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Singular points at berm breakwaters: scale effects, rear, round head and longshore transport

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ABSTRACT

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Design aspects other than the profile development of the seaward side have been investigated in this paper. Aspects such as scale effects, rear stability, round head design and longshore transport have been treated here, based on extensive test series on two different berm breakwater designs. A first conclusion is that scale effects were not present in a 1:35 scale model compared with a 1:7 large scale model with wave heights up to 1.7 m.

A first design rule was assessed on the relationship between damage at the rear of a berm breakwater and the crest height, wave height, wave steepness and rock size. Tests on a berm breakwater head showed that enlarging the berm height at the crest and therefore the amount of rock in the berm was effective with regard to stability.

Finally the onset of longshore transport due to oblique wave attack was studied and compared with literature. Formulae were derived for this onset of transport and also for the range of more serious transport up to longshore transport of coarse gravel.

INTRODUCTION

The berm breakwater concept in its present form is relatively new with regard to the design of traditional rubble mound breakwaters. Severe wave attack on a berm breakwater leads to re-shaping of the seaward slope of this structure. The final profile has an S-shape and is then more stable than the originally built profile. In fact the "as built profile" becomes dynamically stable under severe wave attack and re-shapes into a (more) statically stable profile.

The extensive research of Van der Meer (1988) on dynamically stable slopes, including berm breakwaters, but also rock and gravel beaches, was re-

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analyzed and focussed only on the behaviour of the seaward slope of berm breakwaters in Van der Meer (1992), leading to a computational model on a personal computer. Other basic research on berm breakwater profiles was done by Kao and Hall (1990). However, most literature on berm breakwaters has been focussed on practical applications. In the MAST-project of the European Community, basic research is also focussed on the berm breakwater with the final aim of design rules.

Two practical cases of berm breakwaters were extensively tested at Delft Hydraulics. The more basic aspects of these studies will be treated in depth in this paper and compared with literature where possible. The stability of the seaward slope will not be described, but the singular points such as scale effects, the stability of the rear attacked by overtopping waves, the stability of the head and the longshore transport of material due to oblique wave attack will be investigated.

SCALE EFFECTS

In one particular study a berm breakwater cross-section was tested both in a flume on a scale of 1:35 and in Delft Hydraulics' large Deltaflume on a scale of 1:7. The cross-sections in both facilities are shown in Fig. 1. The berm consisted of 1.7–10 t rock, the berm was just above the water level and the

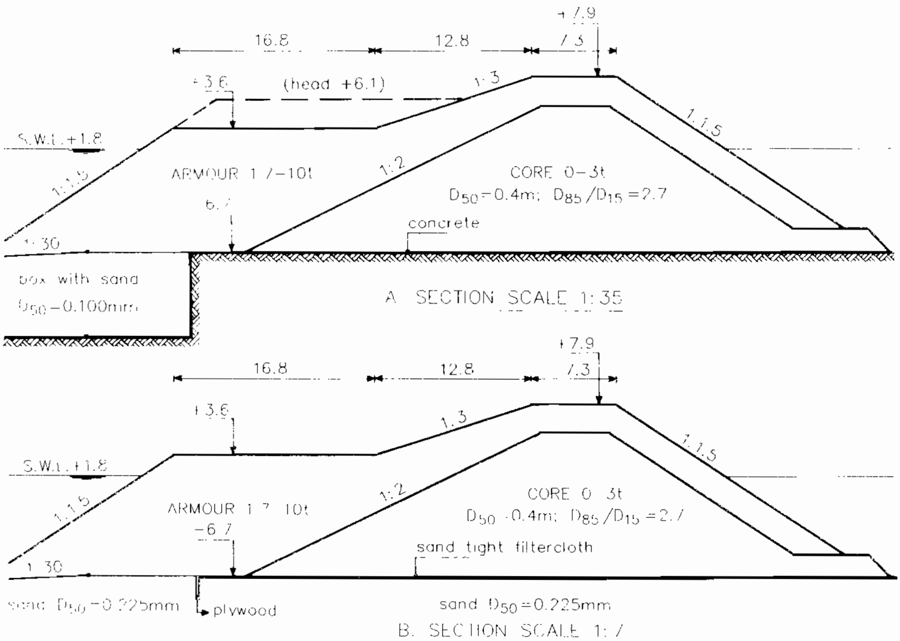


Fig. 1. Cross-sections of a berm breakwater, tested on scales of 1:35 and 1:7.

TABLE 1

Wave boundary conditions for scale effect tests

	At deep water		At structure	Spectrum
Test step	H_s (m)	T_p (s)	H_s (m)	(duration 6 hours)
1	5.09	8.3	4.02	Jonswap
2	7.07	8.5	4.94	Jonswap
3	8.38	11.2	5.64	Jonswap
4	8.83	13.7	5.98	Jonswap
5	10.23	15.6	6.07	Jonswap
6	10.82	17.2	6.10	Jonswap
7	7.36	17.9	4.96	Very narrow
8	7.09	24.4	5.07	Very narrow

berm width was about 17 m. The 0–3 t core was given by $D_{50}=0.4$ m and $D_{85}/D_{15}=2.7$, where D is the sieve diameter. The rock of the berm in the 1:7 scale model amounted to 5–30 kg. A 1:30 sloping foreshore was present in front of the structure. The wave height was depth limited and reached a maximum significant value of about 6 m at the structure and 11 m at deep water.

The structure's berm in the large scale flume was directly placed on 0.225 mm sand. A box with 0.100 mm sand was constructed in front of the structure in the small scale flume. Finer sand was used, scaled more or less to its fall velocity. The omission of a filter layer caused subsidence of rock and this filter layer was placed during the design tests in a wave basin.

The test conditions were identical in both tests and are given in Table 1. The tests consisted of 8 test steps, 6 hours (prototype) each. Various aspects of the two scale effect tests will be compared. These are the profile development, the subsidence of berm rock, and the wave reflection, wave overtopping and wave transmission.

Profile development

The profiles were measured with nine rods and at an interval of one rock diameter. The average of these nine profiles for both tests is shown in Figs. 2 and 3. Figure 2 is taken after test step 5, before the highest waves hit the structure. The seaward profiles are almost identical with small deviations at the upper and lower berm slope. The crest and rear are intact and show almost no deformation. The depth of the scour hole is the same, but its shape is completely different in the seaward direction.

Figure 3 is taken after the final test step. The seaward profile is still similar. The crest and rear both show damage with the most severe damage for the 1:35 test. This damage happened in both cases in the final test step 8. As this is a progressive failure mechanism the difference in displaced amount of rock

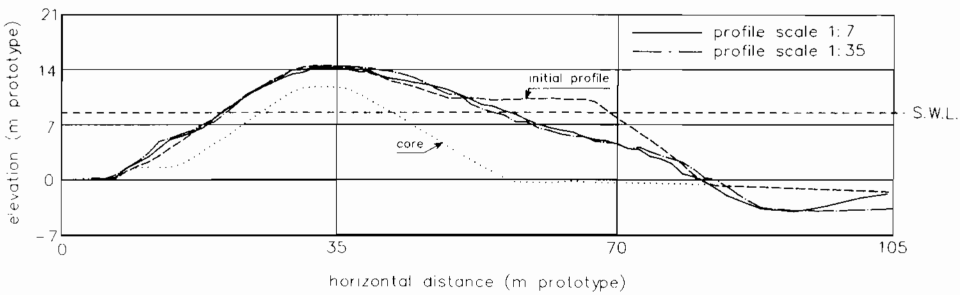


Fig. 2. Comparison of profiles of scale 1 : 35 and 1 : 7 tests after test step 5.

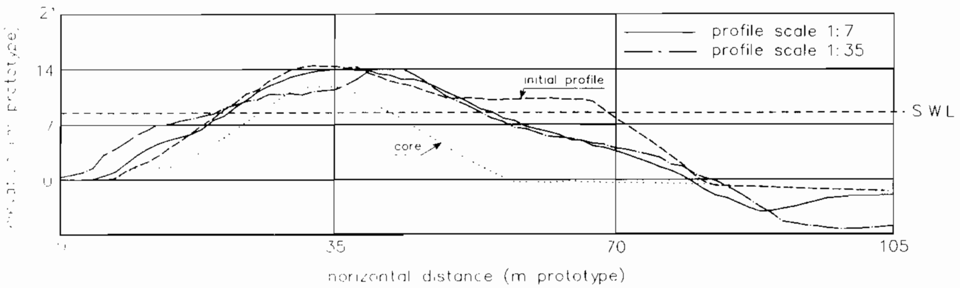


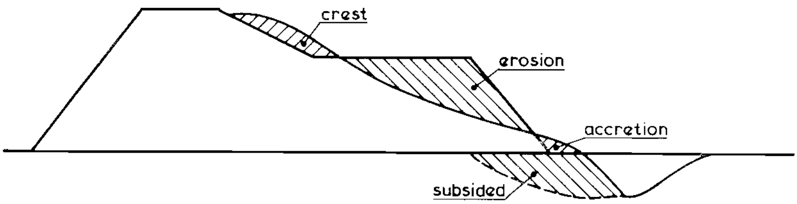
Fig. 3. Comparison of profiles of scale 1 : 35 and 1 : 7 tests after test step 8.

is quite large. The scour hole is deeper in the 1:35 scale model. The difference in behaviour of the scour hole may have led to slightly different wave conditions at the structure.

Based on the comparison of the profiles it can be concluded that seaward slope, crest and rear (except at severe damage) behaved similarly in both tests and that scale effects with regard to these aspects were of no significance. Scale effects were present in the development of a scour hole.

Amount of erosion

Rock in the berm partly moved downward to the toe and partly moved upward to the 1:3 upper slope, see Fig. 4. The main part of the large rock subsided in the sand. The erosion area could be calculated by comparison of the measured profiles before and after the test. Figure 4 gives this erosion in m^3 per m width versus the steps in the test series (see Table 1). Up to step 6 the erosion in both scale models is nearly the same. A difference of only 10% is present after step 8 may be caused by the difference in scour hole. It can be concluded that with respect to the erosion of the berm no significant scale effects were present in the small scale model.



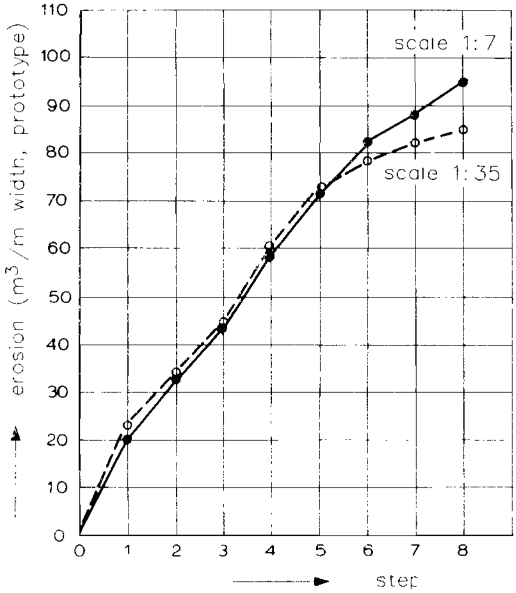


Fig. 4. Comparison of erosion of scale 1 : 35 and 1 : 7 tests.

Wave properties

Besides the structural behaviour of the berm breakwater cross-section, wave properties such as incident waves, wave reflection, wave overtopping and wave transmission can also be compared. Figure 5 shows the measured significant wave heights at deep water, the mean wave period, T_m and the peak period, T_p. The horizontal axis gives the 1:35 scale model and the vertical one the 1:7 scale model. These graphs show that the generated wave boundary conditions were almost identical in both facilities with differences in most cases of only 1%.

The other three graphs in Fig. 5 show the measured wave reflection, wave overtopping and wave transmission. Wave overtopping means the number of waves that reach the crest of the structure and that hit a wave gauge mounted on that location. The number of overtopping waves recorded by this gauge is then related to the number of incident waves, giving a percentage of overtop-

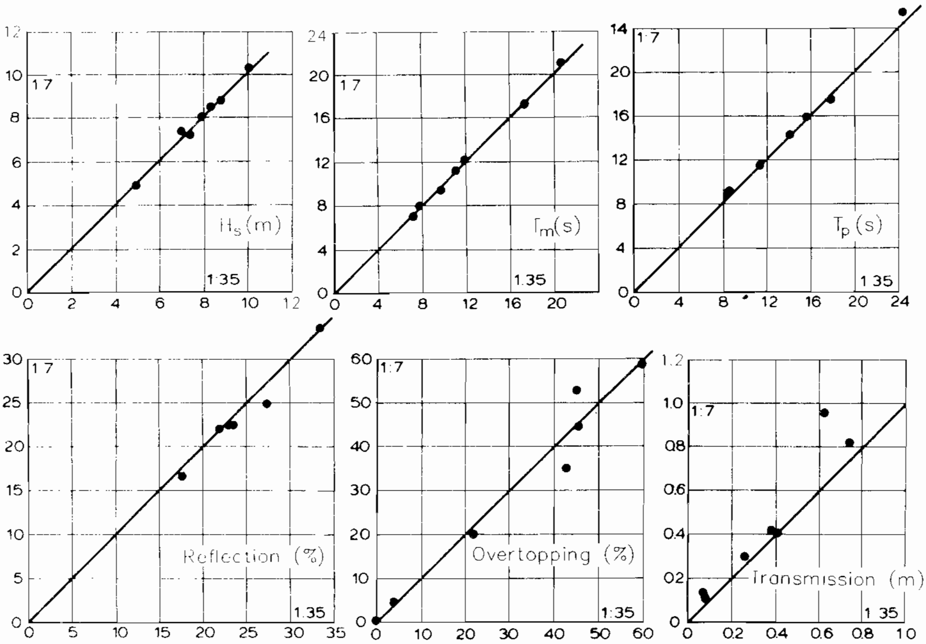


Fig. 5. Comparison of incident wave heights, wave periods, reflection, wave overtopping and wave transmission of scale 1:35 and scale 1:7 tests.

ping waves. The wave transmission is the significant wave height measured by a wave gauge at some distance from the rear of the structure.

Figure 5 shows that reflection is similar in the small and large scale models. The graph with the number of overtopping waves shows more deviation for two of the points. Wave transmission was higher for the 1:7 scale model during all the test steps. Only a small deviation is present for most of the wave heights, but the point with the largest wave transmission (0.95 m) at step 6 differs about 50% from the small scale value (0.63 m).

It can be concluded that no scale effects were present for the wave reflection and the number of overtopping waves. A small but significant higher wave transmission was present in the 1:7 large scale tests, probably partly due to the different flow regimes (turbulent in the large scale test and more laminar in the small scale test) in the armour layer on the crest, but especially in the core material.

Overall conclusion on scale effects

The seaward slope, crest and rear (except for severe damage), and the erosion of the berm behaved similar in both tests and scale effects with regard to these aspects were of no significance. Scale effects were present in the development of a scour hole which may have caused the difference in behaviour of the crest and rear at severe damage. Wave reflection and number of overtopping waves were similar in both tests. Only the wave transmission showed a significant higher value in the large scale model. Based on these conclusions it was clear that the actual design and testing could be based on 1:35 scale model testing in a wave basin.

STABILITY OF THE REAR OF A BERM BREAKWATER

The berm breakwater described in the previous section showed start of damage to the rear at test step 5 (Fig. 2) and showed substantial damage, but not failure, at step 8. Another berm breakwater was tested in a wave basin. The cross-section is shown in Fig. 6. The rock in the berm consisted of 1–7 t or 1–3 t stones. The initial slope of the berm and the upper slope were 1:1.5.

A possible way of comparing berm breakwaters is to use the $H_s/\Delta D_{n50}$ ($=N_s$) number. Here H_s is the wave height at the toe of the structure under design conditions, Δ is the buoyant mass density of the rock and D_{n50} is the nominal diameter of the rock in the berm. D_{n50} is calculated by $(M_{50}/\rho_a)^{1/3}$, where M_{50} is the average mass of the rock and ρ_a the mass density of the rock. In fact the $H_s/\Delta D_{n50}$ value gives the relationship between the wave height and the rock size, including the mass density.

The berm breakwater in Fig. 1 had a maximum $H_s/\Delta D_{n50}$-value of 3.0 and the one in Fig. 6 of 3.1 (for 1–7 t) and 3.9 (for 1–3 t).

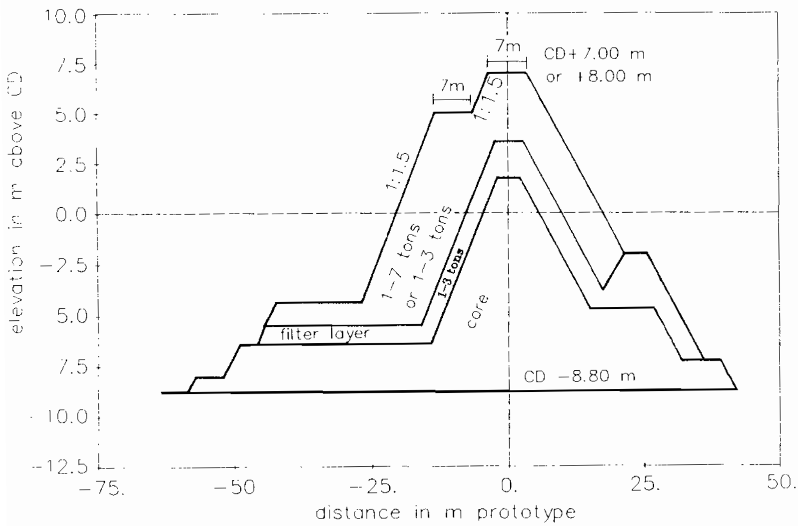


Fig. 6. Cross-section of a berm breakwater tested in a multi-directional basin.

Also from the tests on the breakwater given in Fig. 6, wave conditions could be assessed for which damage started, and moderate and severe damage occurred.

The significant wave height at the toe of the structure (measured during tests where no structure was present), the peak period and the crest height relative to the still water level, R_c, are given in Table 2 for both studies. Also the $H_s/\Delta D_{n50}$-values are given. It is clear that both the wave height and wave period have significant influence on the damage to the rear. This is caused by the fact that a longer wave period gives more overtopping than a shorter period. A combination of a large wave height with a relatively short wave period can cause the same damage as a lower wave height, but with a longer wave period.

The initial cross-sections of both studies were not the same, see Figs. 1 and 6. In both cases, however, the waves were depth limited, the berm reshaped into an S-shape and the upper slope was not eroded. Therefore, it can be concluded that the final profiles after the tests were quite similar.

The analysis of the influence of wave height, period and crest level on the damage to the rear can lead to a practical design formula. As crest height and wave height have the same length dimension, it is logical to use the parameter R_c/H_s as the dimensionless crest height. This parameter is also given in Table 2. The remaining parameter is the wave period. Directly related to the wave period is the fictitious wave steepness, $s_{op}=2\pi H_s/gT_p^2$. This wave steepness is called fictitious as it includes the wave height at the toe of the structure, but the wave length at deep water. In fact, it is the easiest way to use a dimensionless wave period. This wave steepness, s_{op}, is also given in Table 2.

A combination parameter of R_c/H_s with s_{op} to a certain power may lead to similar results for various combinations of wave heights and periods. Based

TABLE 2

Results on stability of the rear of berm breakwaters

Study	H_s (m)	T_p (s)	R_c (m)	R_c/H_s	$H_s/\Delta D_{n50}$	s_{op}	$R_c/H_s * s_{op}^{1/3}$	damage
1	6.1	15.6	6.1	1.00	3.0	0.016	0.25	start
2	4.9	16.0	5.2	1.09	2.5	0.012	0.25	start
2	5.9	13.0	5.2	0.88	3.0	0.022	0.25	start
1	5.1	24.4	6.1	1.20	2.5	0.006	0.21	moderate
2	4.9	16.0	4.2	0.86	2.5	0.012	0.23	moderate
2	5.9	13.0	4.2	0.71	3.0	0.022	0.20	moderate
2	5.6	16.0	4.2	0.75	2.9	0.014	0.17	severe
2	6.6	13.0	4.2	0.64	3.4	0.025	0.18	severe

on the differences in R_c/H_s it is possible to find the optimum value for this power coefficient. The analysis resulted in a coefficient of 1/3, leading to the combination parameter $R_c/H_s * s_{op}^{1/3}$. This parameter is also given in Table 2 and reaches indeed more or less the same values for different wave boundary conditions. Based on Table 2 the following values of $R_c/H_s * s_{op}^{1/3}$ can be given for various damage levels and can be used for design purposes.

$R_c/H_s * s_{op}^{1/3} = 0.25$: start of damage

$R_c/H_s * s_{op}^{1/3} = 0.21$: moderate damage

$R_c/H_s * s_{op}^{1/3} = 0.17$: severe damage

STABILITY OF THE HEAD OF A BERM BREAKWATER

After the scale effect tests in the small and large wave flume, the final check and optimization on the design of Fig. 1 was made in a 10 m wide wave basin and on a scale of 1:35. One of the layouts is shown in Fig. 7. The wave attack

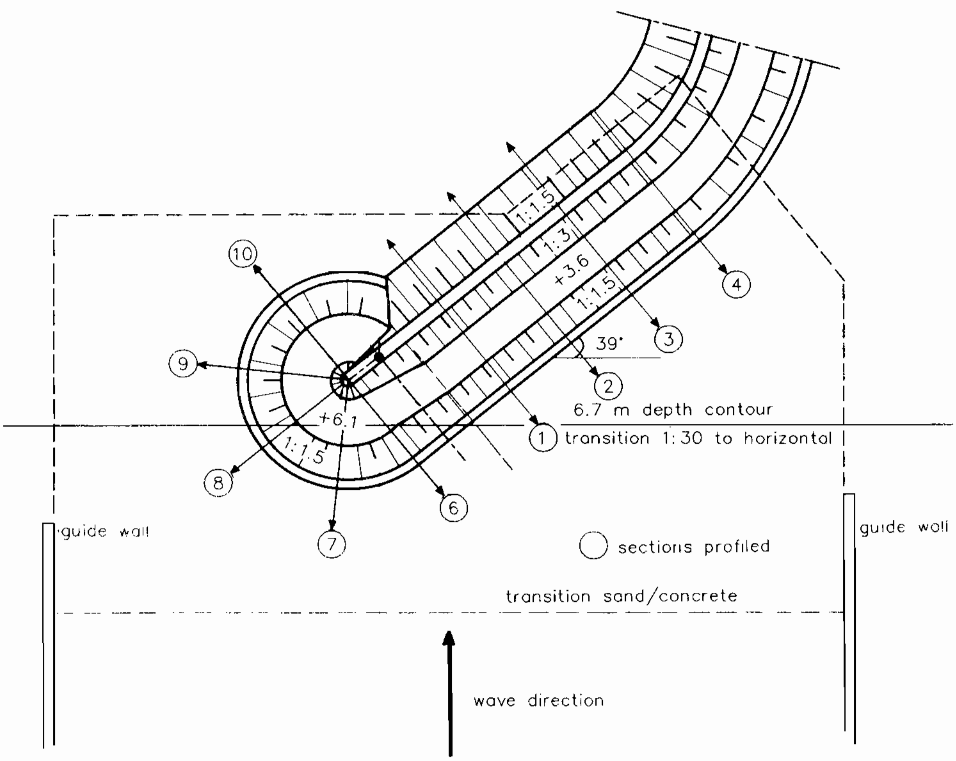


Fig. 7. Lay-out of the berm breakwater tested in a wave basin (cross-sections see Fig. 1).

is directly on the head and has an angle of about 40° on the trunk. The trunk had the same cross-section as given in the upper graph of Fig. 1. The berm elevation was 3.6 m above Chart Datum. The trunk was stable after reshaping.

In order to have more resistance to wave attack on the head, the elevation of the berm of the head was increased to 6.1 m above Chart Datum, giving a larger volume of rock in the berm. The cross-section of the head with elevated berm is also shown in Fig. 1 with the dashed line. Five sections with numbers 6–10 in Fig. 7 give the location where several profiles were taken before, dur-

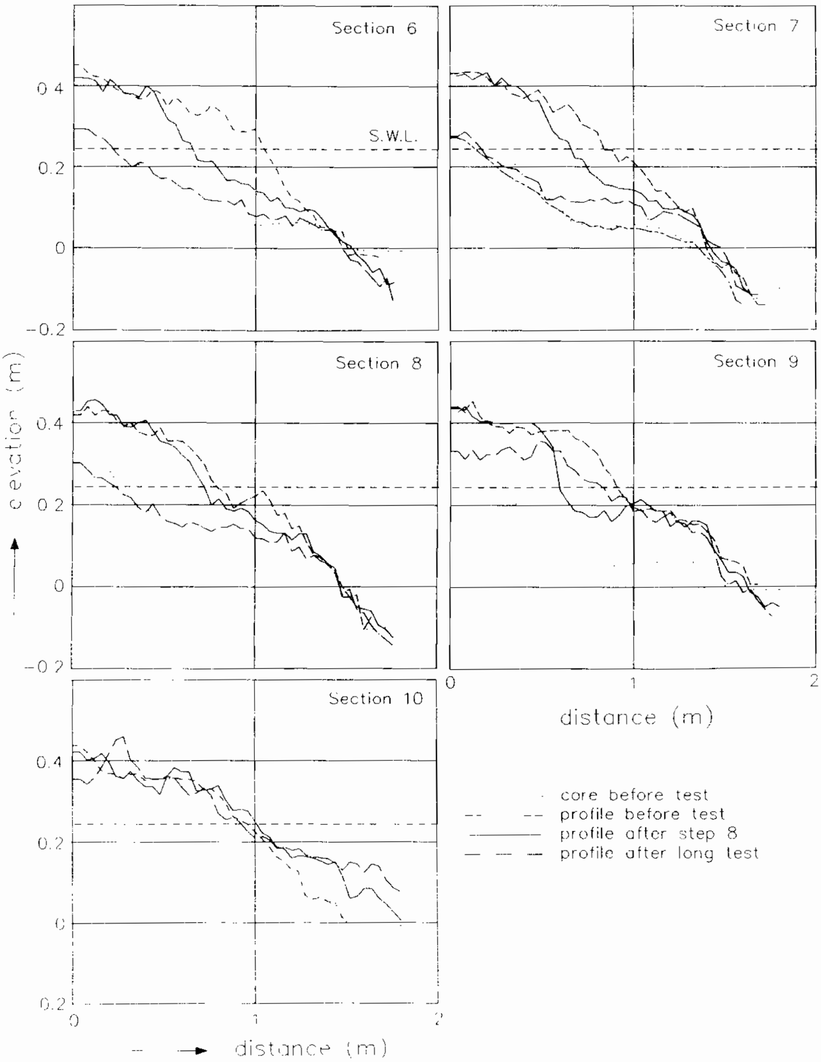


Fig. 8. Cross-sections 6–10 of the head (see Fig. 7) with the profiles after the normal test procedure and after a long duration test.

ing and after the test. A sample in such a profile was taken each 0.04 m in the model, an interval of about one rock diameter. Section 7 had the most severe wave attack.

The same test procedure was followed as described for the scale effect tests and as given in Table 1. Eight test steps of 6 hours each were performed. After this complete test the conclusion on the stability of the round head was that it performed well. The profiles after this test for all sections 6–10 are shown in Fig. 8 by the solid lines. Sections 7–9 show some erosion at the berm around the still water level, but do not show erosion at the crest.

After the normal test procedure it was decided to test the reserve capacity of the head under very severe wave loading. The most severe test step, number 6, with a deep water wave height of 11 m, was run again, but now for a duration of 36 hours, six times longer than in the normal test procedure. The profiles after this condition are also shown in Fig. 8. The erosion at sections 6–9 increased considerably. Even the core became visible at the crest of the round head. Nevertheless the head did not fail in a catastrophic way and survived the extreme wave loading very well.

Figure 9 shows a plan view of the erosion and accretion. It shows the ero-

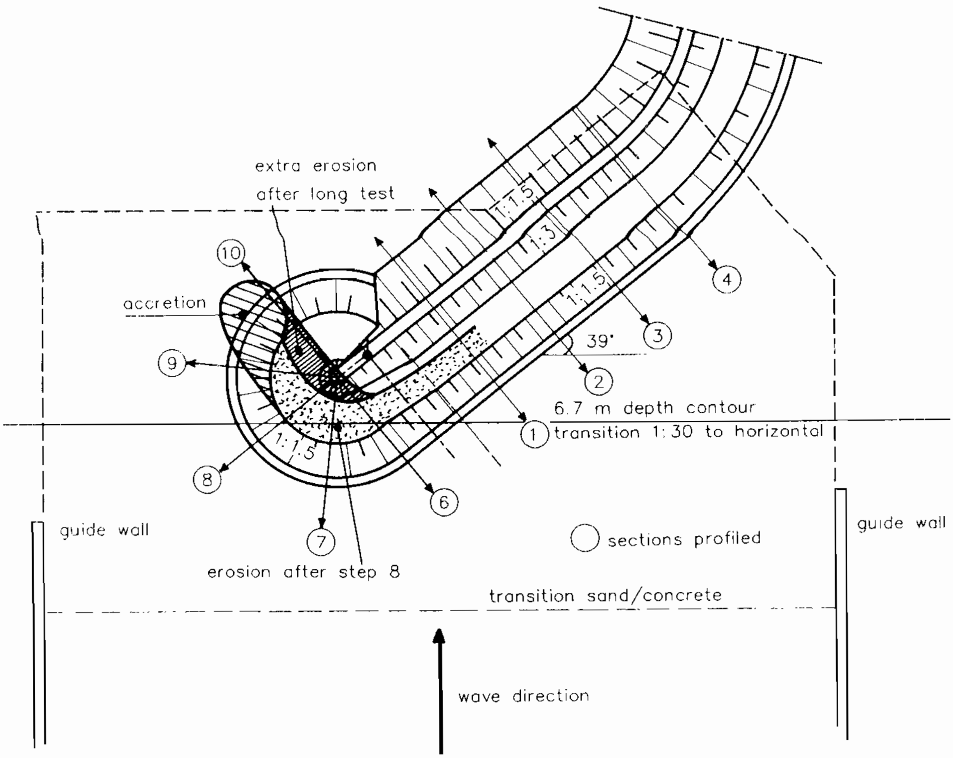


Fig. 9. Plan view of erosion and accretion areas at the head after the normal test procedure and after a long duration test.

sion of the berm at the trunk and the head after the normal test procedure and also the accretion at the inner side of the head at section 10. Besides the transport of material from the berm to the toe of the structure, a part was transported in the wave direction and this caused the accretion at section 10. The severe and long duration test after the normal test procedure showed a considerable increase in erosion at the head (see Fig. 8). The actual amount of eroded material, however, is in fact rather small as can be seen in Fig. 9, where this area is given by the dark shaded part. Although the core became visible after that condition, this was only very locally at the tip of the round head.

The overall conclusion on the stability of the round head was that by increasing the height of the berm and therefore creating a larger amount of rock at the head, can be seen as a good measure for enlarging the stability of the round head of a berm breakwater, using the same rock as for the trunk.

LONGSHORE TRANSPORT OF COARSE MATERIALS

Statically stable structures such as revetments and breakwaters are only allowed to show damage under very severe wave conditions. Even then the damage can be described by the displacement of only a number of stones from the still water level to (in most cases) a location downwards. Movement of stones in the direction of the longitudinal axis is not relevant for these types of structures.

The profiles of dynamically stable structures as gravel/shingle beaches, rock beaches and sand beaches change according to the wave climate. Dynamically stable means that the net cross-shore transport is zero and the profile has reached an equilibrium profile for a certain wave condition. It is possible that during each wave material is moving up and down the slope (shingle beach).

Oblique wave attack gives wave forces parallel to the alignment of the structure. These forces can cause transport of material along the structure. This phenomenon is called longshore transport and is well known for sand beaches. Shingle beaches also change due to longshore transport, although the research on this aspect has always been limited. Rock beaches and berm breakwaters can also be dynamically stable under severe wave action, which means that longshore transport might also cause problems for these types of structures. Therefore the condition of start of longshore transport is important.

The Shore Protection Manual (CERC, 1984) gives the well-known CERC formula for longshore transport of sand. The longshore transport is related to the energy component of the wave action parallel to the coast and the approach is given by:

$$S(x) :: H c_0 \sin 2\beta \qquad (1)$$

where $S(x)$ = material transport rate parallel to the coast (m^3/s), H = wave height (m), c_0 = wave celerity = $gT/2\pi$ (m/s), b = angle of wave attack at the coast, and : : means "proportional to". The longshore transport in this formulation is independent of grain size and is only dependent on the wave condition (wave height, period and direction).

The transport for shingle beaches is determined by bed load (rolling along the bottom) and not by a combination of bed load and suspended load which is the case for sand beaches. Van Hijum and Pilarczyk (1982) have studied longshore transport on gravel or shingle beaches by random wave attack and gave a formula for longshore transport of gravel beaches. Van Hijum and Pilarczyk (1982) used data of Komar (1969) on coarse sand to extrapolate their equation to smaller materials. They concluded that the formula could be applied up to sand beaches.

Van der Meer (1990) reanalyzed the original data and came to a more simple formula for longshore transport of gravel beaches, given by:

$$\frac{S(x)}{gD_{n50}{}^2T_p}=0.0012\frac{H_s\sqrt{\cos\beta}}{D_{n50}}\left(\frac{H_s\sqrt{\cos\beta}}{D_{n50}}-11\right)\sin\beta \qquad (2)$$

The range on which eq. 2 was established was $H_s/\Delta D_{n50}=12$–27, i.e. fairly large gravel in prototype. Figure 10 shows the final results.

Equation 2 shows a dependency on the grain diameter. For small grain sizes, however, the factor 11 in eq. 2 can be deleted and the equation can be rewritten to:

$$S(x)=0.0012\pi H_s c_{op}\sin 2\beta \qquad (3)$$

where c_{op} = the wave celerity $=gT_p/2\pi$. Equation 3 is according to the CERC approach given by eq. 1. The diameter or grain size again has disappeared.

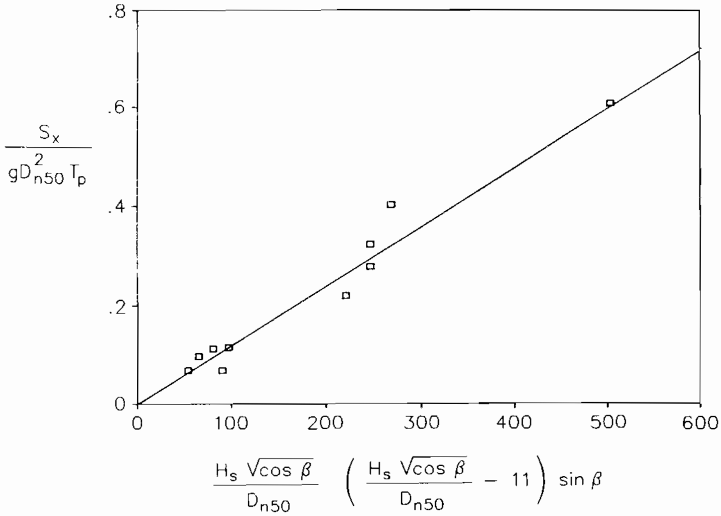


Fig. 10. Longshore transport of coarse materials such as shingle and small rock (no berm breakwaters).

The transition where the grain size no longer has influence can be given by $H_s/\Delta D_{n50} > 50$.

Equation 2 indicates that incipient motion (start of transport) begins when $H_s\sqrt{\cos\beta} > 11 D_{n50}$. This is, however, not correct and gives an underestimation of longshore transport for large diameters, say $H_s/\Delta D_{n50} < 10$. This conclusion was already reached by Burcharth and Frigaard (1987). It means that eq. 2 is not valid for $H_s/\Delta D_{n50} < 10$.

The start of longshore transport is most interesting for the berm breakwater where profile development under severe wave attack is allowed. The berm breakwater can roughly be described by $H_s/\Delta D_{n50} = 2.5$–6. The breakwaters considered in this paper had maximum values between 3.0 and 3.9. Burcharth and Frigaard (1987) performed model tests to establish the incipient longshore motion for berm breakwaters. Their range of tests corresponded to $3.5 < H_s/\Delta D_{n50} < 7.1$. In Burcharth and Frigaard (1988) an extended test series was described. Longshore transport is not allowed at berm breakwaters and therefore Burcharth and Frigaard (1987, 1988) gave the following (as they say, somewhat premature) recommendations for the design of berm breakwaters, which in fact give the incipient longshore motion.

$$\begin{array}{ll} \text{For trunks exposed to steep oblique waves} & H_s/\Delta D_{n50} < 4.5 \\ \text{For trunks exposed to long oblique waves} & H_s/\Delta D_{n50} < 3.5 \\ \text{For roundheads} & H_s/\Delta D_{n50} < 3.0 \end{array} \quad (4)$$

TABLE 3

Test results on longshore transport at berm breakwaters

Present tests						Burcharth and Frigaard (1988)				
		25°		50°				15°	30°	
H_s (m)	T_p (s)	stones/ wave	H_oT_{op}	stones/ wave	H_oT_{op}	H_s (m)	T_p (s)	stones/ wave	stones/ wave	H_oT_{op}
4.9	16.0	0.016	127	0.000	180	0.10	1.5	0.024	0.038	103
4.8	12.0	0.000	93			0.10	2.0	0.047	0.064	137
5.6	13.0	0.005	118			0.13	1.8	0.220	0.125	161
5.9	13.0	0.024	124	0.004	175	0.13	2.5	0.618	0.370	223
5.1	13.0	0.000	107			0.15	1.8	0.724	1.010	185
4.4	16.0	0.000	114			0.15	2.0	0.629	0.551	206
4.9	16.0	0.008	127			0.15	2.2	0.716	1.016	226
5.9	13.0	0.014	124			0.15	2.5	1.051	1.223	257
5.6	16.0	0.061	145	0.016	205	0.15	2.5	0.611	1.513	257
6.6	13.0	0.042	139	0.047	197	0.175	2.5	1.740	1.972	300
5.6	16.0			0.026	205	0.20	2.5	2.756	3.075	343

Analysis of new results on longshore transport

The berm breakwater given in Fig. 6 was tested under angles of wave attack of 25 and 50°. Burcharth and Frigaard (1987, 1988) tested their structure under angles of 15 and 30°. Longshore transport was measured by the movement of stones from a coloured band. The transport was measured for developed profiles which means that the longshore transport during the development of the profile of the seaward slope was not taken into account. The measured longshore transport, $S(x)$, was defined as the number of stones that was displaced per wave. Multiplication of $S(x)$ with the storm duration (the number of waves) in practical cases would lead to a transport rate of total number of stones displaced per storm. Subsequently, the transport rate can be calculated in m^3/storm or m^3/s.

Table 3 gives all the test results on long shore transport, both for the present tests and the tests of Burcharth and Frigaard (1988). Figures 11 and 12 show the same results. From Table 3 it is clear that both a higher wave height and a longer wave period results in larger transport. In Van der Meer (1988, 1991) the combined wave height-wave period parameter H_oT_{op} was used for dynamically stable structures:

$$H_oT_{op} = (H_s/\Delta D_{n50}) * T_p\sqrt{g/D_{n50}} \qquad (5)$$

H_o is defined as the stability number $H_s/\Delta D_{n50}$ and T_{op} as the dimensionless wave period related to the nominal diameter: $T_{op} = T_p\sqrt{g/D_{n50}}$. With the parameter H_oT_{op} it is assumed that wave height and wave period have the same influence on longshore transport. This is slightly different from eqs. 1–3, where the wave height has more influence than the wave period. Figures 11 and 12 give the longshore transport $S(x)$ (in number of stones per wave) versus the H_oT_{op}. Figure 11 gives all the data points. The maximum transport

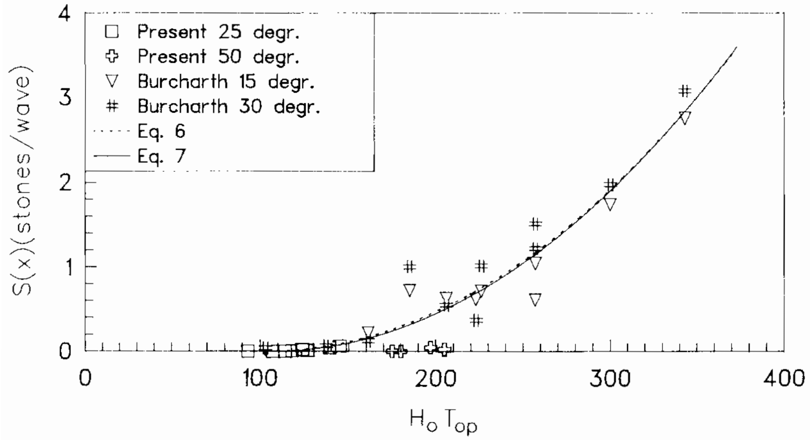


Fig. 11. Longshore transport for berm breakwaters.

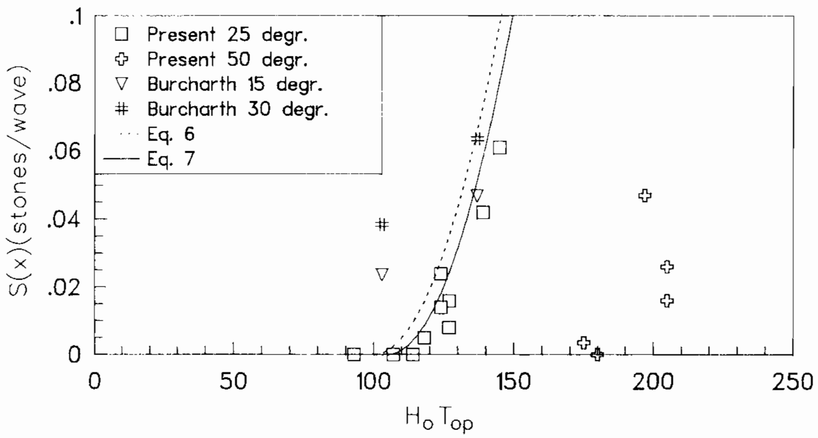


Fig. 12. Onset of longshore transport for berm breakwaters. Detail of Fig. 11 with $S(x) < 0.1$.

is about 3 stones/wave for $H_oT_{op} = 350$, which is in fact a very high rate for berm breakwaters. The $H_s/\Delta D_{n50}$-value in that case was 7.1, considerable higher than the design value for berm breakwaters. Figure 11 also shows that quite a lot of tests had a much smaller transport rate than 0.1–0.2 stones/wave.

Therefore Fig. 12 was drawn with a maximum transport rate of only 0.1 stones/wave. Now only 4 data points remain of Burcharth and Frigaard (1988), the others are from the present tests. Figure 12 shows that the transport for large wave angles of 50° is much smaller than for the other angles of 15–30°. The two lowest points of Burcharth and Frigaard show transport for $H_oT_{op} = 100$, where the present tests do not give longshore transport up to $H_oT_{op} = 117$.

Vrijling et al. (1991) use a probabilistic approach to calculate the longshore transport at a berm breakwater over its total life time. In that case the start or onset of longshore transport is extremely important. They use the data of the present tests and the data of Burcharth and Frigaard (1987), but not the extended series described in Burcharth and Frigaard (1988). Based on all data points (except for some missing data points this is similar to Fig. 11) they come to a formula for longshore transport:

$S(x) = 0$ for $H_oT_{op} < 100 S(x) = 0.000048\ (H_oT_{op} - 100)^2$

$$S(x) = 0.000048 (H_oT_{op} - 100)^2 \tag{6}$$

Equation 6 is shown in Figs. 11 and 12 with the dotted line. The equation fits nicely in Fig. 11, but does not fit the average trend for the low H_oT_{op}-region, see Fig. 12. The equation slightly overestimates the start of longshore transport (except for 2 points of Burcharth and Frigaard). Therefore eq. 6 was adjusted to better describe the start of longshore transport:

$S(x) = 0$ for $H_o T_{op} < 105$

$$S(x) = 0.00005(H_o T_{op} - 105)^2 \quad (7)$$

Equation 7 is shown in Figs. 11 and 12 with the solid line and fits better in the low $H_o T_{op}$-region. The upper limit for eq. 7 is chosen as $H_s/\Delta D_{n50} < 10$, being about the lower limit of eq. 2. With eq. 7 the longshore transport for berm breakwaters has been established for the most severe angles of wave attack. The wave direction of 50° is much more stable than 25° (see Fig. 12). Although a sharp definition can not be given, eq. 7 yields for wave angles between 15–40°. For smaller or larger angles the longshore transport is significantly less.

CONCLUSIONS

Scale effects with regard to the seaward slope, crest and rear (except for severe damage), and the erosion of the berm were of no significance when tests on scales of 1:7 and 1:35 were compared. Scale effects were present in the development of a scour hole. Wave reflection and number of overtopping waves were similar in both cases, only the wave transmission showed a significant higher value in the large scale model.

The following values of $R_c/H_s * s_{op}{}^{1/3}$ can be given for various damage levels to the rear of a berm breakwater caused by overtopping waves and can be used for design purposes.

$R_c/H_s * s_{op}{}^{1/3} = 0.25$: start of damage

$R_c/H_s * s_{op}{}^{1/3} = 0.21$: moderate damage

$R_c/H_s * s_{op}{}^{1/3} = 0.17$: severe damage

The overall conclusion on the stability of the round head was that increasing the height of the berm at this head and therefore creating a larger volume of rock, can be seen as a good measure for enlarging the stability of the round head of a berm breakwater, using the same rock as for the trunk.

Longshore transport depends on the type of structure (sand beach, shingle beach, rock beach or berm breakwater) and the wave climate. The longshore transport for coarser material than sand can be described by the following ranges and formulae:

Gravel/shingle beach:

$H_s/\Delta D_{n50} > 50$ up to sand beaches:

$$S(x) = 0.0012\pi H_s c_{op} \sin 2\beta \quad (1)$$

Rock/gravel beach:

$10 < H_s/\Delta D_{n50} < 50$:

$$\frac{S(x)}{g D_{n50}{}^2 T_p} = 0.0012 \frac{H_s\sqrt{\cos\beta}}{D_{n50}} \left(\frac{H_s\sqrt{\cos\beta}}{D_{n50}} - 11 \right) \sin\beta \quad (2)$$

Berm breakwater:

$H_s/\Delta D_{n50} < 10$, for angles of 15–40°:

$S(x) = 0 \quad$ for $H_o T_{op} < 105$

$$S(x) = 0.00005 (H_o T_{op} - 105)^2 \quad (7)$$

The results of model tests that were described in this paper constitute an important contribution to a better understanding of the physical processes and failure mechanisms. This paper came up with formulae on the behaviour of berm breakwaters. These formulae can be used to make the conceptual design of the cross-section and the head. Nevertheless, it is important that all berm breakwaters to be built in prototype should be tested (in a wave basin), after this conceptual design.

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