ANALYSIS OF WAVE REFLECTION FROM STRUCTURES WITH BERMS THROUGH AN EXTENSIVE DATABASE AND 2DV NUMERICAL MODELLING

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This paper analyses wave reflection from permeable structures with a berm, including reshaping cases. Data are obtained from recent wave flume experiments and from 2DV numerical simulations performed with the COBRAS-UC code. The objectives of this research were to identify the proper representation of the average structure slope to be included in the breaker parameter and to check the performance of the formula for the reflection coefficient developed for straight slopes by the Authors. Based on the observation that for reflection, differently from what happens for overtopping and run-up, the whole slope below sea water level (SWL) is important, the slope to appear in the breaker parameter is evaluated as a weighted average of the structure slope below the berm level and the average slope in the run-up/run-down area. The inclusion of this slope in the proposed formula allows to extend its prediction capacity to structures with a berm and a fair agreement with both experiments and simulations is obtained.

INTRODUCTION

Wave reflection from coastal structures is of high practical relevance to coastal engineers since it may induce dangerous sea states close to harbours entrances, too high reflections in harbours and intensified sediment scour, which can lead to structure destabilization. This fact has prompted numerous theoretical and model scale studies of wave reflection on different kind of slopes, which have yielded a variety of predictive schemes. Most of these schemes, both for smooth and rubble mound structures, related the reflection coefficient K_r to the surf similarity parameter ξ only, e.g. Battjes (1974), Seelig & Ahrens (1981), Postma (1989). From the work by Postma (1989) it is known that the wave period has more influence than wave height on the reflection behaviour, so the use of ξ introduces some scatter, but it also allows incorporating different slopes.

The reflection behaviour for various types of straight slopes, such as smooth structures, rock slopes (permeable and impermeable core), slopes with all kind of artificial armour units, has been analysed in depth by Zanuttigh and Van der

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Meer (2006) by means of an extensive reflection database, which includes part of the DELOS wave transmission database (Van der Meer et al., 2005) and of the CLASH wave overtopping database (Steendam et al, 2004; Bruce et al., 2006). They developed a new formula for all types of straight slopes in design conditions and performed a preliminary analysis on the extension to non-straight slopes (Zanuttigh and Van der Meer, 2007).

The objectives of the present research are:

- to develop a formula for the prediction of K_r in presence of a berm;
- based on the already developed formula for straight slopes, to identify the proper evaluation of the structure slope and thus of ξ in presence of a berm.

To achieve these objectives, two methodologies have been selected:

- analysis of reflection coefficients for structures with berms through an extensive database of more than 800 data (Lissev, 1993; Lykke Andersen, 2006; Sveinbjörnsson, 2008);
- numerical simulations with the 2DV COBRAS-UC code developed by the University of Cantabria (Losada et al., 2008).

THE ANALYSIS

The prediction formula for straight slopes

The formula by Zanuttigh and Van der Meer (2006) for predicting K_r reads

$$K_r = \tanh\left(a \cdot \xi_o{}^b\right) \tag{1}$$

This formula can be applied to all types of straight slopes; is based on the breaker parameter ξ_o, which is evaluated using the spectral period at the structure toe $T_{m-1,0}=m_{-1}/m_0$; represents physical bounds; is validated for straight slopes in design conditions: $R_c/H_{si} \geq 0.5$, $H_{m0}/D_{50} \geq 1.0$, $s_o \geq 0.01$. The agreement among straight rock permeable and impermeable, armour units and smooth slopes (around 600 data) and Eq. (1) is shown in Fig. 1.

It has been shown that the coefficients a and b in Eq. (1) depend only on the roughness factor γ_f as defined for overtopping. By fitting the well known values of γ_f for smooth structures, structures with permeable and impermeable core, the following expressions were derived

$$a = 0.167 \cdot \left[1 - \exp(-3.2 \cdot \gamma_f)\right], \; b = 1.49 \cdot (\gamma_f - 0.38)^2 + 0.86 \tag{2}$$

Since γ_f has been measured or determined for a lot of materials (Bruce et al., 2006), the dependence of a and b on this parameter allows to straightforward extend Eq. (1) to a wide variety of slopes obtaining good predictions without any refitting.

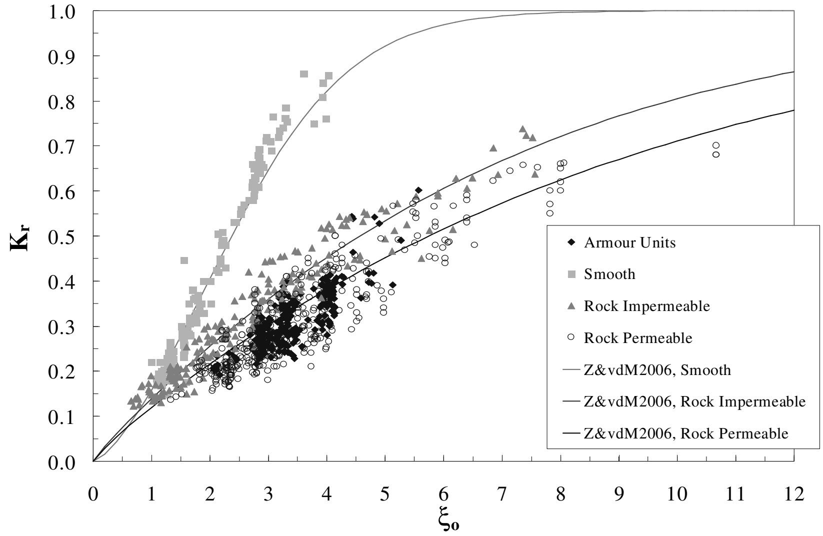

Figure 1. Overall comparison of the formula by Zanuttigh and Van der Meer (2006), eq. (1) with all data of non or hardly overtopped straight slopes.

The problems posed by structures with berm

Limited information on wave reflection from composite slopes is available in the literature. Alikhani (2000) found that for reshaping berm breakwaters the slope of the structure has no influence on the reflection coefficient, because higher waves cause flatter slopes and compensate the incident wave energy. Based on his experimental results, he developed the following formula

$$K_r = 0.044 \cdot s_{op}^{-0.46} \tag{3}$$

where s_{op} is the peak wave steepness off-shore the structure.

Lykke Andersen (2006) observed that for a reshaping berm breakwater the slope and hence the surf similarity parameter vary along the slope making it difficult to represent the breaking on the structure and the phase lag between reflections from different parts of the structure with a single value of ξ. By analysing his wide database, he concluded that the slope above SWL is less important for reflection and he found a reasonable correlation between K_r and the breaker parameter at the structure toe based on peak wave period ξ_{op}, when the breakwater slope is calculated as the average slope between SWL and $1.5 \cdot H_{m0t}$ below SWL. Fair predictions were obtained by Lykke Andersen by introducing this average slope, directly measured from structure profiles, in the formula by Postma (1989). Indeed this method is not applicable when the berm is at SWL and does not predict any effect of the berm when it is emerged or deeply submerged (more than $1.5 \cdot H_{m0t}$).

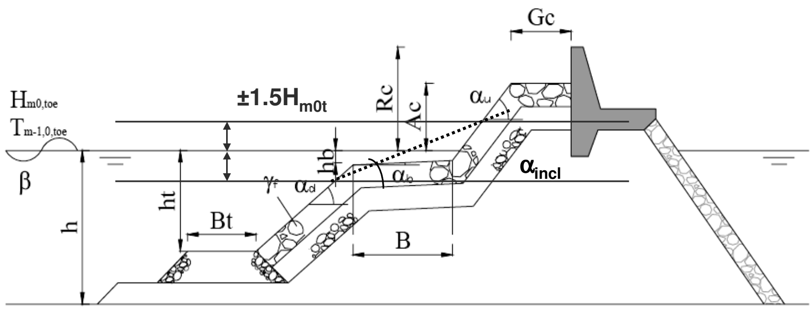

Figure 2. Structure parameters in the reflection database, based on CLASH schematization, redrawn from Steendam et al. (2004).

For rubble mound breakwaters, the only formula that does not include the structure slope was proposed by Muttray et al. (2006)

$$K_r = 1/(1.3+3h \cdot 2\pi/L_0) \quad (4)$$

where h is water depth at the toe and L_o the off-shore wave length. Eq. (4) was validated against a limited dataset for a typical cross section with 1:1.5 slopes, no berm, an armour layer of accropodes and a core of gravel.

Starting from the work by Lykke Andersen (2006), Zanuttigh and Van der Meer (2006) carried out a preliminary analysis on a limited dataset composed by permeable and impermeable structures with a berm, whose results can be summarized as follows:

- what reflects is the slope below SWL;
- for combined slopes an average slope has to be included in ξ;
- reflection is influenced by wave breaking and run-up. The lower the run-up the greater the reflection, and the greater the energy dissipation by breaking on the berm, the lower the reflection. The presence of a toe and/or a berm should thus be accounted for whenever it may affect these processes, more specifically also when the berm is placed in the run-up area till $+1.5 \cdot H_{m0t}$.

In the attempt to consider the presence of the berm even when it is at SWL or above it, the Authors suggested to use the following average structure slope:

$$\xi_o = \frac{\left[\tan\alpha_d \cdot \left(h - 1.5H_{m0t}\right) + \tan\alpha_{inc} \cdot 1.5H_{m0t}\right]/h}{\sqrt{H_{m0t}/L_0}} \quad \text{if } h > 1.5H_{m0t}$$
$$\xi_o = \tan\alpha_{incl} \Big/ \sqrt{H_{m0t}/L_0} \quad \text{if } h \le 1.5H_{m0t} \quad (5)$$

The weighted average slope in Eq. (5)

- is performed over the water depth at the structure toe h;
- makes use of the average slope, α_{incl}, in the whole run-up/down area.

In the present contribution the applicability and the performance of Eq. (5) is checked in depth against rock permeable structures with a berm.

THE DATA

The experimental dataset

The data used for the purpose of this analysis include the work by Lissev (1993) and the more recent datasets produced by Lykke Andersen (2006) in the wave flume 21.5x1.2x1.5m at Aalborg University and by Sveinbjörnsson (2008) in the wave flume 25x0.8x0.9 m at Delft University of Technology, see Fig. 3. Berm breakwaters testes by both Lissev (1997) and Lykke Andersen (2006) are reshaping, whereas the ones tested by Sveinbjörnsson (2008) are non-or hardly reshaping berm breakwaters (Icelandic type).

Irregular waves with a Jonswap spectrum were generated and the wave height in each test series gradually increased in steps with constant wave steepness till a stable deformation of the breakwater was reached. Wave data were recorded from three wave gauges in front of the structures and are analysed accordingly to Mansard and Funke (1987) method. Characteristics of structure geometry and incident wave conditions are summarized in Table 1.

Table 1. Main characteristics of the experimental dataset (min/max values): *h* is the water depth, H_{m0t} is incident wave structure height, s_o is wave steepness based on wave spectral period, R_c is structure crest freeboard, h_b is berm submergence, *B* is berm width, D_{50} is the average stone diameter, *m* is the foreshore slope. The 'L' label corresponds to Lissev (1993), 'A' to Lykke Andersen (2006), 'S' to Sveinbjörnsson (2008) dataset.

	#	$cot\alpha_d$	$cot\alpha_{inc}$	h, m	R_c/H_{m0t}	H_{m0t}/h	h_b, m	B/H_{m0t}	s_o %	H_{m0t}/D_{50}	m
L	69	3.79	2.64	0.79	0.94	0.06	0.05	-	2	1.31	1000
		3.85	2.80		6.22	0.38			6	8.63	
A	695	1.13	1.25	0.26	0.64	0.14	-0.12	0.00	1	1.92	20
		7.80	4.34	0.47	1.97	0.50	0.15	8.47	6	7.84	
S	71	1.50	2.00	0.55	0.68	0.13	-1.13	1.69	4	3.53	-
			2.60	0.65	2.17	0.28	-0.08	3.61	6	7.48	

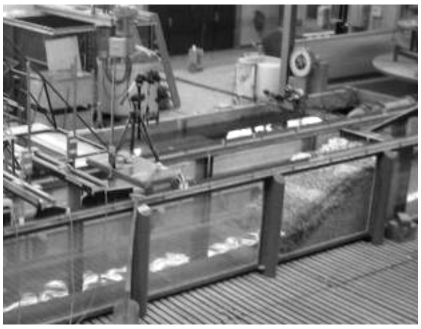

Fig. 3 Typical cross section tested by Lykke Andersen (2006) at the left and by Sveinbjörnsson (2008) at the right.

The numerical dataset

Numerical simulations were carried out with the 2DV COBRAS-UC code developed by the University of Cantabria (Losada et al., 2008, Lara et al., 2008). To avoid scale effects, the runs were performed at prototype scale in a 400x24 m numerical wave flume.

Tested wave conditions include:

- 2 wave heights, H_{m0}* (design conditions) and 2/3 H_{m0}* ;
- 2 wave steepnessess s_{op}, to represent storm waves and swell waves or broken waves over a shallow foreshore,

whereas water depth h is kept constant (h=2.5 H_{m0}*).

Three wave attacks are thus globally analysed:

- Wave A: H_{m0} =4.5 m, s_{op}=0.02, h=11.25 m;
- Wave B: H_{m0} =4.5 m, s_{op}=0.04, h=11.25 m;
- Wave C: H_{m0} =3.0 m, s_{op}=0.04, h=11.25 m.

Wave attacks lasted around three hours to represent on average 300 waves per test, see an example of wave generation within the code in Fig. 4. The reflection coefficient was derived by applying the Mansard and Funke (1987) method to three wave gauges placed in front of the structure, at a distance from the structure toe equal to 1.5 times the maximum wave length.

Tested structure geometries, schematized in Figure 5, consist of rock slopes (similarly to Sveinbjörnsson, 2008) characterized by

- three different berm widths (B=0-3 H_{m0}*-6· H_{m0}*);
- three different berm submergence (h_b=-2/3·H_{m0}* -0-+2/3· H_{m0}*);

Time was insufficient to check the effects induced by a berm and different structure slopes: only flat berms ($cot\alpha_b$=0) and constant upstream and downstream structure slopes were used ($cot\alpha_d$ = $cot\alpha_u$=1.5).

The slopes are composed by three layers: a 2-stones outer-layer, a 2-stones under-layer and a core, whose characteristics are reported in Table 3. The size of the Dn_{50} for the outer layer was designed for the limit stability condition (Van der Meer (2002) for Wave A.

Figure 6 shows some sample snapshots representing the same incident wave attack (Type A) hitting at the same instant three slopes: a straight one, a slope with a horizontal berm at mean sea level and a slope with a horizontal submerged berm. It may be estimated from this figure that, particularly in the case of the berm at SWL, the wave breaking and dissipation induced by the berm reduces wave reflection.

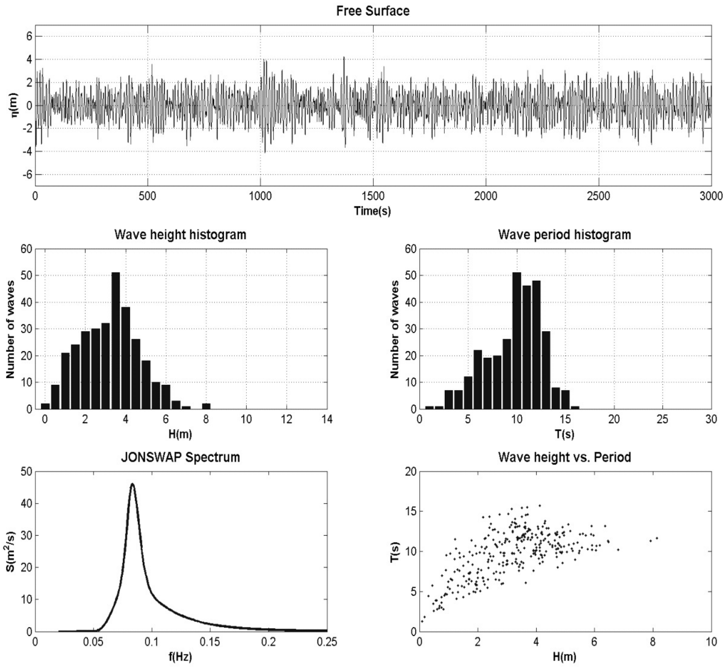

Figure 4. Example of wave generation by the COBRAS-UC model. Target sea state: H_s=4.5 m T_p=12.01 s, 307 waves, sea storm duration: 3 hours.

Table 2. Main characteristics of the numerical dataset: *h* is the water depth, H_{m0t} is incident wave structure height, T_p is peak wave period, h_b is berm submergence, *B* is berm width, D_{50A} is the armour average stone diameter, structure slopes follow the scheme given in Figure 2.

Test	H_{m0t} m	T_p, s	*h*, m	D_{50}, m	$cot\alpha_d$	$cot\alpha_u$	$cot\alpha_b$	$cot\alpha_{inc}$	h_b, m	*B*, m
1	4.34	12.01	11.25	1.50	1.5	1.5	--	1.50	--	--
2	3.91	8.49	11.25	1.50	1.5	1.5	--	1.50	--	--
3	2.73	6.93	11.25	1.50	1.5	1.5	--	1.50	--	--
4	4.36	12.01	11.25	1.50	1.5	1.5	0.0	2.40	0.0	13.5
5	3.93	8.49	11.25	1.50	1.5	1.5	0.0	2.75	0.0	13.5
6	2.69	6.93	11.25	1.50	1.5	1.5	0.0	2.80	0.0	13.5
7	4.37	12.01	11.25	1.50	1.5	1.5	0.0	2.50	3.0	13.5
8	3.94	8.49	11.25	1.50	1.5	1.5	0.0	2.58	3.0	13.5
9	2.83	6.93	11.25	1.50	1.5	1.5	0.0	3.10	3.0	13.5
10	4.36	12.01	11.25	1.50	1.5	1.5	0.0	2.55	-3.0	13.5
11	3.95	8.49	11.25	1.50	1.5	1.5	0.0	2.72	-3.0	13.5
12	2.79	6.93	11.25	1.50	1.5	1.5	0.0	3.15	-3.0	13.5
13	4.36	12.01	11.25	1.50	1.5	1.5	0.0	2.88	0.0	27.0
14	3.88	8.49	11.25	1.50	1.5	1.5	0.0	3.08	0.0	27.0
15	2.76	6.93	11.25	1.50	1.5	1.5	0.0	3.63	0.0	27.0
16	4.39	12.01	11.25	1.50	1.5	1.5	0.0	2.77	3.0	27.0
17	3.89	8.49	11.25	1.50	1.5	1.5	0.0	2.95	3.0	27.0
18	2.84	6.93	11.25	1.50	1.5	1.5	0.0	3.57	3.0	27.0

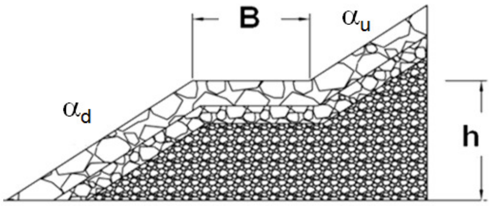

Fig. 5 Tested cross-section in numerical simulations.

Table 3. Main characteristics of the structure layers, see scheme in Fig. 5. The parameters α and β appear in the Forchheimer equation and are constants depending on flow shape in pores; n is layer porosity.

	α	β	n	D_{50}, m
Outer layer	200	0.7	0.45	1.5
Under layer	200	1.1	0.35	0.7
Core	200	0.8	0.25	0.1

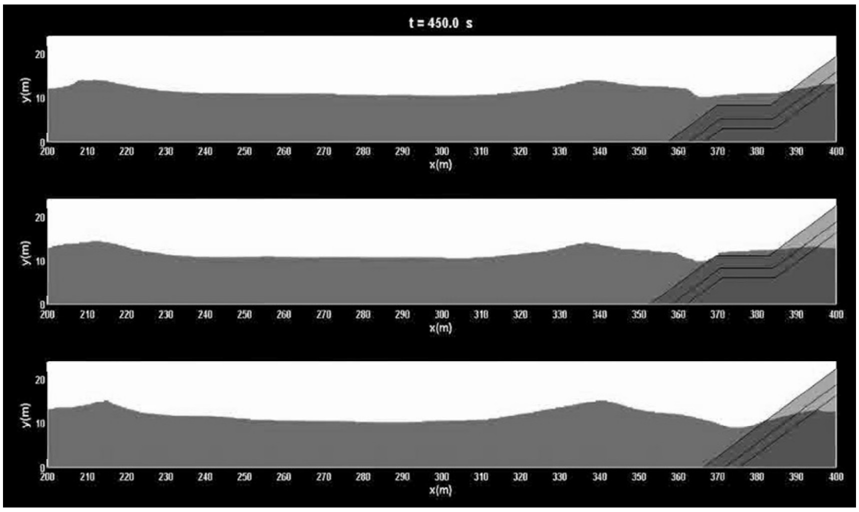

Figure 6. Incoming waves (type A wave attack) against a slope with submerged berm (Test 10), a slope with berm at mean sea level (Test 4), a straight slope (Test 1).

THE RESULTS

Experimental data are compared with existing formulae and with the proposed new formula – Zanuttigh and Van der Meer (2007) - given by coupling Eq.s (1) and (5) and selecting γ_f=0.45 for all cases.

Figure 7 and 8 present the laboratory datasets against the formulae by Alikhani (2000) and by Zanuttigh and Van der Meer (2007), respectively. Data are divided into stable and reshaping conditions, since the first formula was developed explicitly for reshaping cases. From Figure 7 it is evident that Alikhani (2000) can be properly applied to the reshaping cases of Lykke Andersen (2006), but shows quite a lot of scatter for the other datasets even in reshaping conditions, especially for Sveinbjörnsson (2008). In Figure 8 the data are on average well fitted by Eq.s (1) and (5) and show the same scatter as the data of the rock permeable straight slopes (Zanuttigh and Van der Meer, 2006).

Table 4 summarises the performance of the formulae by Alikhani (2000), Eq. (3); Muttray et al. (2006), Eq. (4); Zanuttigh and Van der Meer (2006), Eq. (1). It is not evident from the comparison with experimental data if the calculation of the structure slope from Eq. (5) instead than unsing α_{incl} really improves the performance of Eq. (1). Indeed in both cases Eq. (1) provides for almost all tests the lowest error, with the exception of the reshaping data of Lykke Andersen (2006) that are better represented by Alikhani (2000).

Table 4. Percentage RMS errors obtained from prediction formulae against the experimental datasets (labels as in Tab. 1, with separation among reshaping R and non-reshaping NR cases). Formulae compared are by Muttray et al. (2006), 'M', Eq. (4); Alikhani (2000), 'Al', Eq. (3); Zanuttigh and Van der Meer (2006), Eq. (1), 'Z&VM'.

		M	Al	Z&VM		
	#			α_d	α_{incl}	$\alpha_{Eq. (5)}$
L, R (all)	69	5.39	9.01	4.73	6.61	4.04
A, R	427	8.73	3.80	10.12	4.28	6.38
A, NR	268	22.06	8.61	12.39	7.17	5.44
A, all	695	13.87	5.65	11.00	5.39	6.02
S, R	29	7.42	9.56	4.75	5.40	3.29
S, NR	42	8.19	6.96	8.16	4.36	5.84
S, all	71	7.88	8.02	6.77	4.78	4.80
ALL	**835**	**12.66**	**6.13**	**10.12**	**5.44**	**5.75**

Numerical simulations were used to check the effect of a berm in a more controlled environment, with the particular aim at identifying the proper slope to be included in the expression for the breaker parameter.

Figure 9 shows the reflection coefficient as function of the wave steepness s_o, which appears to have the greatest effect on wave reflection: with increasing s_o, K_r decreases. As expected, K_r for a structure with emerged berm is close to the case of a similar homogeneous straight slope and it tends to decrease with increasing berm submergence at least for the lower values of s_o.

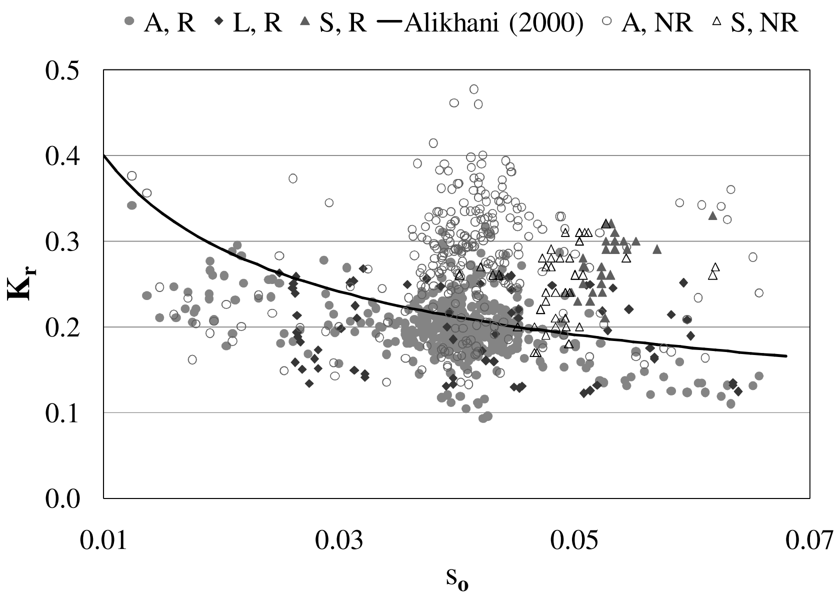

Figure 7. Comparison among the experimental dataset and Alikhani (2000) formula, Eq. (3). Labels as in Tab. 1 and Tab. 4.

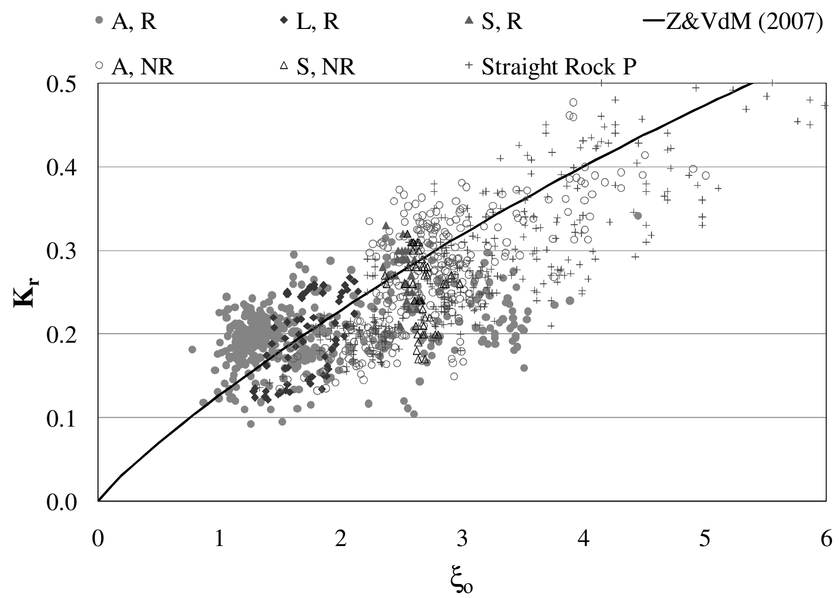

Figure 8. Comparison among the experimental dataset and Zanuttigh and Van der Meer (2007) formula, Eq.s (1)+ (5). Labels as in Tab. 1 and Tab. 4. The figure contains also data of rock permeable straight slopes.

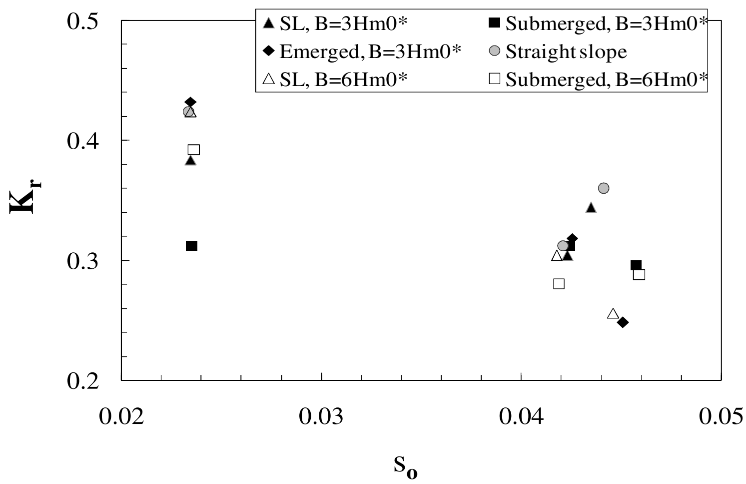

Figure 9. Reflection coefficients obtained by numerical simulations as function of wave steepness.

Figure 10 shows the values of K_r compared to Eq. (1), being γ_f=0.45 for all cases. The values for straight slopes are perfectly fitted by the curve. If ξ is computed based on α_d, all values for structures with berms are lower than the curve, whereas if α_{incl} is used they fall all above the curve. If the average structure slope is expressed by Eq. (5), the curve well fits the data cloud. The percentage errors obtained with the different slopes are reported in Tab. 5, from which it can be concluded that

- Eq. (5) provides the best representation of the average structure slope;
- Eq.s (1) and (5) together can accurately predict K_r for structures with berm.

Table 5. Percentage RMS errors obtained from prediction formulae against the numerical dataset (labels as in Tab. 4).

#	M	AI	Z&VM		
			α_d	α_{incl}	$\alpha_{Eq. (5)}$
18	5.39	9.01	4.73	6.61	4.04

CONCLUSIONS

Great progress has been made in estimating wave reflection coefficients as shown in Zanuttigh and Van der Meer (2006, 2007). Two ways for analysing wave reflection from berm breakwaters were adopted: extra data sets (around 850 data) and numerical simulations with the 2DV Cobras-UC code.

The final conclusion is that the calculation of the average structure slope by a weighted average between the down slope and the slope in the run-down/up area, Eq. (5) by Zanuttigh and Van der Meer (2007), is confirmed.

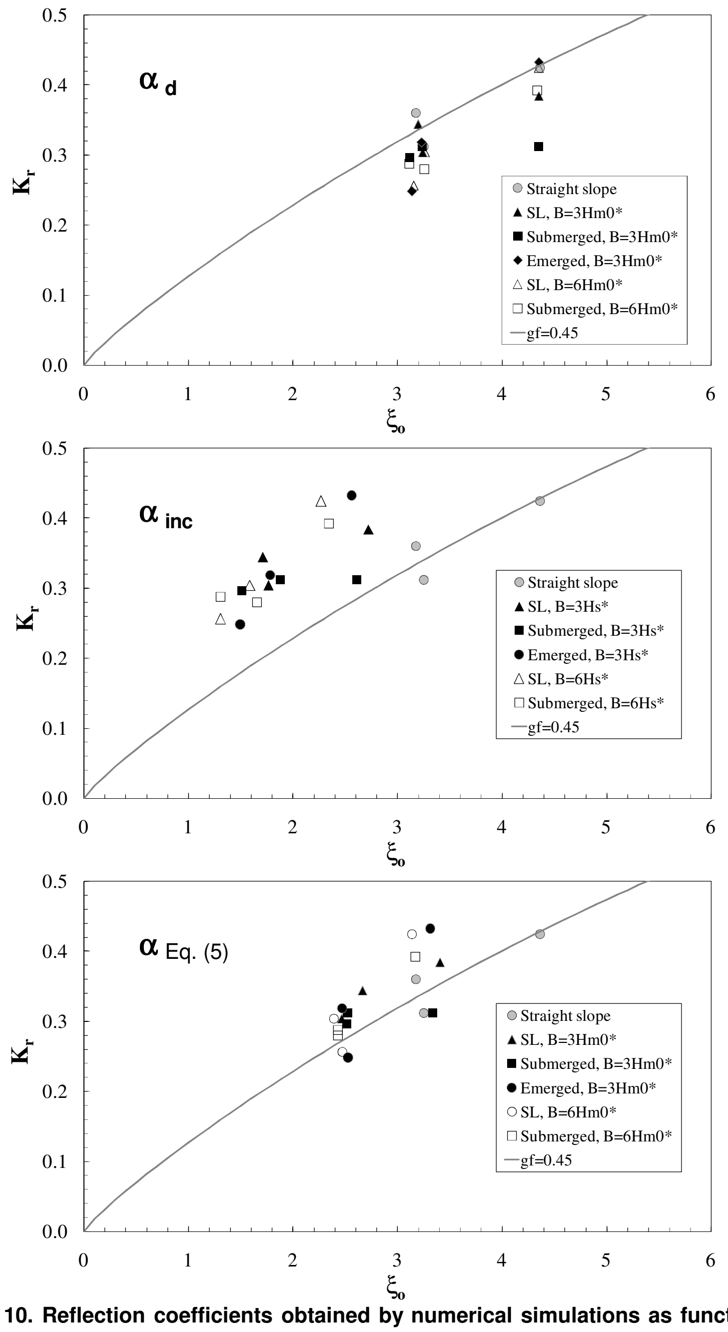

Figure 10. Reflection coefficients obtained by numerical simulations as function of the breaker parameter evaluated using, from top to bottom, the downstream slope, the average slope in the run-down/run-up zone and the one derived from Eq. (5).

The formula by Zanuttigh and Van der Meer (2006), Eq. (1), extended to berm breakwaters by means of Eq. (5), provides a good agreement with numerical and experimental datasets for stable and reshaping structures.

REFERENCES

Alikhani, A. 2000. *On Reshaping Breakwaters*. Ph.D. thesis, Hydraulics & Coastal Engineering Laboratory, Aalborg University.

Battjes, J.A., 1974. Surf similarity. *Proc. 14th Conf. on Coastal Eng*., 466-480.

Bruce, T., Van der Meer, J. W., Franco, L. and J. Pearson, 2006. A comparison of overtopping performance of different rubble mound breakwater armour. *Proc. ICCE 2006*, vol. 5, 4567-4579.

Lykke Andersen, T., 2006. *Hydraulic Response of Rubble Mound Breakwaters*. Aalborg University, PhD thesis, 429 pp.

Lissev, N. 1993. *Influence of the core configuration on the stability of berm breakwaters. Experimental model investigations*. Report No. R-6-93, Dep. of Struct. Eng., Univ. of Trondheim, The Norwegian Inst. of Technology.

Losada, I.J., Lara, J.L., Guanche, R., Gonzalez-Ondina, J.M. 2008. Numerical analysis of wave overtopping of rubble mound breakwaters. *Coastal Eng.*, 55 (1), 47-62.

Mansard, E.P.D., Funke, E.R., 1987. *On the reflection analysis of irregular waves.* Natl. Res. Counc. Rev. Can. Hydr. Lab. Tech. Report, TR-HY-O17

Muttray, M., Oumeraci, H., Oever, E. 2006 Wave reflection and wave run-up at rubble mound breakwaters *Proc. ICCE 2006*, vol. 5, 4314-4324.

Postma, G. M., 1989. *Wave reflection from rock slopes under random wave attacks*, PhD thesis, Delft University of Technology.

Seelig W.N. & J.P. Ahrens, 1981. *Estimation of wave reflection and energy dissipation coefficients for beaches, revetments and breakwaters*. CERC Tech. paper 81-1, USACE, Vicksburg, MS.

Steendam G.J., van der Meer J.W., Verhaeghe H., Besley P., Franco L., M. van Gent M., 2004. The international database on wave overtopping. *Proc. ICCE 2004*, 4: 4301-4313.

Sveinbjörnsson, P. I. 2008. *Stability of Icelandic type Berm Breakwaters*. MSc Thesis, TU Delft, Dep. of Hydr. Eng.

Van der Meer J.W., 2002. Wave run-up and wave overtopping at dikes. *Technical Report of the Technical Advisory Committee on Water Defences in the Netherlands.*

Van der Meer J.W., Briganti R., Zanuttigh B. and B. Wang, 2005. Wave transmission and reflection at low crested structures: design formulae, oblique wave attack and spectral change. *Coastal Eng.*, 52(10-11): 915-929

Zanuttigh, B. and J.W. van der Meer, 2006. Wave reflection from coastal structures, *Proc. ICCE 2006*, vol. 5, 4337-4349.

Zanuttigh, B. and J. W. van der Meer, 2007. Wave reflection for composite slopes and oblique waves, *Proc. Coastal Structures 2007*, in press.

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ANALYSIS OF WAVE REFLECTION FROM STRUCTURES WITH BERMS THROUGH AN EXTENSIVE DATABASE AND 2DV NUMERICAL MODELLING

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Breakwaters
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Composite slope
Reflection coefficient
Breaker parameter
Database
Numerical model