

Reshaping breakwaters in deep and shallow water conditions

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Abstract

In order to investigate the influence of water depth on the profile of reshaping breakwaters, model tests have been performed in deep, intermediate and shallow water conditions. The results show that the reduction factor for the dimensions of a reshaped profile in intermediate and shallow water conditions does not depend only on the wave height reduction. Other influencing factors are identified as breaking depth, wave steepness and foreshore slope. Moreover, at any water depth condition, the characteristic wave height corresponding to the frequency of stone movements on a reshaped profile is larger than the significant wave height. In the present tests, good results are obtained using the 1/50 wave height. Finally, whereas the offshore evaluated reflection coefficient increases with the water depth at the structure, the same coefficient evaluated in front of the structure shows an opposite trend.

Introduction

In the last decade researchers have paid great attention to the characteristics of berm breakwaters (Burcharth & Frigaard, 1987; van der Meer, 1988; Juhl & Jensen, 1990; Hall & Kao, 1991). Such structures are also named sacrificial or reshaping breakwaters, as sometimes a berm is present also in the armor layer of a conventional non reshaping breakwater. In a reshaping breakwater, if a sufficient amount of material is provided, a stable S-shape profile is eventually developed. During the reshaping phase, the material is removed from zones where it is unstable and relocated into more stable positions; a sorting process is also active during reshaping since the less stable units in the mound are removed first; at the end of reshaping if a low mobility remains active throughout the phase, a profile with a uniform distribution of mobility is naturally formed. The self armoring response of reshaping breakwaters to wave forces makes them economically attractive because finer rock material can be used than for a conventional breakwater; construction and maintenance can also use less expensive equipments.

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Most laboratory investigations aimed to foresee the behavior of the reshaping profiles under deep water conditions. In particular, laboratory investigations have considered 2- and 3-dimensional stability of reshaping breakwaters, referring mainly to the trunk section, but also to the rear side (Andersen & al., 1992) and to the roundhead (Burcharth & Frigaard, 1987). Only few tests have considered reshaping of the seaward profile in shallow water conditions. A large number of breakwaters in the Mediterranean area operate in shallow water conditions and moderate tidal excursion; in these conditions waves, before hitting the structure, undergo a relevant energy decay and an even more pronounced decay of extreme wave height.

This paper investigates the influence of water depth in front of the structure on the seaward profile development of a reshaping breakwater, comparing results of 2-dimensional model tests performed at Danish Hydraulic Institute (DHI) and Estramed laboratories for deep, intermediate and shallow water conditions.

Description of the model tests

Flumes

Physical model tests were carried out using the facilities at DHI (Spring 1992) and Estramed (Summer 1993, Summer 1994) laboratories. Tests were performed in glass walled flumes.

In deep water tests (referred to as DHI-92) the flume presented a horizontal foreshore with water depth 0.60 m. Intermediate and shallow water tests were performed using two different flumes. The first flume (referred to as Estramed-93) presented a bottom (fig. 1a) consisting of a foreshore slope 1:20 from 0.60 to 0.36 m water depth, followed by a milder 1:100 slope reaching 0.30 m water depth at structure toe. The second (referred to as Estramed-94) presented a bottom (fig. 1b) with a foreshore slope 1:20 from 0.60 to 0.34 m water depth, followed by a milder 1:100 slope; on this slope the structures were placed at depth 0.30 and 0.14 m. Characteristics of the flume are summarized in table 1.

Table 1. Tests set-up characteristics (measures in m)

No.	Test series	Width	Effective length	Toe depth	Foreshore slope
1	DHI-92	0.60	16.0	0.60	0
2	Estramed-93	1.50	22.6	0.30	1:100
3	Estramed-94	0.75	13.2	0.30	1:100
4	Estramed-94	0.75	33.6	0.14	1:100

Materials

For present tests, armor materials and wave characteristics were chosen in order to obtain values of the stability number $H_s/\Delta D_{n50}$, the index of the armor layer mobility, close to 3. Moreover, water depth was varied in order to obtain values of

the ratio h/H_{so} as low as 0.85. Values of h/H_{so} larger than 3.0 provided deep water conditions; values of h/H_{so} around 2.0 gave intermediate water conditions.

All tested structures were built with crushed stones whose characteristics for the armor and the core are given in table 2. Gradation curves were based on samples of more than 200 stones.

Table 2. Rock material characteristics (measures in cm)

No.	Test series	Armor D_{n50}	Armor D_{n85}/D_{n15}	Core D_{50}	Core D_{85}/D_{15}	Δ	Shape
1	DHI-92	3.4	1.42	1.1	2.8	1.68	Angular
2	Estramed-93	2.5	1.50	0.9	2.5	1.65	Angular
3	Estramed-94	2.6	1.66	0.9	2.5	1.65	Angular
4	Estramed-94	1.5	1.66	0.9	2.5	1.65	Angular

Structure shapes

Berms were built approximately 0.1 m above mean water level (fig. 2) and a certain amount wider than the foreseen erosion. Seaward slopes of as built structures were in the range 1:1.1 to 1:1.5. The geometrical characteristics of the structures are summarized in table 3.

Table 3. Berm breakwaters sections (measures in cm)

No.	Test series	Toe depth	Berm width	Berm freeboard	Crest width	Crest freeboard
1	DHI-92	60	70	10	30	20
2	Estramed-93	30	55	10	30	20
3	Estramed-94	30	55	10	30	20
4	Estramed-94	14	35	6	20	12

Wave attacks

In all the tests irregular waves of the Pierson-Moskowitz spectral shape were generated by hydraulically actuated piston-type wave makers without re-reflection absorption system.

In DHI tests, where depth was constant everywhere offshore the breakwater, 4 wave gauges were installed half way between the wave paddle and the breakwater and some further ones closer to the toe of the structure. The four gauges were used to estimate incident and reflected spectra, while the gauges at the structure toe were used as phase reference for velocity measurements. In Estramed tests, 3 gauges were installed in front of the breakwater and 3 offshore, not far from the wave paddle. In this way both offshore and inshore incident and reflected waves were measured. Waves were recorded continuously throughout all the tests. Wave gauges were statically calibrated (in green water); non linear effects are stronger in shallow water conditions and are not accounted for by the adopted separation method between

incident and reflected spectra (Goda & Suzuki, 1976; Mansard & Funke, 1980). Measures of incident and reflected wave height in shallow water conditions should be regarded as affected by greater experimental uncertainties.

Offshore wave characteristics were measured at 0.60 m water depth and are summarized in table 4.

All tests consisted of a reshaping phase during which design wave conditions were applied lasting 6x1000 waves. Profiles were surveyed by a mechanical sounding, i.e. measuring the depth of the profile relative to a horizontal reference plane. Profile surveys were repeated at least 3 times during reshaping; after 1000, 3000 (or 2000) and 6000 waves. Data were digitized and visually analyzed in order to estimate the characteristic dimensions of the profile according to Vellinga (1986) parameterization scheme. It was observed that the largest portion of berm erosion occurred during the attack of the first few hundreds waves. After this phase, stones moved singularly and rarely.

A small amount of wave overtopping occurred during tests at DHI and no overtopping at all was observed during the other tests.

Table 4. Offshore wave characteristics (measures in cm, s)

No.	Test series	Section	H_{s0}	$H_{1/50}$	T_m	T_p	C_R
1	DHI-92	1	17.2	26.8	1.96	3.20	0.35
2	DHI-92	1	18.2	28.3	1.79	2.56	0.32
3	DHI-92	1	18.4	28.7	1.91	2.70	0.31
4	Estramed-93	2	18.0	28.1	1.90	2.45	0.33
5	Estramed-94	3	17.4	27.0	2.26	3.65	0.32
6	Estramed-94	3	20.1	32.0	2.06	2.62	0.31
7	Estramed-94	3	26.1	40.1	1.57	1.69	0.29
8	Estramed-94	4	17.5	27.1	2.39	3.25	0.23
9	Estramed-94	4	16.6	24.2	2.09	2.46	0.14
10	Estramed-94	4	16.9	23.3	1.48	1.55	0.13

Waves on the structures

Incident waves

Data from present tests have been compared with values computed according to Goda (1985) for a foreshore slope 1:100. Data are presented (fig. 3) as a reduction factor, i.e. as a ratio between characteristic values having the same exceeding frequency in offshore and inshore conditions. Our data present some differences from values foreseen by Goda's formulae; this is probably due to the evaluation procedure and to reflection at present tests. However, mentioned differences do not influence the wave height ratios.

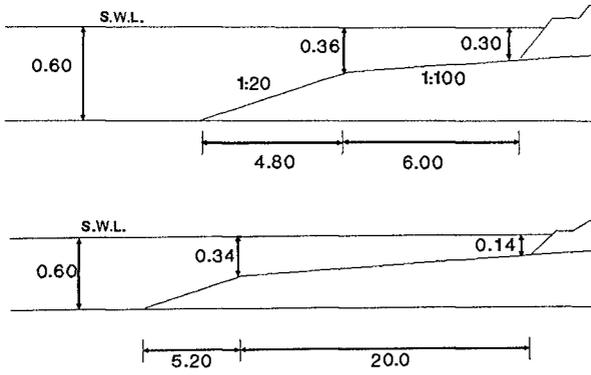


Fig. 1. Bottom conditions at Estramed tests

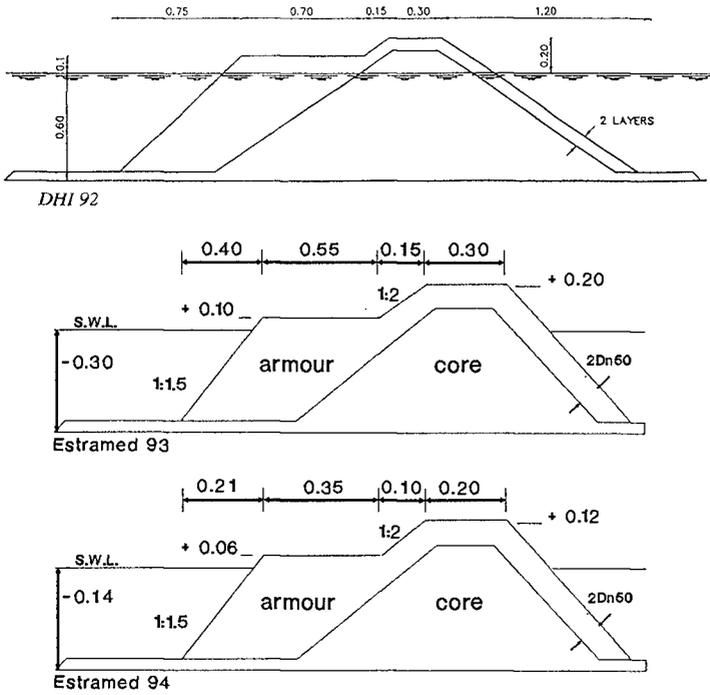


Fig. 2. Cross section of the as built structures (measures in m)

Because inshore wave characteristics were measured at a small but finite distance from the structure, on a water depth larger than at the structure toe, incident wave height on the structure was obtained from measured data applying a small correction which was evaluated according to Goda's formulae. Table 5 gives the observed and calculated inshore wave characteristics.

Wave height distribution

For deep water conditions, in the case of irregular waves with a narrow spectrum (which is a very frequent case in nature and is almost certainly the case for the highest waves affecting the breakwater in deep water conditions) the wave height distribution is of Rayleigh type, or at least it shows a Rayleigh type upper tail of such an extension that the whole third of highest waves fits the Rayleigh distribution well. As a consequence, the ratio between any characteristic wave height and the significant one is a constant. For instance, $H_{1/10}/H_{1/3} = 1.27$, $H_{1/50}/H_{1/3} = 1.55$ and $H_{max}/H_{1/3} = 1.8$.

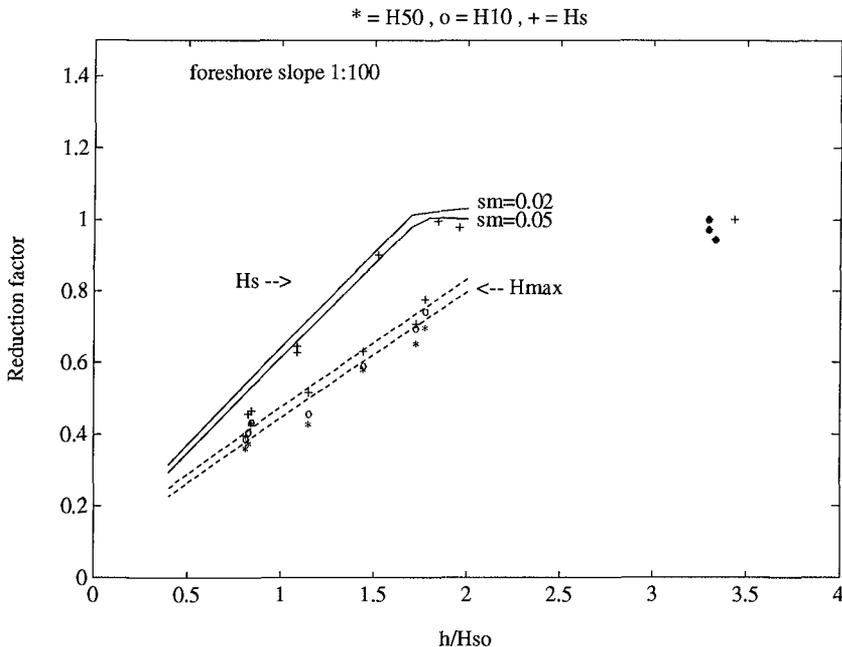


Fig. 3. Wave height reduction factors against relative water depth

In shallow water breaking acts selectively on the highest waves and produces super- and sub-harmonics, broadening the spectrum. For the range of relative water depths of our intermediate and shallow water tests, the ratio between the characteristic wave heights was significantly lower than the mentioned values

derived from Rayleigh distribution. The observed ratios in shallow and intermediate water conditions are:

$$H_{1/10}/H_{1/3} = 1.19 \pm 0.04$$

$$H_{1/50}/H_{1/3} = 1.33 \pm 0.12$$

Table 5. Inshore wave characteristics (measures in m and s)

No.	Test series	h at point of measure	H_s	$H_{1/50}$	T_m	C_R	h at toe	H_s	$H_{1/50}$
1	DHI-92	0.60	0.172	0.268	1.96	0.35	0.60	0.170	0.265
2	DHI-92	0.60	0.182	0.283	1.79	0.32	0.60	0.182	0.282
3	DHI-92	0.60	0.184	0.287	1.91	0.31	0.60	0.182	0.284
4	Estramed-93	0.34	0.134	0.181	1.53	0.53	0.30	0.131	0.177
5	Estramed-94	0.32	0.123	0.176	2.09	0.63	0.30	0.123	0.176
6	Estramed-94	0.32	0.136	0.194	2.05	0.52	0.30	0.131	0.186
7	Estramed-94	0.32	0.134	0.173	1.58	0.51	0.30	0.129	0.166
8	Estramed-94	0.16	0.068	0.113	2.40	0.67	0.14	0.078	0.129
9	Estramed-94	0.16	0.077	0.111	1.94	0.66	0.14	0.078	0.112
10	Estramed-94	0.16	0.077	0.097	1.26	0.56	0.14	0.072	0.090

In shallow water the wave height distribution is distorted compared to the Rayleigh one, and a different distribution must be considered (Glukhovskiy, 1966; Klopman & Stive, 1989). In general no 1-parameter distribution will describe the wave height statistics both in shallow water and in deep water conditions. Only for particular purposes a unique characteristic wave height can be used; for instance, aiming to assess the safety of a brittle structure, the maximum wave height may be considered, as in fact it is normally done for vertical wall breakwaters.

Characteristic wave height

Reshaping of the seaward profile of a berm breakwater is a result of stone movements. During our laboratory investigation, it was observed that their frequency ranged from few single events, when movement starts, to about 50 events in 1000 waves, for the assumed reshaping conditions, producing contemporary movements in more than one place. Therefore we have assumed that a good representative wave height for describing this phenomenon for any water condition is $H_{1/50}$.

The frequency of stone movements is described by the *surface damage level* which is defined as:

$$S = N_d \cdot D_{n50}^2/A \tag{1}$$

representing the probability of moving of a generic stone from a one-grain thick layer of the reshaped profile.

The wave height and wave steepness effects on stone movement frequency (Tomasicchio & al., 1994) along a reshaped profile can be described, both in deep and shallow water conditions, by a unique relation if $H_{1/50}$ is considered (fig. 4). Wave attack intensity is described by the *modified stability number* defined as:

$$N_s^{**} = \frac{H_k}{C_k \Delta D_{n50}} \cdot \left(\frac{s_m}{s_{mk}} \right)^{-1/5} \quad (2)$$

N_s^{**} is a modification of the traditional stability number N_s accounting for the effect of a non-Rayleighian wave height distribution and of wave steepness. If we assume $H_{1/50}$ as the wave height H_k , $C_k = 1.55$. As reference wave steepness we assume $s_{mk} = 0.03$. For this wave steepness and for deep water conditions the values of N_s and N_s^{**} are equal.

An interpretation of eqn. (2) comes from consideration that, in the case of an orthogonal wave attack, the assumptions that onshore wave energy flux is conserved and that waves break as shallow water waves can be translated as:

$$\frac{1}{8} \rho g H_o^2 c_{go} = \frac{1}{8} \rho g H_b^2 c_{gb} \quad (3)$$

where

$$c_{go} = \frac{1}{2} \sqrt{g/k_o}, \quad c_{gb} = \sqrt{g h_b}$$

and

$$H_b = \gamma \cdot h_b \quad (4)$$

They imply:

$$\frac{H_b}{H_o} = \left(\frac{\gamma}{4k_o H_o} \right)^{1/5} \propto \left(\frac{H_o}{L_o} \right)^{-1/5} \quad (5)$$

According to Komar & Gaughan (1972) the best agreement with reality is obtained assuming $\gamma = 1.4$ or the proportionality constant in the last relation = 0.56. The comparison of eqn. (2), (3) and (5) shows that the modified stability number N_s^{**} assumes as relevant wave intensity parameter the onshore energy flux or, if preferred, the breaker height provided the above assumptions hold.

Scale effects

Reynolds number referred to the mound where waves break on is usually defined combining the characteristic length D_{n50} with the typical breaking wave velocity $\sqrt{g H_s}$: $Re = D_{n50} \sqrt{g H_s} / \nu$. With the kinematic viscosity, ν , equal to 10^{-6} m²/s we obtain in our tests $2.1 \cdot 10^4 < Re < 4.6 \cdot 10^4$. Jensen and Klinting (1983) found that in rubble mound breakwater models there is no evident viscosity scale effect if $Re > 0.6 \cdot 10^4$.

Profile dimensions in shallow water

A reshaped or dynamically stable profile for a given wave climate can be described by a small number of geometrical parameters (van Hijum & Pilarczyk, 1982; van der Meer, 1988). Empirically based numerical models to determine such a profile in deep water conditions are available (van der Meer, 1988, 1992 and 1993).

Depth induced breaking cause a reduction in every characteristic wave height and in the dimensions of the developed profile (fig. 4), compared to a breakwater attacked by the same waves in deep water.

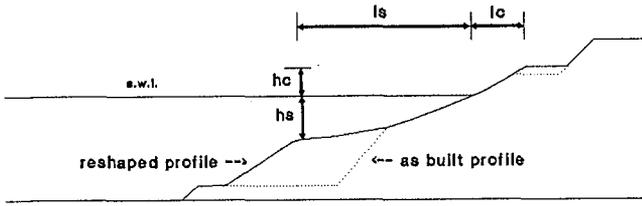


Fig. 4. Parameters for describing the developed profile in shallow water conditions

Accounting for the wave height reduction just in front of the structure is however not sufficient and, as shown first by van der Meer (1988), in order to evaluate the profile parameters, a further reduction factor must be applied.

The relation providing step dimensions suggested by van der Meer (1988) are:

$$h_s = H_{si} \cdot N^{0.07} \cdot 0.22 \cdot s_m^{-0.3} \tag{6}$$

$$l_s = D_{n50} \cdot N^{0.07} \cdot (H_0 T_0 / 3.8)^{1/1.3} \tag{7}$$

Based on very few experimental data, van der Meer proposed the following functional relationship for the reduction factor of the step dimensions, r , depending on the water depth to incident wave height ratio h/H_s :

$$r = 1 - 0.75 \left(2.2 - \frac{h}{H_s}\right)^2 \quad \text{if } \frac{h}{H_s} < 2.2 \tag{8.1}$$

$$r = 1 \quad \text{if } \frac{h}{H_s} > 2.2 \tag{8.2}$$

Comparing the two sets of experimental data, the differences between van der Meer's and our test conditions and analysis procedure must be pointed out, see table 6:

Table 6. Differences in laboratory investigations and in the analyses

	van der Meer tests	our tests
Mobility range	$3.5 < N_s < 12$	$2.9 < N_s < 3.4$
Foreshore slope	1:30	$\leq 1:100$
Shape of the built structure	1:3 uniform slope	berm type
Wave measurements	without structure	with the structure in place
Wave analysis	in the time domain	reflection analysis in frequency domain

Figure 5 shows some of the surveyed profiles. After the first few thousands waves if wave conditions are kept constant, the reshaped profile conserves a poor memory of the built profile. The main trace of the original shape observed in our tests refers to the crest: in fact in our tests the crest was usually not clearly above the original berm elevation; therefore the crest position may not depend only on wave intensity but also on the original shape.

The observation of the profiles from our and from other Authors tests suggests the following comments on the qualitative behavior:

- for a breakwater in shallow water, the reshaped profiles can certainly be described by a reduced number of parameters than described by Vellinga (1986): the step height h_s , the step length l_s , the crest height h_c and/or the crest length l_c ;
- the crest elevation above the original berm was in our tests not evident, not even after a very long reshaping phase; in such case the independent parameters identifying the crest are reduced to only one;
- the step dimensions are reduced in the breaker zone much more than the significant wave height does and therefore a correction factor is necessary;
- the correction cannot be interpreted only as the effect of the reduction of the characteristic wave height ($H_{1/50}$) compared to the significant one, which is not more pronounced than 25%;
- the reduction seems to be due to the change of the kinematic behavior of waves hitting the breakwater: the wave action on the armor stones changes from that of an irrotational green-water breakers plunging on the rubble mound to the action of a white-water roller-shaped bore running up the mound;

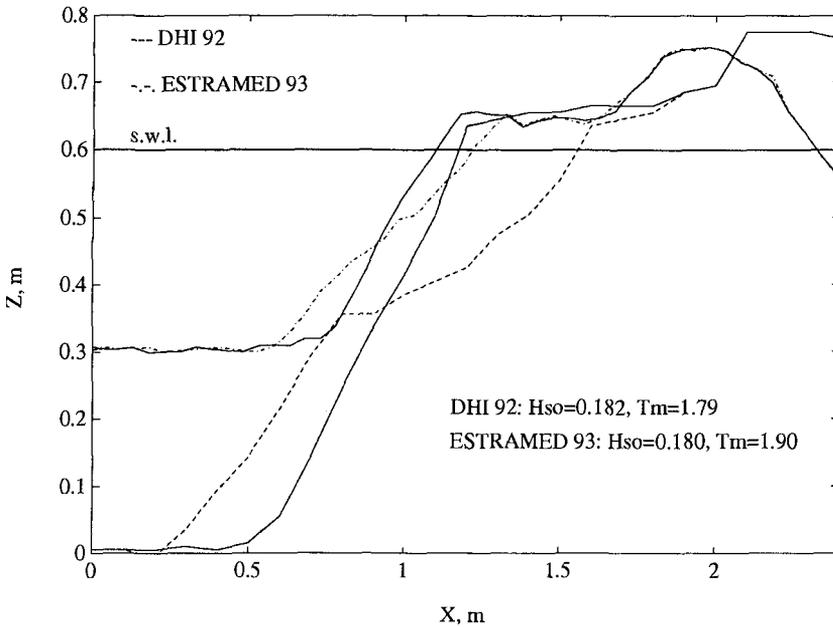


Fig. 5. Surveyed as built and reshaped profiles

- the *corner* separating the step from the lower slope is rather rounded and the mentioned slope is relatively short, therefore sometime the position of the corner is not well defined and the relevance of the slope angle is usually modest;
- the quantification of the dimensions related to stone size is relevant; table 7 gives the observed ratios between typical profile dimensions and the nominal diameter of the armor material.

Table 7 Relative size of step dimensions

h at toe, m	h/H_s	h_s/D_{n50}	l_s/D_{n50}
0.30	2.3 ± 0.1	$4 \div 5$	$12 \div 13$
0.14	1.8 ± 0.1	$3 \div 4$	$8 \div 12$

Table 8 presents the dimensions of the profiles surveyed in our tests at the end of each reshaping phase. The following figures present also results relative to surveys performed during the reshaping progress.

Table 8. Relevant dimensions of the profile at reshaping end

No.	Test series	Repet.	N	h_c	l_c	h_s	l_s
1	DHI-92	1	6000	0.113	0.34	0.310	0.64
2	DHI-92	1	6000	0.128	0.32	0.272	0.66
3	DHI-92	1	6000	0.145	0.34	0.245	0.68
4	Estramed-93	5	5570	0.101 $\pm .006$	0.19 $\pm .02$	0.144 $\pm .009$	0.43 $\pm .04$
5	Estramed-94	1	6000	0.157	0.35	0.103	0.36
6	Estramed-94	1	6000	0.135	0.26	0.107	0.34
7	Estramed-94	1	6000	0.132	0.32	0.100	0.27
8	Estramed-94	1	6000	0.050	0.10	0.050	0.12
9	Estramed-94	1	6000	0.060	0.15	0.063	0.17
10	Estramed-94	1	6000	0.058	0.14	0.053	0.16

In figures 6 data from present tests are reported according to van der Meer's interpretation scheme. Inspection of figs 6 indicates that the data scatter is relatively large and that apparently the relative water depth is not the unique relevant parameter explaining the physical process which results in the reduction of l_s and h_s ; other possible parameters influencing the reduction factor are wave period, foreshore slope and breaking depth and probably the mobility conditions.

Figures 7 show the observed values of the reduction factor against the ratio between the depth at structure toe and the depth at breaking point of the characteristic wave (1/50). The figures indicate that the reduction of profile dimensions l_s and h_s are highly influenced by breaking of the characteristic waves, which, compared to significant wave breaking, occur at a water depth proportionally greater.

Quantitative results show that:

- our reduction factor are more pronounced than those observed by van der Meer;
- the reduction of the step length is more pronounced than of the step height; average limit values in shallow water conditions are around 0.3 and 0.5, respectively;
- test do not show a perfect repeatability even in the same experimental set-up; compare for instance the results of tests 1 to 3 carried out under very similar wave conditions, or the 5 collectively represented as test no. 4, or test no. 7 which reproduce conditions similar to test no. 4 in a different channel;
- part of the scatter in the reduction coefficient may be interpreted as the effect of the imperfection of the formulae which are actually used below the range of mobility conditions in which they were calibrated; the formulae proposed later by van der Meer (1992) for lower mobility are strongly sensitive to the equivalent slope angle α_1 and α_2 , which are not always well defined (for instance for a very wide berm lower than the significant wave height they become both very small, raising the mobility threshold to unrealistic values), and are no more robust than the original ones; the original ones, (6) and (7), were actually used;

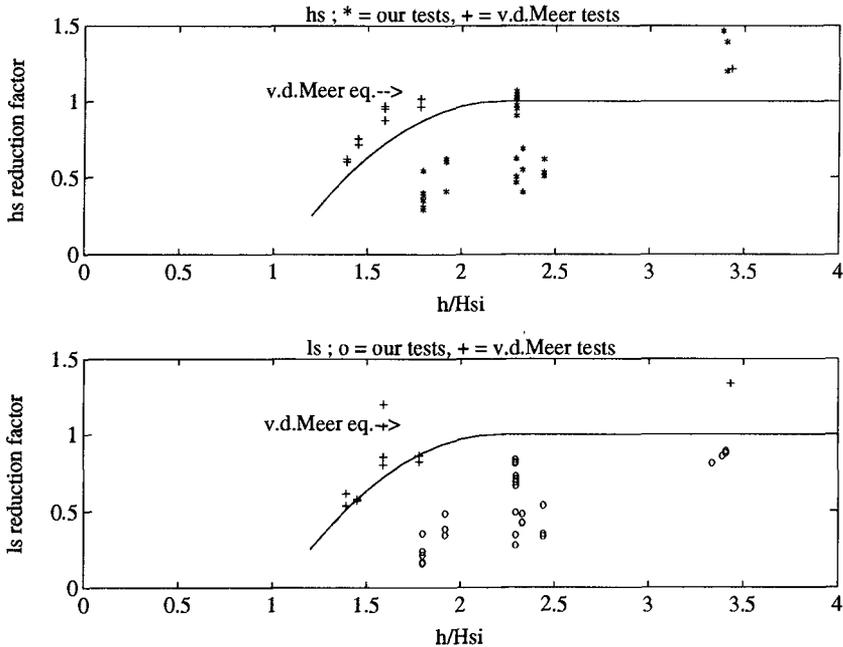


Fig. 6. The reduction factor r of the step dimensions h_s and l_s as function of the water depth to local wave higher ratio h/H_s .

- the effect of the different evaluation procedure of the significant wave height between our and van der Meer tests has been evaluated in some of our cases where

measurements were performed both in presence and absence of the breakwater; the relation $H_s = 4.0\sqrt{m_0}$ is experimentally well satisfied at 0.30 and 0.60 m water depth, whereas at 0.14 m depth the coefficient should be reduced to 3.9; the decomposition of incident and reflected waves by the least square method provides a measure of the accuracy of the assumed model: 95-96% of the total variance is represented in intermediate and deep water conditions, whereas the percentage decreases to 87% in shallow water; the resultant wave height is underestimated by few percentage points (from 2% in intermediate and deep water up to 5% in shallow water); the effect should not be disregarded, but it seems however less important than the control of reflection: i.e., when waves are measured in the absence of the structure, waves reflected by the structure should be accurately absorbed in order to obtain a better estimate of the incident wave height than provided by the spectral reflection analysis.

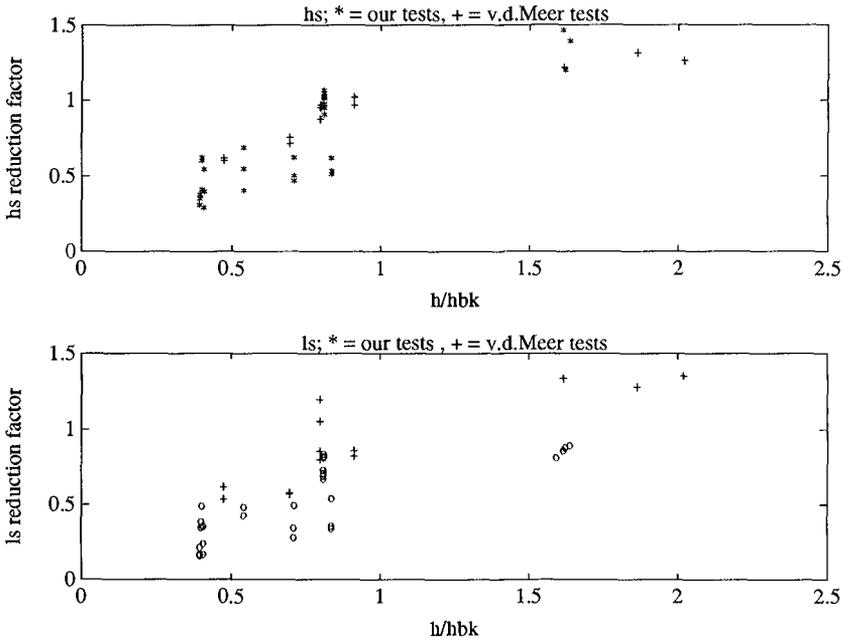


Fig. 7. The reduction factor r of the step dimensions h_s and l_s as function of water depth to breaking depth ratio: h/h_b

Wave reflection at reshaping breakwaters

Reflection coefficients observed in our tests are summarized in table 9. Wave reflection from berm breakwaters in shallow water conditions is rather heavy

($\approx 60\%$) if measured in front of the structure but it is highly reduced when evaluated offshore the breakers. A comparison of at toe and offshore values indicates that in shallow water conditions reflected waves loose approximately 1/3 of their height through the breaker zone.

Table 9. Observed reflection coefficient

Test series	Depth, m	H_s/H_{s0}	C_R at toe	C_R offshore
DHI-92	0.60	1.00	0.32 ± 0.02	0.32 ± 0.02
Estramed-93	0.34	0.74	0.54 ± 0.04	0.31 ± 0.02
Estramed-94	0.32	0.61 ± 0.09	0.62 ± 0.05	0.30 ± 0.02
Estramed-94	0.16	0.42 ± 0.04	0.63 ± 0.06	0.18 ± 0.05

Conclusions

The behavior of reshaping breakwaters in deep and shallow water conditions is qualitatively similar but quantitatively different.

For a reshaping breakwater located in the breaker zone, the profile of the breakwater can be described by a reduced number of parameters.

The step and crest dimensions decrease with H_s , but H_s is not the unique influencing factor, since the dimensions of the step are evidently smaller than those expected according to deep water relations and to H_s incident on the breakwater.

The major cause of the difference between expected and actual dimensions is shown to be the change of kinematics of waves breaking on the structure.

Water depth at the structure toe influences by the breaking process the wave height distribution; its shape changes through the breaker zone. A characteristic wave height, including the effect of limited water depth and representing stone movements in the typical mobility range of breakwaters, is greater than H_s ; good results were obtained adopting $H_{1/50}$.

In particular, consideration of data from present and from van der Meer tests confirms the relevance of the breaking process on the reduction of the profile dimensions, which is well represented by a factor depending on the position of the structure relative to characteristic breakers, quantified as the ratio between the depth at structure toe and the breaking depth of characteristic waves.

Reflection of the breakwater increases with decreasing water depth reaching remarkably high values. Reflected waves are however attenuated through the breaker zone, whereas the encountered incident waves become greater. The offshore apparent reflection coefficient is therefore much lower and decreases with the depth at the structure.

Acknowledgments

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List of symbols

A	= area of the reshaped profile;
C_R	= reflection coefficient;
c_{go}, c_{gb}	= offshore and at breaking group velocity;
$D_{\#}$	= sieve diameter of which # % of the material is finer;
$D_{n\#}$	= nominal (volume) diameter of which # % of the material is finer;
h	= water depth;
h_c, h_s	= crest and step height of the reshaped profile;
H_k	= characteristic wave height;
H_s	= inshore incident significant wave height;
H_{so}	= offshore incident significant wave height;
$H_{1/50}$	= average of the highest 1/50 wave heights;
k	= wave number;
l_c, l_s	= crest and step length of the reshaped profile;
N	= number of waves in a wave attack;
N_d	= number of displaced stones after 1000 waves;
N_s	= stability number: $H_s/\Delta D_{n50}$;
N_s^{**}	= modified stability number;
r	= reduction factor;
R_e	= Reynolds' number;
s_m	= fictitious wave steepness, $\frac{2\pi H_s}{gT_m^2}$;
T_m	= mean wave period;
S	= surface damage level;
Δ	= relative mass density of rock;
ν	= kinematic viscosity of water;
ρ	= water density.

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