, effects of wind, schematisation-effects, slightly different data acquisition and data analysis procedures and scale effects. Nevertheless, the observed differences are considered acceptable to further investigate wave run-up.

In prototype thin water layers (between 0.02 m and 0.1 m) were also recorded as wave run-up while in the model tests the step-gauge could not record water layers thinner than 0.1 m (prototype scale). Therefore, comparison between the wave run-up levels measured in prototype (indicated by 'P' in table 2), including thin water layers, and the step-gauge in the model tests (indicated by 'M1' in table 2), not including thin water layers, is not straightforward. However, linear extrapolations based on the measured wave run-up levels with a minimum water layer of 0.1 m (step-gauge) and the measured wave run-up levels with a minimum water layer of 0.2 m (wave-gauge along the slope), yields estimates of wave run-up levels including thin water layers. These levels are indicated by 'M2' in table 2. These 'M2'-levels are used for comparison with prototype measurements.

Table 2: Comparison between prototype measurements and 2D model tests.

Measured storms and wave run-up levels in prototype and physical model tests.														
	MWL (MP3)		$H_{s\text{-}T}$ (MP3)		$H_{s\text{-}T}$ (MP5)		$H_{s\text{-}T}$ (MP6)		$z_{2\%}$ (NAP)			$Ru_{2\%}/H_{s\text{-}T\text{-}MP6}$ *differences*		
Test	*P*	*M*	*P*	*M*	*P*	*M*	*P*	*M*	*P*	*M1*	*M2*	*P*	*M2*	%
1.01	2.10	2.14	4.24	4.29	2.61	2.69	2.94	2.62	8.3	6.8	7.5	2.12	2.05	-3.3
1.02	2.01	2.01	4.24	4.13	2.65	2.68	2.81	2.56	7.6	6.9	7.4	1.99	2.09	5.1
1.03	2.18	2.21	3.84	3.83	2.61	2.77	2.99	2.69	8.7	7.5	8.4	2.17	2.30	6.0
1.04	1.64	1.62	4.24	4.38	2.39	2.58	2.64	2.53	6.9	6.9	7.1	1.99	2.15	7.9
1.05	1.60	1.59	3.08	3.08	2.37	2.39	2.60	2.30	6.4	5.8	5.8	1.86	1.81	-3.0
1.06	2.00	2.02	3.70	3.76	2.66	2.70	2.78	2.58	7.7	6.8	7.3	2.04	2.04	0.0
P = 'Prototype'; M = 'Model tests'; M1= 'step-gauge result'; M2= 'extrapolated to zero water layer'.														

Table 3: Comparison between prototype measurements and 3D model tests.

Measured storms and wave run-up levels in prototype and physical model tests.										
	MWL (MP3)	$H_{s\text{-}T}$ (MP3)		$H_{s\text{-}T}$ (MP6)		$z_{2\%}$ (NAP)		$Ru_{2\%}/H_{s\text{-}T\text{-}MP6}$ *differences*		
Test	*P*	*P*	*M*	*P*	*M*	*P*	*M*	*P*	*M*	%
1.01	2.10	4.24	4.10	2.94	2.81	8.3	7.61	2.12	1.96	-7.5
1.02	2.01	4.24	4.32	2.81	2.85	7.6	7.33	1.99	1.88	-5.5
1.03	2.18	3.84	3.87	2.99	3.02	8.7	9.23	2.17	2.33	7.5
1.04	1.64	4.24	4.30	2.64	3.20	6.9	8.55	1.99	2.19	10.1
1.05	1.60	3.08	3.14	2.60	2.70	6.4	7.23	1.86	2.08	11.7
1.06	2.00	3.70	3.70	2.78	2.83	7.7	7.52	2.04	1.95	-4.4
P = 'Prototype'; M = 'Model tests'.										

The comparison is also made for the non-dimensional wave run-up level, where the wave run-up level is the height above the *mean* water level (*MWL*) and the wave heights are the total significant wave heights ($H_s = H_{1/3}$) measured at MP6. Although wave run-up levels are normally defined as the height above the *still* water level (*SWL*), the *mean* water level has been used for this comparison because for the prototype circumstances the *mean* water level is available, unlike the *still* water level. The differences in percentage are listed in the last column of table 2 and table 3. The average of the difference between prototype and 2D model test values (absolute values) is 3.9 %, whereas the average of the difference between prototype and 3D model test values (absolute values) amounts 7.8%. Figure 7 and figure 8 show the comparison between these wave run-up levels measured in prototype and in the 2D and 3D model tests. Although these differences can also be caused by many factors such as schematisation and scale effects, related to for instance the roughness of the slope and the effects of wind on wave run-up, the agreement is very good.

In addition to the comparison with prototype storms a parameter analysis was performed in the flume where wave heights, wave periods, water levels and wave energy spectra were varied. For these results reference is made to Van Gent (1999, 2000).

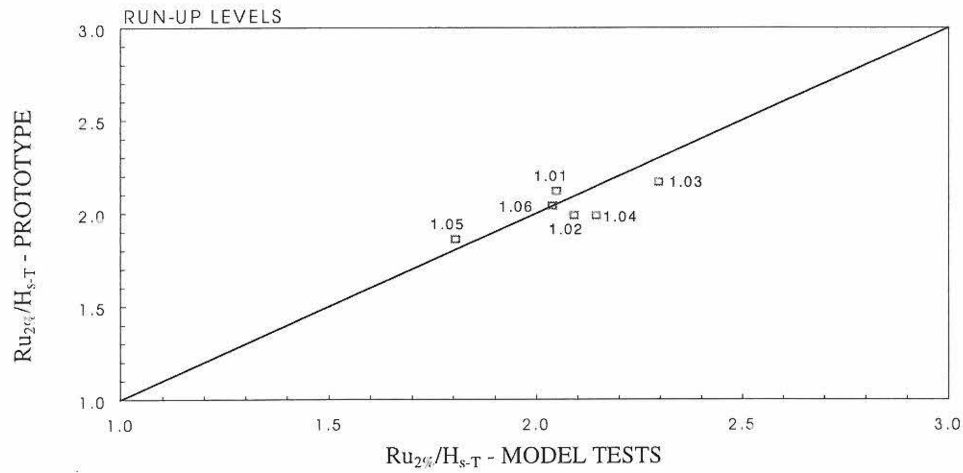

Figure 7: Comparison between wave run-up $Ru_{2\%}$ (normalised by the total significant wave height $H_{s\text{-}T}$) measured in 6 prototype storms and obtained from 2D model tests.

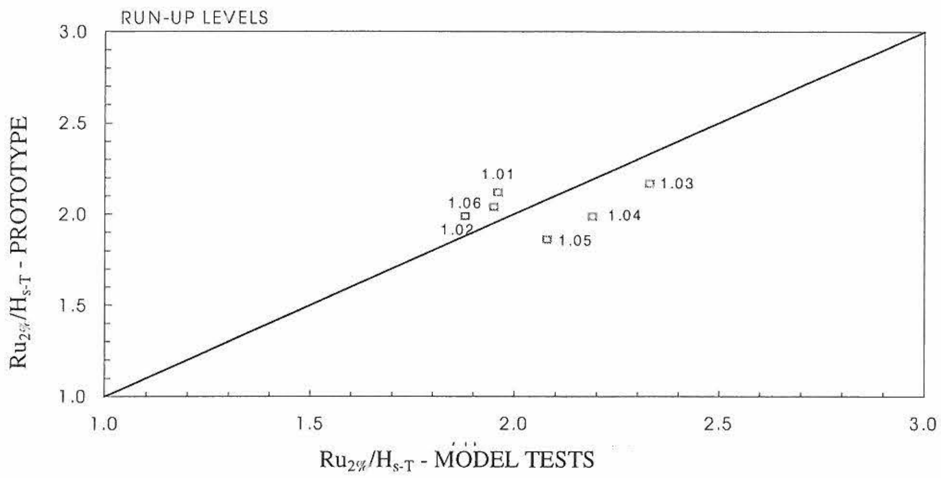

Figure 8: Comparison between wave run-up levels $Ru_{2\%}$ (normalised by the total significant wave height $H_{s\text{-}T}$) measured in 6 prototype storms and obtained from 3D model tests.

In addition to the comparison with prototype storms a parameter analysis was performed where wave heights, wave periods, water levels, wave direction and directional spreading were varied. For these results reference is made to Murphy (2001).

PROTOTYPE MEASUREMENTS AT ZEEBRUGGE BREAKWATER

In Zeebrugge prototype measurements are carried out on a rubble mound breakwater armoured with 25 ton grooved cubes (figure 9) (Troch et al. (1999)). Waves are measured by two wave rider buoys located at a distance of 150 and 215 m from the breakwater axis. Two different measuring devices are used for the measurement of wave run-up: a "spiderweb system" (SP) and a run-up gauge (RU). The "spiderweb system" is a set of 7 step gauges placed vertically between the armour units and the jetty bridge. Each step gauge measures the surface elevation of the uprushing water tongue. The wave run-up level is extrapolated from these measurements. A run-up gauge is mounted on top of the armour units.

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. During all storms wave run-up has been measured by the SP and during the last 9 storms also the RU was operational.

Again, the 2% exceedence level of the expected wave run-up Ru (relative to MWL) 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 depth in front of the structure, the length of the time series is important when half a tide cycle is analysed as the wave run-up value is calculated relative to a *constant* water level. Thirty minutes time series are used in the analysis of half a tide cycle (symmetric in time with regard to t_{HW}).

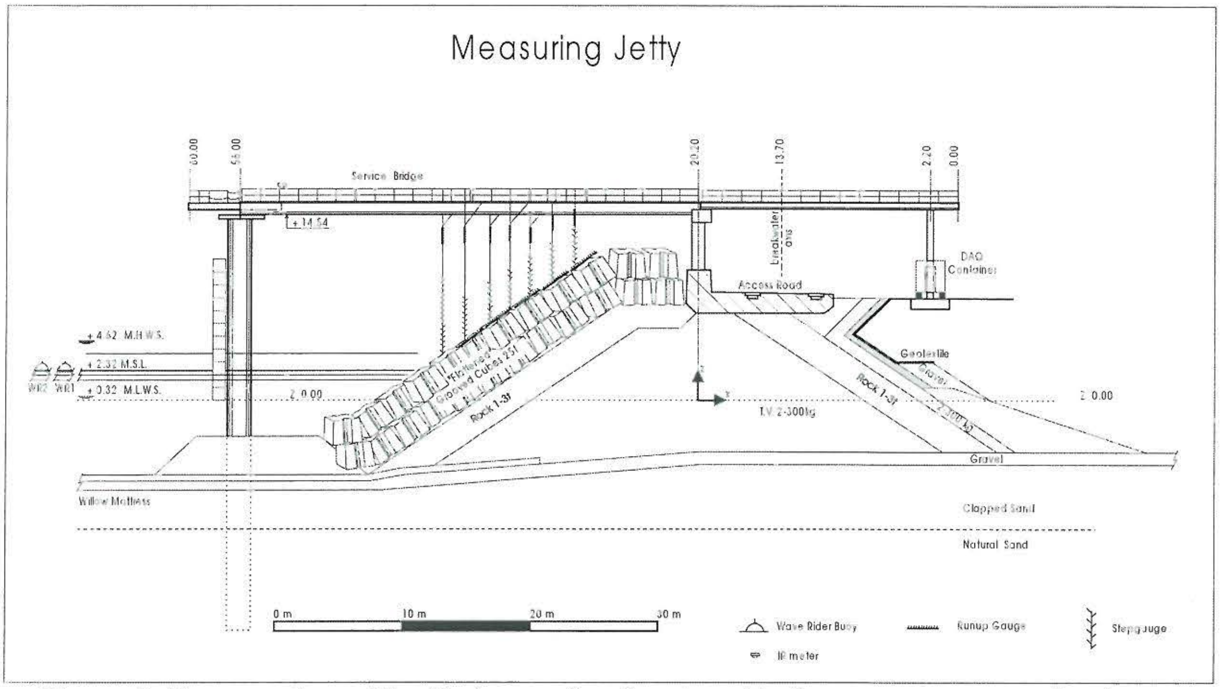

Figure 9: Cross section of the Zeebrugge breakwater with the prototype measuring jetty.

When time series with a period of time of 2 hours at high tide are analysed in their entirety, a mean dimensionless wave run-up value $\frac{Ru_{2\%}}{H_{mo}}$ of 1.74 is obtained when the RU data (9 storms) are processed. The analysis of the SP data (13 storms) yields a mean $\frac{Ru_{2\%}}{H_{mo}}$ value of 1.76. The mean water depth in front of the breakwater is d = 9.50 m. Both wave run-up measuring devices yield comparable results.

When 30 minutes time series are used in the analysis of the 2 hour period at high tide, $\frac{Ru_{2\%}}{H_{mo}}$ = **1.75** 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.

The results of an analysis of the data of half a tide cycle (using time series of 30 minutes) are mentioned in table 4 and plotted in figure 10. Different values of the exceedence probability x (1%, 2%, 5%, 10%, 25%, 50% as well as the maximum and the significant *Ru*) for dimensionless wave run-up $\frac{Ru_{x\%}}{H_{mo}}$ have been used. Wave run-up levels are slightly higher during flood than during ebb tide. This may be caused by different currents.

An interesting aspect from table 4 is that dimensionless wave run-up values increase when water depth (or mean water level (MWL)) decreases. The lower the exceedence probability x, the more the dimensionless wave run-up values increase (figure 10).

Table 4: 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	2.07	2.35	2.59	2.58
$\frac{Ru_{1\%}}{H_{mo}}$	2.48	2.19	1.91	2.07	2.21	2.15
$\frac{Ru_{2\%}}{H_{mo}}$	2.24	2.01	**1.75**	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

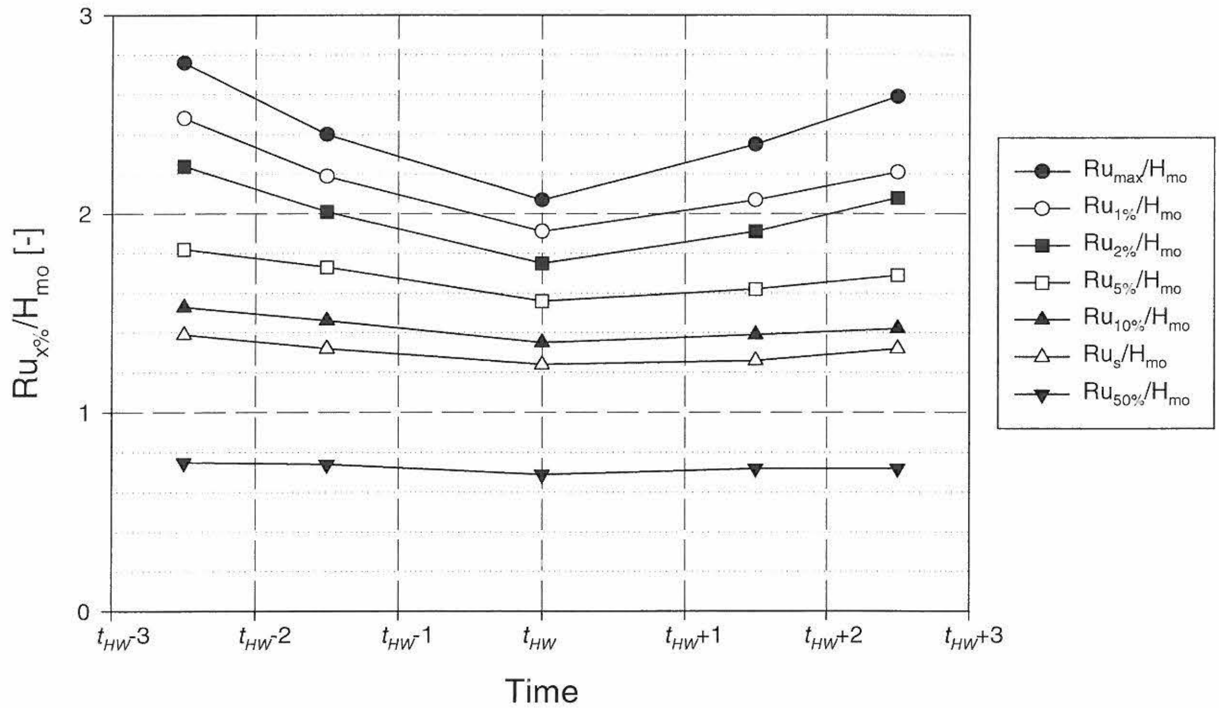

Figure 10: Dimensionless prototype wave run-up $\frac{Ru_{x\%}}{H_{mo}}$ vs. time.

A part of the explanation why dimensionless wave run-up values depend on the water depth 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 when water depth decreases also. 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) have been built at Flanders Hydraulics (FH) (1:30) and at Universidad Politécnica de Valencia (UPV) (1:30) and a 3D-model (1:40) has been built at Aalborg University (AAU). The armour units in the top layer are placed according to the actual position in full scale. Six 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 11). The step gauge is a comb of which the needles can be adjusted to the profile 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. The obtained results are summarised in table 5.

Figure 11: A novel step gauge for laboratory wave run-up measurements on a breakwater slope *(designed and constructed at Ghent University).*

In this paper, only the perpendicular incident waves are taken into account. In table 5, 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. All wave run-up values are measured by the novel step gauge. From table 5, two conclusion can be drawn: firstly, a clear difference between prototype measurement results and the physical modelling results of FH and AAU is noticed.

Secondly, the small scale model test results of UPV have the same order of magnitude of the prototype values.

Table 5: Laboratory results for Zeebrugge breakwater.

	length of time series	$\frac{Ru_{2\%}}{H_{mo}}$ [-] prototype measurements	ξ_{om} [-]	$\frac{Ru_{2\%}}{H_{mo}}$ [-] FH	$\frac{Ru_{2\%}}{H_{mo}}$ [-] UPV	$\frac{Ru_{2\%}}{H_{mo}}$ [-] AAU
Aug. 28, 1995	2h 15min	1.66	3.76	1.42		
Jan. 19, 1998	2h 30 min	1.73	3.70	1.53		
Jan. 20, 1998	2h	1.79	3.64	1.40		
Feb. 7, 1999	2h	1.73	3.55	1.39		
Nov. 6, 1999	2h	1.82	3.45	1.44	1.81	1.36
Nov. 6-7, 1999	2h	1.84	3.64	1.55	1.76	1.28

DISCUSSION OF ZEEBRUGGE RESULTS

Wave run-up was preliminary investigated in the MAST II project 'Full scale dynamic load monitoring of rubble mound breakwaters': 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. 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 physical modelling results.

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

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

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: firstly, equation (1) results from tests with regular waves and secondly, the results reported by Allsop et al. (1985) relate to structures with highly permeable mounds. Thirdly, 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:

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

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

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

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 4.

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 12.

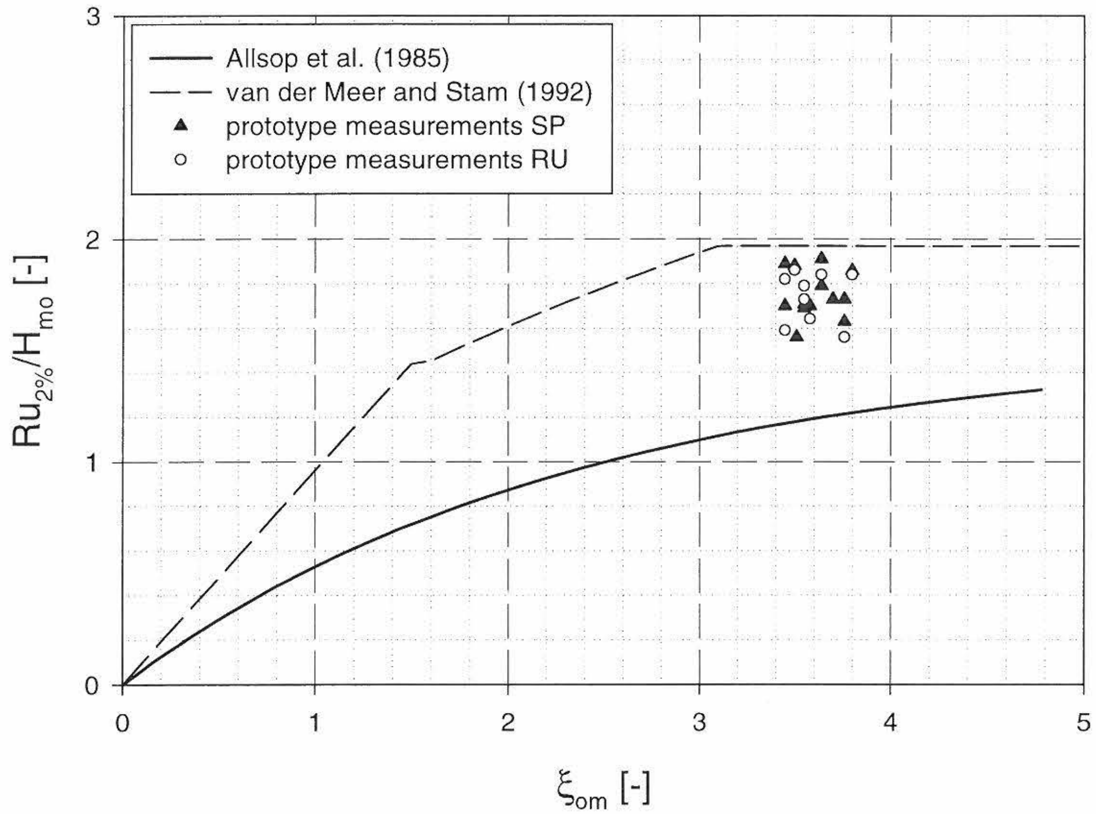

Figure 12: Comparison between dimensionless wave run-up values from prototype (from t_{HW}-1 to t_{HW}+1, SP (13 storms) & RU (9 storms), 2 hours time series) and from literature.

For the prototype value ξ_{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 ξ_{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 4 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:

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

Using the standard surf parameter ξ_{op} (calculated using T_p in stead of $T_{0,1}$), the run-up coefficients a and b equal respectively 1.022 and 0.247. Figure 13 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 with irregular waves on riprap protected slopes.

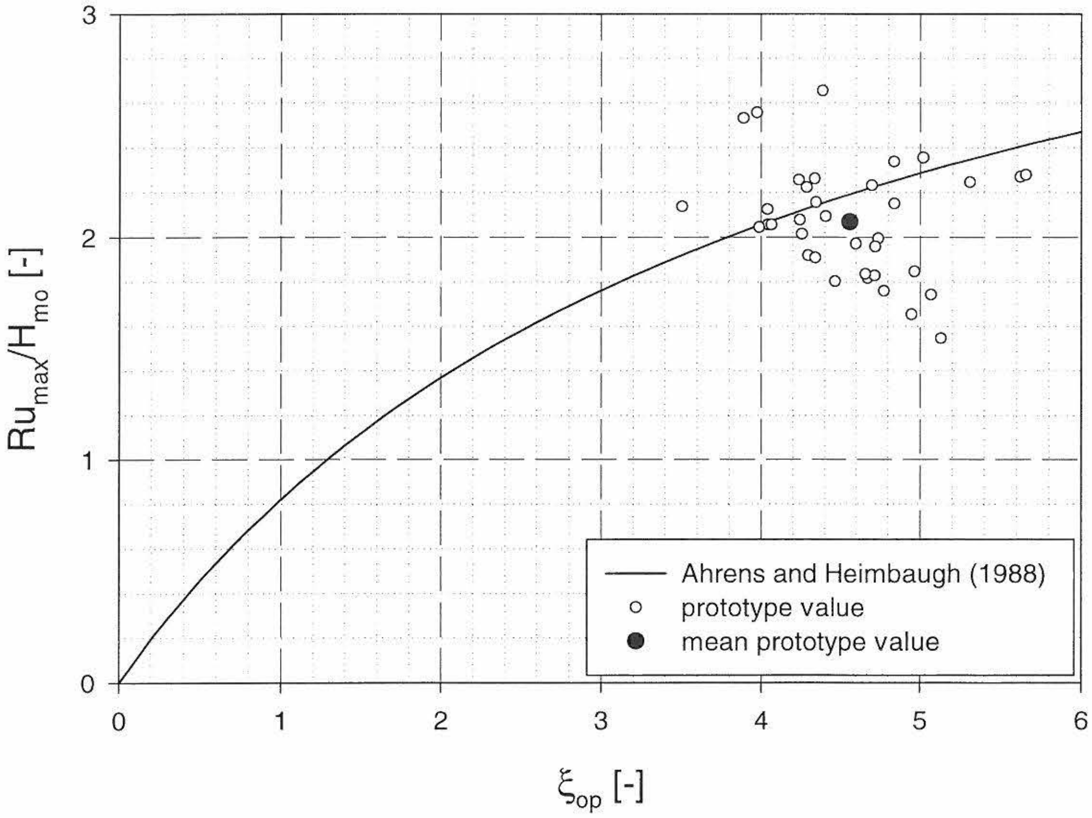

Figure 13: Comparison of prototype data to formula (3) (t_{HW}-1 t_{HW}+1, RU, 9 storms, 30 minutes time series).

From the graph in figure 12, it can be concluded that equation (2) yields a clear underestimation of the prototype wave run-up values. It seems that wave run-up on a rubble mound breakwaters armoured with grooved cubes may be evaluated by the formulae for riprap 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 are noticed (figure 14). Two laboratories (FH and AAU) yield almost the same $\frac{Ru_{2\%}}{H_{mo}}$ value of 1.40 at high tide. UPV finds a much higher value (comparable to the prototype value at high tide). A slight dependency (but the trend is not as strong as detected at prototype) on the water level is noticed in the 3D laboratory (AAU), whereas the dimensionless 2% wave run-up value of the FH laboratory remains quasi constant with changing water level (figure 15). UPV finds a comparable dependency on the water level as AAU finds, but the AAU values are lower.
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 ε and wave height distributions produced in different laboratories are noticed.

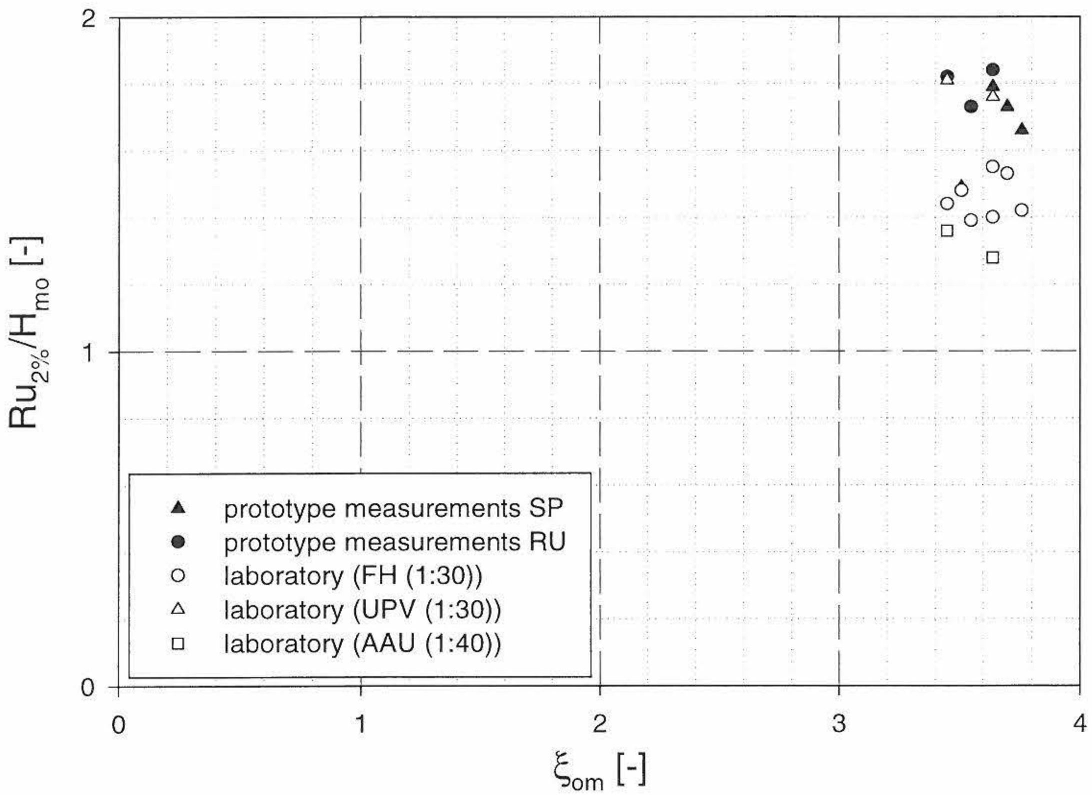

Figure 14: Comparsion between prototype measurements and small scale model test results *(cf. data in table 5).*

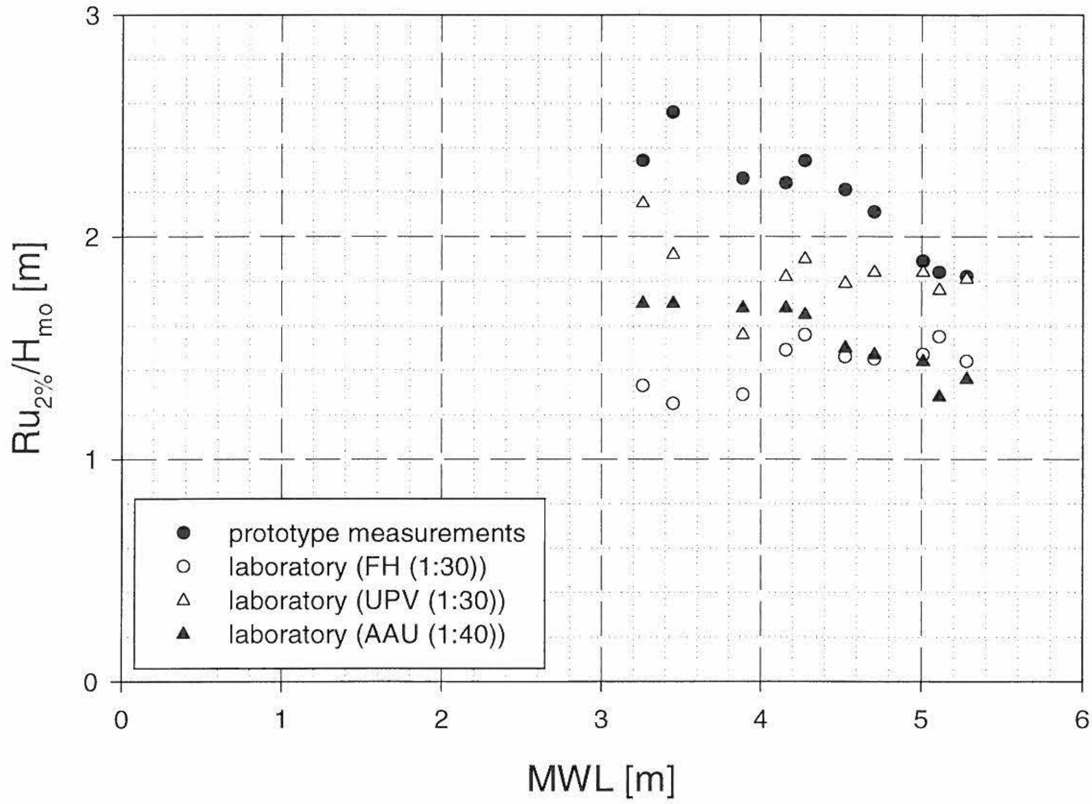

Figure 15: Comparsion prototype measurements (RU) and small scale model test results *(Nov. 6 & Nov. 6-7, 1999).*

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

CONCLUSIONS

Based on the synthesis of measurements on the Petten Sea-defence the following conclusions and recommendations are drawn:

- Prototype measurements of waves and wave run-up have been performed at the Petten Sea-defence (Rijkswaterstaat-RIKZ). The wave field at this site is dominated by wind waves with severe wave breaking at the relatively shallow parts of the foreshore (two bars and shallow water at the toe of the dike). The reliability of the instruments under heavy storm conditions has been demonstrated and together with the use of data verification techniques, this measurement campaign resulted in a valuable data-set of wave dynamics and wave run-up. The use of a video camera during storm conditions also provided important impressions of processes, such as the short-crested nature of the waves, the influence of the groins on waves, and variations in the wave run-up along the dike.
- Physical model tests (WL I Delft Hydraulics) show a good agreement with storms measured in prototype. The non-dimensional wave run-up levels differ only 3.9% on average. Considering the observed differences between prototype measurements and physical model tests it can be concluded that the schematisation and scale effects in the physical model tests were acceptably small.

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

- The mean dimensionless 2% wave run-up value equals 1.75 at a water depth of 9.50 m at the toe of the breakwater (ξ_{om} = 3.59) and increases when water depth decreases up to a mean value of 2.24 at mean tide.
- A clear difference between prototype measurements and small scale model test results is observed for the Zeebrugge rubble mound breakwater. Various factors leading to the difference between prototype measurement and small scale model test results have been highlighted.

In general it is concluded that the comparison of prototype measurements and results from laboratory investigations indicate that for dikes the wave run-up results correspond rather well, while for rubble mound breakwaters the observed differences require further investigations to draw firm conclusions on measurements-, schematisation- and scale-effects.

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

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