Coastal Engineering, 16 (1992) 205–234
Elsevier Science Publishers B.V., Amsterdam

Stability of the seaward slope of berm breakwaters

Jentsje W. van der Meer
Delft Hydraulics, P.O. Box 152, 8300 AD Emmeloord, Netherlands

(Received 24 April 1991; accepted after revision 1 August 1991)

ABSTRACT

Van der Meer, J.W., 1992. Stability of the seaward slope of berm breakwaters. *Coastal Eng.*, 16 (1992) 205–234.

An extensive research on stability of statically and dynamically stable rock slopes and gravel beaches was reanalysed and focussed on berm breakwaters. Berm breakwaters are initially dynamically stable under severe wave attack, but become more or less statically stable after reshaping. The effects on the seaward profile of wave height, period, storm duration, spectral shape, initial slope, rock size, rock shape and grading, water depth and angle of wave attack, are described in a qualitative way. Relationships between these variables and the profile parameters are described, leading to a computer program. This program was verified with independent data of berm breakwaters from various international institutes. The program showed to give a good prediction of the behaviour of the seaward slope of berm breakwaters and can, therefore, be used as a conceptual design tool.

INTRODUCTION

Severe wave attack on a berm breakwater leads to reshaping 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 reshapes into a (more) statically stable profile.

An extensive research on dynamically stable slopes, including berm breakwaters, but also rock and gravel beaches, was described by Van der Meer (1988). That reference is used as basis for this paper. Parts were taken and modified with the attention focussed on berm breakwaters only. Earlier work described the stability of mainly shingle beaches (Van der Meer and Pilarczyk, 1986) and the application of a computational model on a hypothetical case of a berm breakwater (Van der Meer, 1987b).

Dynamically stable profiles can roughly be classified by $H_s/\Delta D_{n50}$ between 3 and 500. Berm breakwaters belong to the category $H_s/\Delta D_{n50} < 4–6$. This paper was therefore focussed on tests of Van der Meer (1988) with $H_s/\Delta D_{n50} < 6$, and with a rather steep seaward slope.

0378-3839/92/$05.00

The paper gives first an overall view of governing variables, followed by the test program, the qualitative analysis of profiles and the development of a computational model. Finally the verification of the model on independent berm breakwater tests is given, based on Van der Meer (1990).

This paper describes only the stability of the seaward slope of berm breakwaters. Other important design aspects, such as scale effects, stability of the rear, design of the head and longshore transport, are described in an other paper (Van der Meer and Veldman, 1992).

GOVERNING VARIABLES

A general list of governing variables for stability of rock slopes and gravel beaches was given by Van der Meer (1988). This list was shortened with respect to dynamic stability and was used to set-up the test program.

The surf similarity parameter, ξ_m, is a function of the slope angle, α, and a fictitious wave steepness, s_m: $\xi_m = \tan\alpha/\sqrt{s_m}$, with $s_m = 2\pi H_s/gT_m^2$. (H_s = significant wave height, g = gravitational acceleration, T_m = average wave period). It is also possible to use T_p (the peak period) instead of T_m. This will be discussed later on. Dynamically stable profiles have varying and curved slopes and cannot be characterized by a slope angle. This means that the dimensionless wave period parameter is given by the fictitious wave steepness s_m (fictitious as H_s is taken at the toe of the structure and the wave length $gT_m^2/2\pi$ at deep water). In fact it gives the influence of a dimensionless wave period, rather than of a wave length.

The water depth in front of the structure determines whether the lowest point of movement is influenced by this depth or that the water depth is large enough to form a profile which is independent on the water depth. A variation in water level (tide) will also have an influence on the position of the profile for given wave boundary conditions. The wave height is again defined as the wave height just in front of the structure, which means that the shape of the foreshore must be taken into account in order to determine this wave height, before profile calculations can be made. It is important to investigate the influence of the water depth just in front of the structure on the formation of the profile. The governing variable can be given by $h(x=\text{toe},t)/H_s$.

The wave height can be limited by the water depth. Water depths should be applied from this depth limited conditions (roughly $h/H_s = 1.2$–2) up to depths where the profile is not longer influenced by changes in depth. Water depths in this case should be larger than the lowest point of the developed profile.

The crest height, R_c/H_s, influences the profile if the crest is relatively low. Generally the range to be investigated should lay between the still water level and the maximum runup, in the order of 1–2 H_s. The stability of the rear of a low crested structure (the rear is attacked by overtopping waves) will not be taken into account. Therefore, the crest width, w_c/D_{n50} or w_c/H_s, will be ignored. It is assumed that the crest is wide enough to avoid damage to the rear.

The angle of wave attack, ψ, has influence on the profile as wave runup, run-down and breaking vary with varying angles of wave attack. Van Hijum and Pilarczyk (1982) have investigated gravel beaches for $\phi=30°$. Their results will be taken into account in this paper. Generally the range should roughly be between $\psi=0°$ and $50°$.

Rock is more or less angular, while gravel (shingle) is rounded. The shape of the material can not be ignored beforehand for dynamically stable slopes. More or less cubical rock, flat and long rock and rounded rock (shingle) give the practical range.

The mechanical strength (or quality) of rock has to be considered in prototype designs, especially for dynamically stable structures with large rock, as berm breakwaters. The quality of the rock is less important for small scale investigation and will not be considered.

Most statically stable profiles are designed as a uniform slope, characterized by the slope angle, cotα. Dynamically stable profiles can not be described by the slope angle. In that case only for model tests a uniform initial slope can be considered and characterized by cotα. In most cases the initial slope will have an arbitrary shape. Most berm breakwaters are described by a broken profile; steep upper and down slope with a horizontal berm above SWL.

Finally, the profile itself can be described by a number of height and length parameters which can be related to the nominal diameter, D_{n50}, or to the wave height, H_s. The nominal diameter is defined by $D_{n50}=(M_{50}/\rho_a)^{1/3}$, where M_{50}=average stone mass, 50% value on mass distribution curve, ρ_a=mass density of rock.

The governing variables for dynamic stability will be treated in more detail in order to establish possible ranges of application.

The wave height parameter $H_s/\Delta D_{n50}$, or stability number N_s, classifies various types of structures, where Δ is the buoyant density. The lower values of $H_s/\Delta D_{n50}$ should be the same as the higher values for static stability ($H_s/\Delta D_{n50}$=2–4). The maximum value for dynamic stability is determined by the smallest possible diameter (D_{n50}=4 mm) and the largest (prototype) wave heights. Assuming H_s=3–4 m and $\Delta=1.7$ the maximum value will be in the order of $H_s/\Delta D_{n50}$=450–600. This paper will concentrate on structures with $H_s/\Delta D_{n50}<6$. Tests in the original work of Van der Meer (1988) concerned the range $H_s/\Delta D_{n50}$=3–260. The range of wave steepness can be set at s_m=0.01–0.06.

The influence of the number of waves on profile development will probably differ from statically stable slopes (the parameter $S/\sqrt{N}$), see Van der Meer (1987a), where S is the damage to a statically stable slope. It can be expected that profile development for small material will occur faster than the development of damage, as the resistance to wave action is much smaller for the

TABLE 1

Final list of governing variables with respect to dynamically stable slopes

Variable	Expression	Range
Wave height parameter	$H_s/\Delta D_{n50}$	3–500
Wave period parameter (wave steepness)	s_m	0.01–0.06
Profile parameters	–	–
Number of waves	N	250–10,000
Initial slope	cotα or arbitrary shape	–
Grading of the material	D_{85}/D_{15}	1–2.5
Shape of the stone	–	angular, rounded, flat
Spectral shape parameter	κ	0.4–0.9
Crest height	R_c/H_s	SWL–runup
Water depth in front of the structure	$h(x=\text{toe},t)/H_s$	–
Angle of wave attack	ψ	0–50°

smaller grains used in dynamically stable structures. Although most of the reshaping of the profile will have taken place after 1000–3000 waves, some long duration tests up to 10,000 waves are valuable. Also measurements after short durations (N=250–1000) should be considered. The possible range of application can roughly be defined as N=250–10,000.

Initial slopes can be uniform or can have an arbitrary shape. A developed profile can even be the initial profile for another wave condition. The grading can be defined between D_{85}/D_{15}=1–2.5. The spectral shape parameter, κ see Van der Meer (1988), is defined by κ=0.4–0.9.

The final list of governing variables for dynamically stable rock slopes and gravel beaches with the possible range of application is given in Table 1.

TEST SET-UP AND PROGRAM

Dynamic stability is defined by the formation of a profile which can deviate substantially from the initial profile. All the changes of the slope have to be taken into account. Dynamic stability can roughly be classified by $H_s/\Delta D_{n50}$> 2–4. A transition area exists between static stability and dynamic stability which is given by $H_s/\Delta D_{n50}$ between 2 and 6.

Tests were conducted in a small scale flume and in the large Delta flume. Both facilities have been described by Van der Meer (1988, Section 3.2). Also the surface profiler described in that section was used to measure the profile developed. The same test procedure was followed as for the tests on static stability. This means that each complete test consisted of a pre-test sounding, a test of 1000 waves, an intermediate sounding, a test of 2000 more waves, and a final sounding.

Crushed rock or shingle was used for the tests. The range of $H_s/\Delta D_{n50}$=

3–13 was investigated with nominal diameters $D_{n50}=0.011$ m and 0.026 m. The largest diameter of 0.026 m gives $H_s/\Delta D_{n50}$-values between 3 and 6 and these tests are interesting with respect to berm breakwaters. Normally, a grading was used with $D_{85}/D_{15}=1.50$. Some tests were performed with gradings with $D_{85}/D_{15}=1.25$ and with 2.25. The wave heights during these tests ranged from $H_s=0.13$ to 0.26 m and the wave periods from $T_m=1.3$ to 3.0 s.

Test program

The research of Van der Meer (1988) on dynamic stability was divided into four parts:

- $H_s/\Delta D_{n50}=3$–13. This range was investigated in the small scale flume. Most governing variables mentioned in Table 1 were investigated in this range. The range with $H_s/\Delta D_{n50}=3$–6 (diameter 0.026 m) is most interesting with respect to berm breakwaters.
- $H_s/\Delta D_{n50}=13$–32. This range was investigated by Van Hijum and Pilarczyk (1982). Tests were performed in the same small scale flume as for the present tests. The influence of oblique wave attack, however, was investigated in a wave basin.
- $H_s/\Delta D_{n50}=7$–21. In this range tests were performed with varying water levels and with storm surges. They will not be described here.
- $H_s/\Delta D_{n50}=25$–250. Large scale tests in the Delta flume were performed on small shingle. This range can only be investigated on a large scale since small-scale investigations would give unacceptable diameters in the order of 1 mm and smaller, for which the fall velocity of the material becomes more important than the diameter. These tests are not described here.

The basic tests to investigate the influence of wave height, wave period, diameter and initial slope on the profile (in the area with $H_s/\Delta D_{n50}=3$–13) are tests 307–341. Tests were performed with crushed rock and not with the more rounded shingle. Two diameters of stone were used: $D_{n50}=0.026$ m for $H_s/\Delta D_{n50}<6$ and $D_{n50}=0.011$ m for $H_s/\Delta D_{n50}>6$. Two uniform initial slopes were investigated, 1:5 (according to Van Hijum and Pilarczyk) and 1:3, which is more interesting for berm breakwaters.

Generally nine tests were performed for each diameter and each slope angle mentioned above. These nine tests can be described as a matrix of wave heights and periods. Three wave heights were performed with for each wave height three different wave periods. Wave heights and periods were chosen in such a manner that series of three tests were present with only one variable.

Summarizing these basic tests, each initial slope (1:5 and 1:3) and each diameter ($D_{n50}=0.011$ m and 0.026 m) was tested with:

$H_s = 0.14$ m	and	$T_m = 1.3$ s,	$T_m = 1.8$ s,	$T_m = 2.5$ s,	
$H_s = 0.18$ m	and		$T_m = 1.8$ s,	$T_m = 2.5$ s,	$T_m = 3.0$ s
$H_s = 0.24$ m	and		$T_m = 1.8$ s,	$T_m = 2.5$ s,	$T_m = 3.0$ s

In total, 35 tests on this aspect were performed.

Further tests were performed to investigate the influence of other variables mentioned in Table 1. First tests were performed with a very narrow spectrum (tests 342–347). The spectrum was described in Section 2.7 and Figure 2.5 of Van der Meer (1988). The $H_s/\Delta D_{n50}$ range was 3–6.

The influence of the shape of the rock was investigated in tests 348–356. Tests were performed with nicely rounded shingle and with flat rock. The ratio of maximum/minimum dimensions was measured of 200 stones and an exceedance curve was established for rounded shingle, angular rock and flat and long rock. These curves are shown in Fig. 1. The $H_s/\Delta D_{n50}$ range was 3–6.

The concept of berm breakwaters was explored and applied by Baird and Hall (1984). The berm breakwater consists of a gentle part above the still water level, a horizontal berm at and a steep slope below the water level (natural angle of repose). In total 16 tests were performed on berm breakwaters (tests 380–395). The upper slope was 1:3 and the lower slope 1:1.5. The

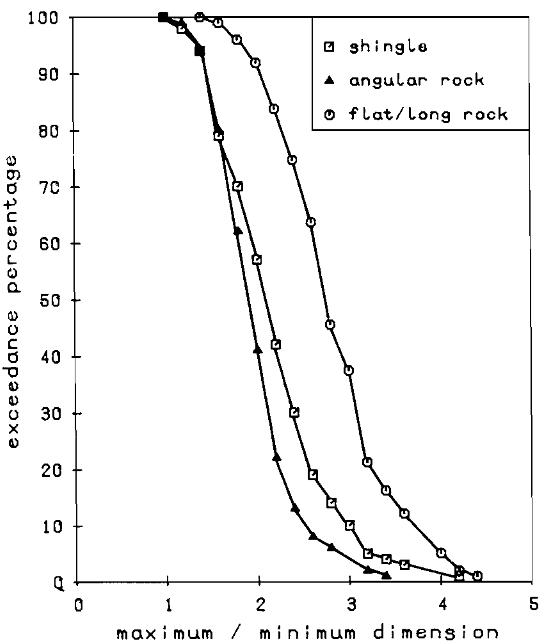


Fig. 1. Shape of shingle, angular rock and flat long rock.

level of the horizontal berm was varied between 0.10 m above, on, and 0.10 m below the still water level. The $H_s/\Delta D_{n50}$ range was 3–6.

In test 396, the technician who built all the models was asked to build an arbitrary initial slope in the way he preferred. This slope was tested in order to verify the model for dynamic stability. The $H_s/\Delta D_{n50}$ value was 4.6.

The grading of the stone was varied in tests 397–408. A narrow grading with $D_{85}/D_{15}=1.25$ and a wide grading with $D_{85}/D_{15}=2.25$ were tested. The $H_s/\Delta D_{n50}$ range was 7–13.

The influence of a low crest was investigated in tests 409–415. The crest level was 0.05 m above the still water level. The crest width amounted to 0.10 m (about 4 diameters) in the first part and to 1.2 m in the second part of the series. The $H_s/\Delta D_{n50}$ range was 3–6.

Finally a foreshore was constructed with a 15 m long slope of 1:30. The water depth at the toe of the structure ranged from 0.20 m to 0.40 m. Waves were breaking on this foreshore with the smallest water depths applied. The tests are described with numbers 416–421. The $H_s/\Delta D_{n50}$ range was 3–13.

A part of the work of Van Hijum and Pilarczyk (1982) consisted of three-dimensional tests with oblique wave attack. Ten tests were performed on a 1:5 uniform slope.

The influence of variation in water level and of storm surges on the profile was investigated during test 360–377, but will not be described here.

The research of Van der Meer (1988) was finished with tests in the large Delta flume. These tests concern stability of small shingle and are not of interest for berm breakwaters. Other tests in this flume, on scale effects at berm breakwaters, are described by Van der Meer and Veldman (1992).

Summarizing the test program, about 120 tests were performed on dynamic stability in the small scale flume (tests 301–421). The research of Van Hijum and Pilarczyk resulted in 42 tests (tests 501–560). Nine tests were performed in the Delta flume (tests 801–809). The tests which are interesting with respect to berm breakwaters only, are the tests with $H_s/\Delta D_{n50}=3$–6 and these are summarized in Table 2.

Recently more basic tests have been performed on stability of dynamically

TABLE 2

Tests of Van der Meer (1988) with $H_s/\Delta D_{n50}=3$–6

Tests	Slope	Remarks
324–332	1:3	basic tests
342–347	1:3	spectral shape
348–356	1:3	round and flat rock
380–396	1:1.5	berm breakwater profile
409–415	1:1.5	low-crested structure
416–419	1:3	limited water depth

stable shingle slopes. Powell (1990) performed 131 tests on single beaches. The following ranges were applied: $D_{n50}=0.01$–0.03 m, $H_s=0.5$–3.0 m and $T_m=3.4$–11.1 s (on a scale of 1:17). This means that the lowest $H_s/\Delta D_{n50}$ value was about 17. In fact a large part of the test program can be considered as a repetition of the tests of Van der Meer (1988). The tests do not describe berm breakwaters as they were solely focussed on shingle beaches.

Kao and Hall (1990) performed basic tests on a berm breakwater. The influence on the profile of wave height, period, spectral shape, number of waves, grading and rock shape were studied. The main conclusions will be repeated in this paper. The $H_s/\Delta D_{n50}$ was varied between 2–5.

ANALYSIS OF PROFILES

A final list of governing variables for dynamically stable rock slopes and gravel beaches was given in Table 1 together with the possible range of application. Comparison of this list with the test program described in the previous section shows that all variables mentioned were investigated, mostly in the complete range indicated.

A first analysis was done by comparing profiles for various tests with only

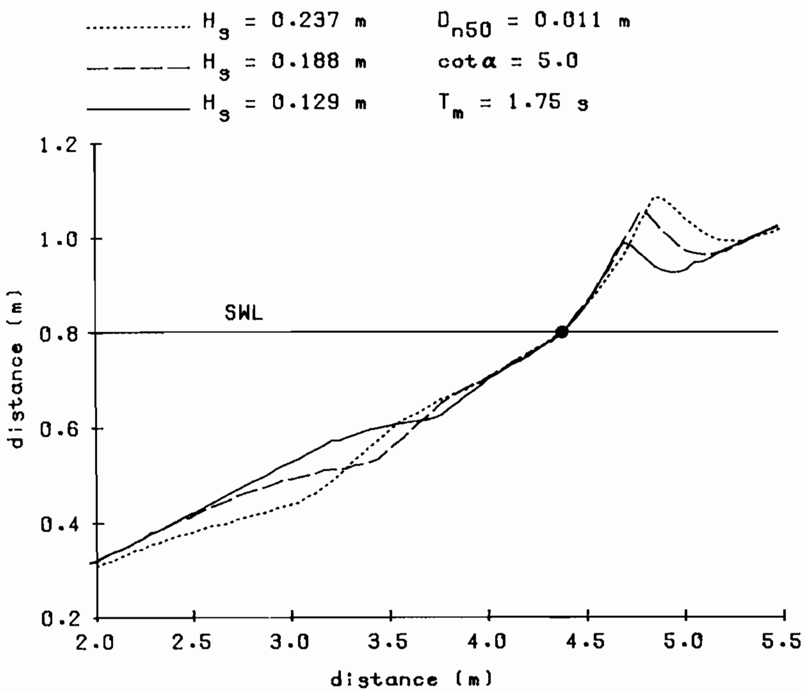


Fig. 2. Influence of wave height.

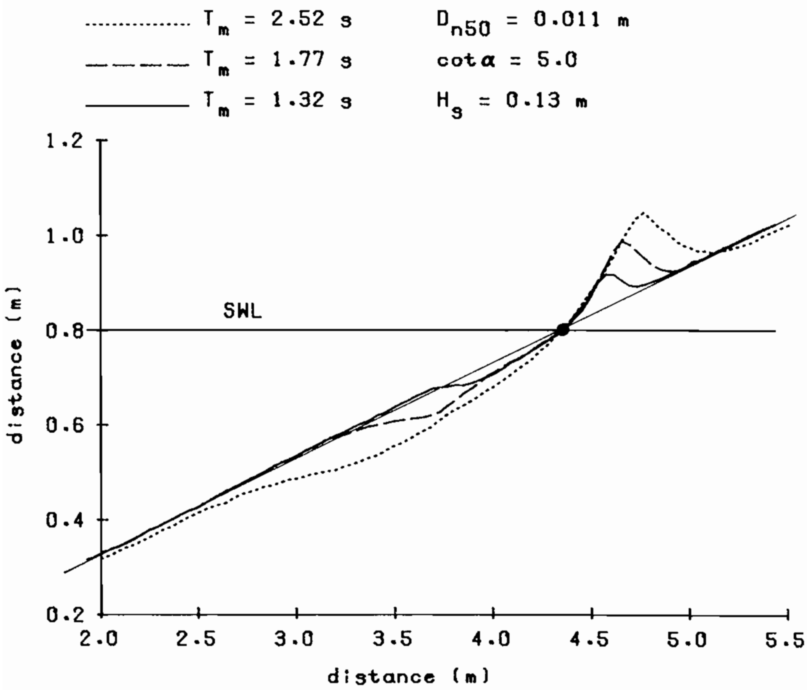


Fig. 3. Influence of wave period.

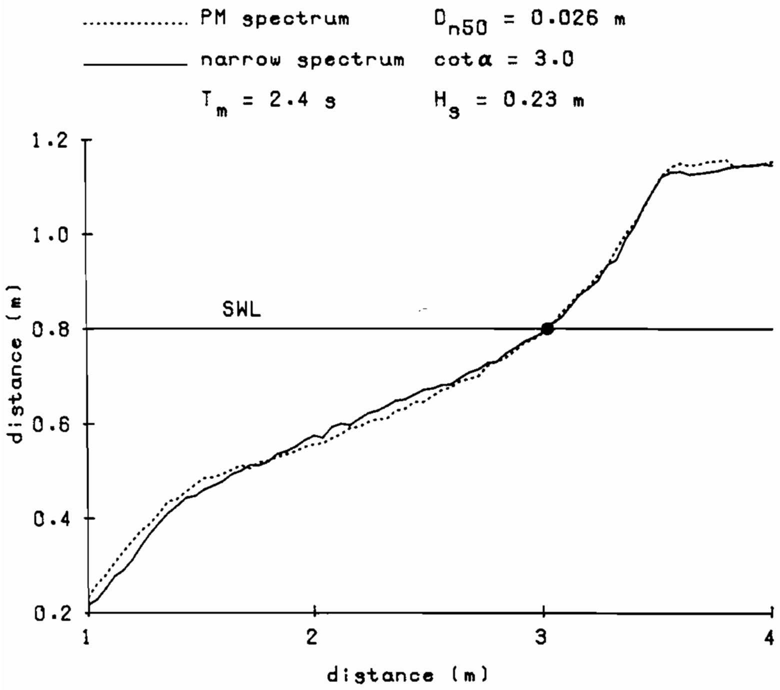
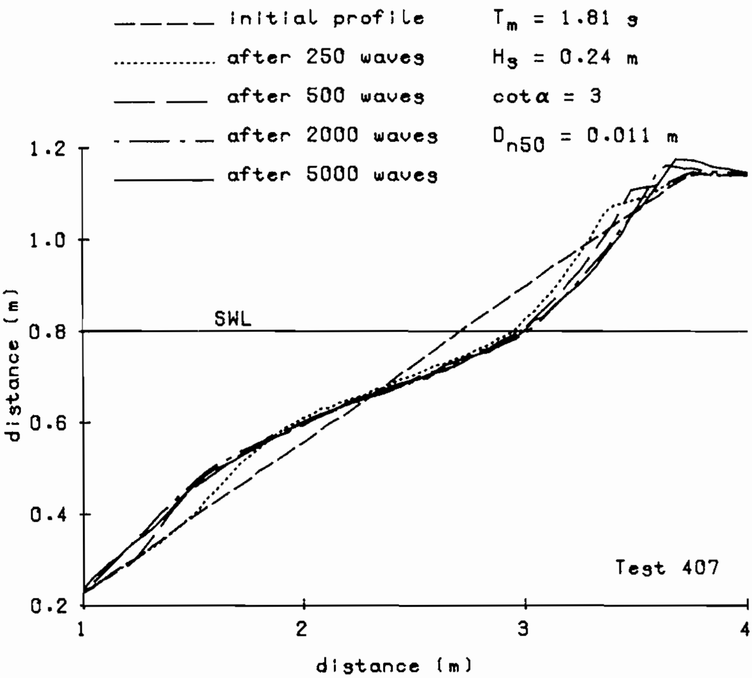


Fig. 4. Influence of spectral shape.



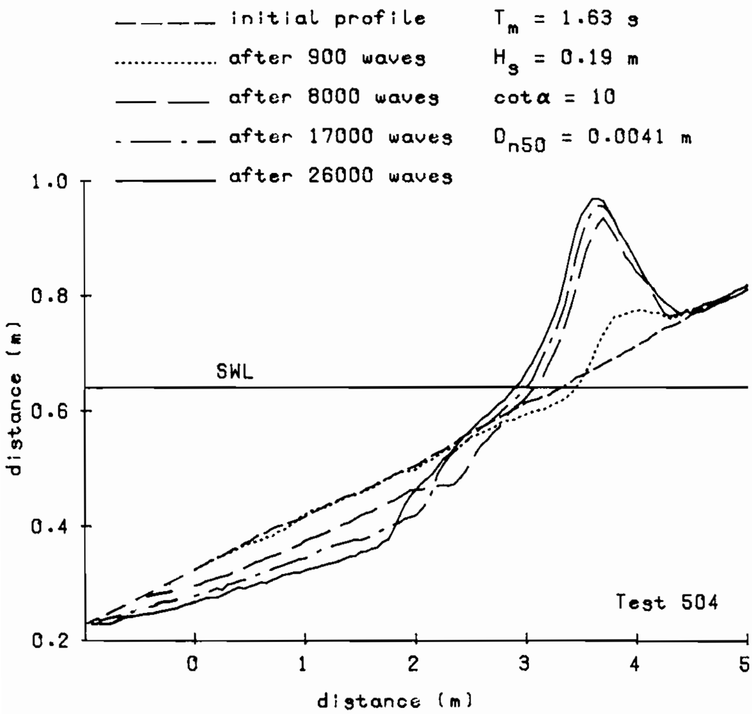


Fig. 5. Influence of storm duration.

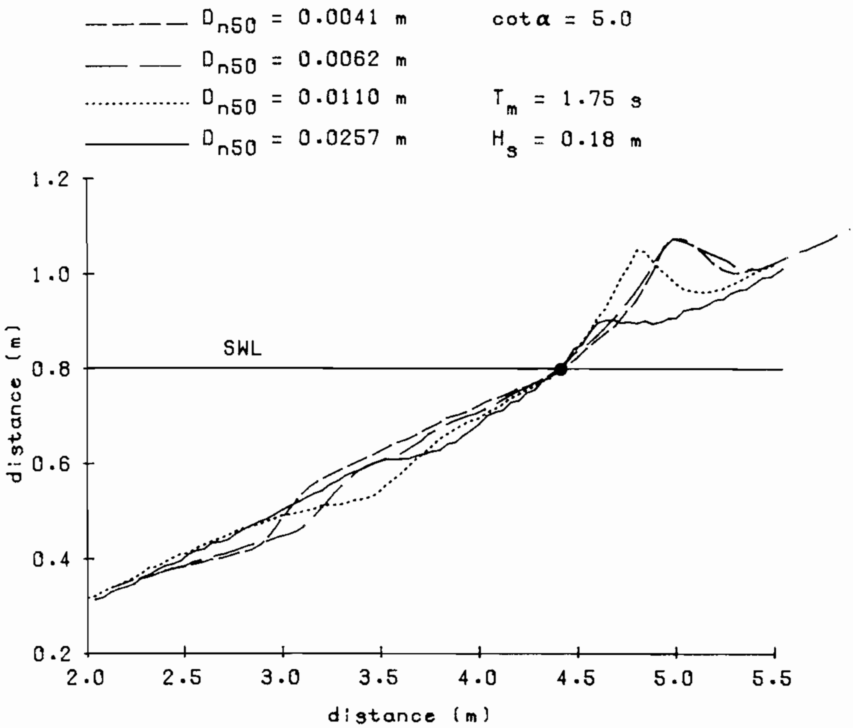


Fig. 6. Influence of diameter.

one variable changed. From this qualitative analysis, conclusions can be derived on the influence of the various variables on the profile. These conclusions were then used to develop a model for dynamic stability.

In fact, for each variable various sets of profiles are available for comparison. Analysis of these sets shows the trend for the variable to be described. In this paper only one set is shown for each variable which characterizes the general trend found for all sets of comparable profiles. Most figures are compared by plotting the profiles at the same intersection with the still water level. This point is indicated by a dot in the figures. Only Fig. 5 is drawn at the original location. All sets of profiles are shown in Figs. 2 to 14.

Influence of wave height and period

Figure 2 shows the profiles measured for three tests (tests 316, 318 and 321). The initial slope was a 1 : 5 uniform slope, the wave period was $T_m = 1.75$ s and the diameter was $D_{n50} = 0.011$ m for all tests. The significant wave heights were $H_s = 0.129$, 0.188 and 0.237 m, respectively; the lowest wave height in

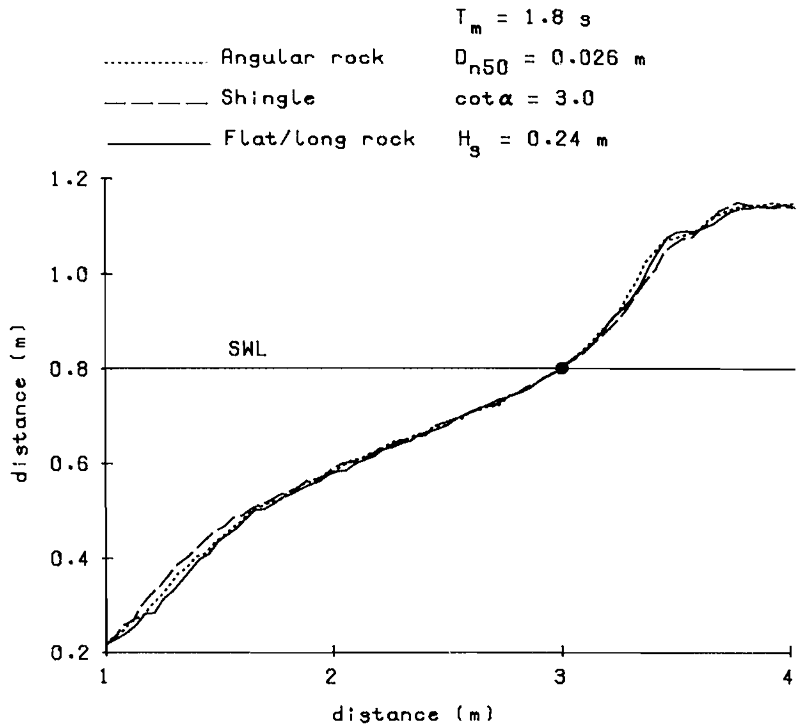


Fig. 7. Influence of shape of stone.

fact produced the smallest changes in the slope. From Fig. 2 it can be concluded that the wave height has a large influence on the profile.

Figure 3 shows the influence of the wave period (tests 315, 316 and 317). The initial slope was again a 1 : 5 uniform slope and the nominal diameter was $D_{n50}=0.011$ m. The significant wave height for all three tests was $H_s=0.13$ m. The wave periods were $T_m=1.32$, 1.77 and 2.52 s; the shortest period in fact produced the smallest changes in slope. A similar conclusion can be drawn as for the wave height namely that the wave period has a large influence on the profile. From Figs. 2 and 3 can be seen that the wave height and wave period have the same order of influence on the profile. Both the wave height (Fig. 2) and the wave period (Fig. 3) were varied by a factor 2 and show in both cases more or less similar variation in profiles.

Kao and Hall (1990) found no significant influence of the peak period on the profile. The influence of the wave period may be less pronounced for low $H_s/\Delta D_{n50}$-values (smaller than 5) than suggested by Fig. 3 which is valid for higher $H_s/\Delta D_{n50}$ values.

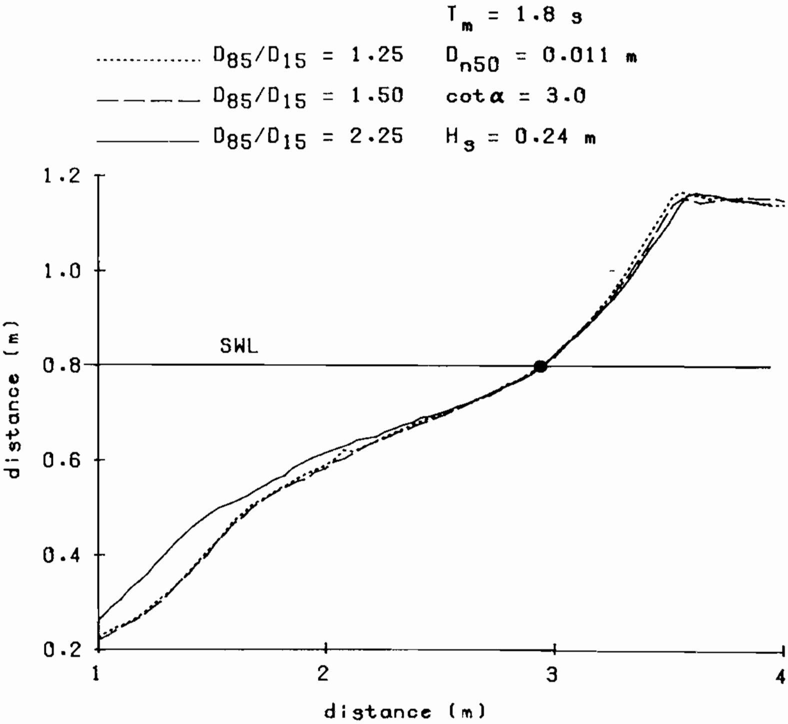


Fig. 8. Influence of grading.

Influence of spectral shape and storm duration

Tests 342–347 were performed with a very narrow spectrum. The profiles of tests 331 (PM spectrum) and 347 are compared in Fig. 4. From this figure it can be concluded that the influence of the spectral shape on the profile is very small.

A comparison was made by using the same average wave period, T_m. From Fig. 3 it was concluded that a longer wave period results in a longer profile. If the same peak period was used for comparison, the PM spectrum would show a larger difference with the narrow spectrum. The narrow spectrum would remain the same as shown in Fig. 4 as $T_m = T_p$ for this spectrum. The ratio $T_p/T_m = 1.15$ for the PM spectrum would result in a less high and long profile than shown in Fig. 4. But also then the difference would be small. On the basis of the tests, it can be concluded that the wave spectral shape has no significant influence on the profile, provided that the average period is used to compare profiles. In that case random waves can be described by the significant wave height and average period only and the spectral shape parameter, κ, can be deleted. A similar conclusion was reached by Kao and Hall (1990), but they

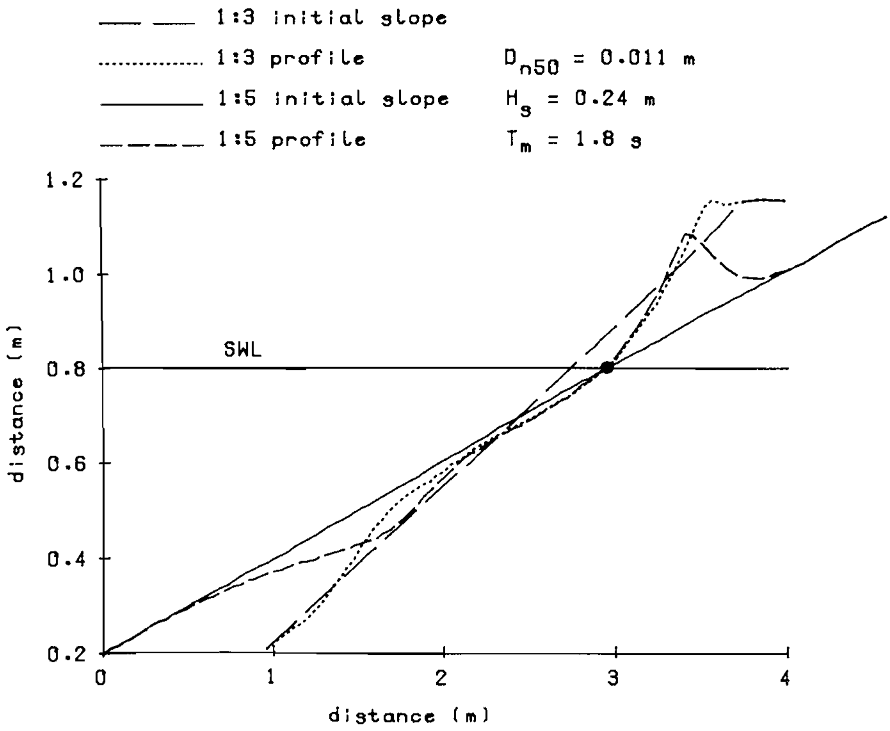


Fig. 9. Influence of initial slope.

used the peak period for comparison. The reason to use the average wave period was not supported by these tests and more research may be required.

Generally profiles were measured after 1000 and 3000 waves. A small number of tests was performed with a longer storm duration and more intermediate soundings. The profiles of test 407 are shown in the upper graph of Fig. 5. Profiles were measured after 250, 500, 1000, 2000, 3000, 4000 and 5000 waves. In this case profiles were not plotted with the same intersection at the still water level, but at their original location. This was done as all profiles belong to the same test. The lower graph in Fig. 5 shows profiles after various storm durations on a very gentle slope of 1 : 10.

From Fig. 5 it can be concluded that a large part of the profile develops within the first few hundred waves. With a longer duration the crest moves up the slope and the profile becomes longer. Even after fairly long wave attack the crest still increases in height. The crest height is largely influenced by the storm duration, as also concluded by Kao and Hall (1990).

Influence of diameter, rock shape and grading

Figure 6 shows the influence of the diameter (tests 309, 318, 375 and 508). The initial slope was a 1:5 uniform slope, the wave height was $H_s = 0.18$ m

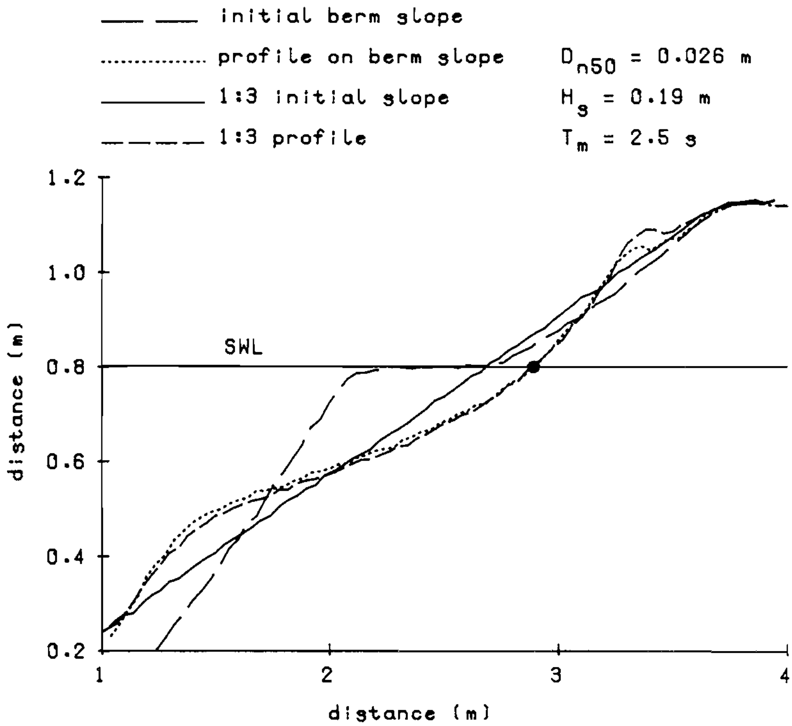


Fig. 10. Influence of initial slope.

and the wave period $T_m = 1.7$ s. The nominal diameters were respectively $D_{n50} = 0.0257$, 0.011, 0.0062 and 0.0041 m. The largest diameters produced the smallest changes in the profile.

From Fig. 6 it can be concluded that the nominal diameter has influence on the profile. For small diameters ($D_{n50} = 0.0062$ and 0.0041 m), however, it can be concluded that some parts of the profile, for example the crest height, are not much influenced by the diameter. The wave runup determines the crest height, more or less independent of the diameter of the material.

Nice rounded shingle, angular rock and flat/long rock were used in different tests to investigate the influence of the shape of stone on the profile. Figure 7 shows the comparison of three profiles with different material shapes.

No difference is found between angular and flat/long rock. The rounded shingle has a tendency to form a lower crest height and a longer berm. The differences are small, however, and it can be concluded that the shape of rock has no or only minor influence on the profile. The same conclusion was reached by Kao and Hall (1990).

Generally a grading was used with $D_{85}/D_{15} = 1.50$. A narrow grading with $D_{85}/D_{15} = 1.25$ was used in tests 397–402 and a wide grading with

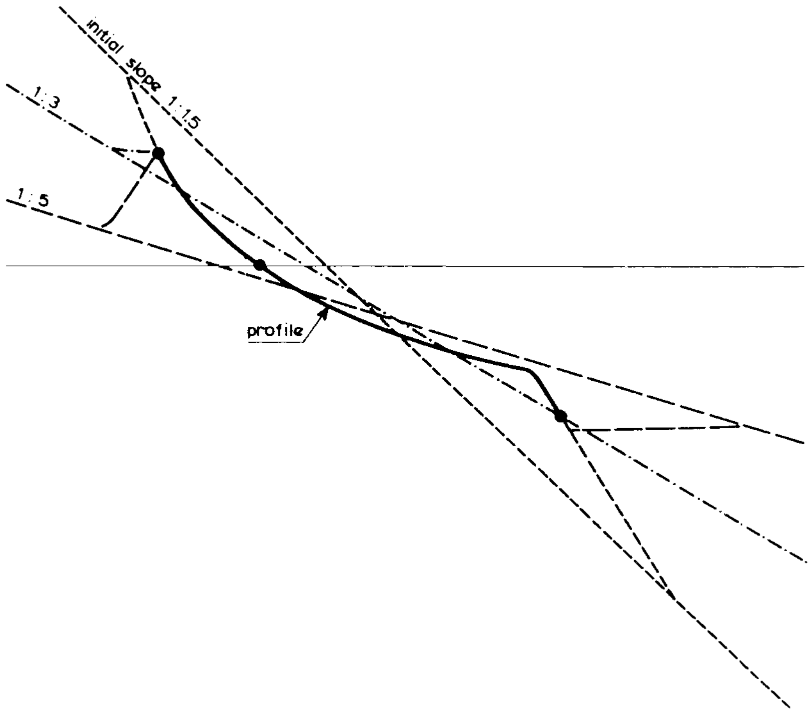


Fig. 11. General influence of initial slope.

$D_{85}/D_{15}=2.25$ in tests 403–408. The profiles found for three different gradings are shown in Fig. 8.

From this figure it follows that the grading with $D_{85}/D_{15}=1.25$ and 1.50 show almost no differences. The wide grading shows the same profile above the still water level, but shows a much longer profile below this level. Therefore the influence of a wide grading on the profile below the still water level cannot directly be ignored.

Kao and Hall tested 4 gradings ranging from $D_{85}/D_{15}=1.35$–5.4. The latter value is a very wide grading. They state: "The length of the profile increased with increasing grading for the first three gradings. This trend, however, reversed for the very wide grading with $D_{85}/D_{15}=5.4$. It appeared that as the width of the grading and thus the maximum stone size increases, the large quantity of stones exceeding a certain upper threshold size has a more dominant effect on stability than does the voids. The presence of these large stones no doubt had a considerable influence over the stability of the armour layer".

Influence of initial slope

In most tests the initial slope was a 1 : 3 or 1 : 5 uniform slope. In other tests a berm breakwater was tested with a 1 : 3 upper slope, a horizontal berm above,

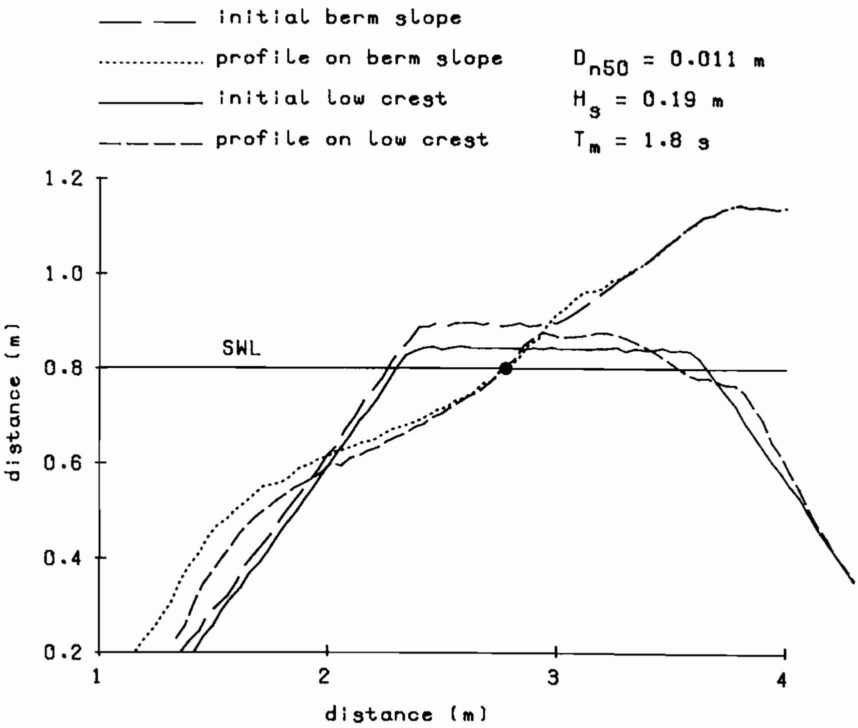


Fig. 12. Influence of crest height.

at, or below the still water level, and a 1 : 1.5 slope for the lower part. Low crested structures were also tested. Figure 9 shows a comparison of two tests with the same boundary conditions, but with different initial slopes. The initial slopes were 1 : 3 and 1 : 5 uniform slopes.

Figure 10 shows the comparison of a 1 : 3 uniform slope and a berm profile. From these figures it can be concluded that in spite of the different initial slopes, the same profile is reached between the crest and the transition to a steep slope (the step) at the deep water end of the profile.

Figure 11 shows a 1 : 5, a 1 : 3 and a 1 : 1.5 uniform initial slope with the developed profiles.

In fact only the upper and lower parts of the profile depend on the initial slope (the dotted lines). The largest part of the profile is the same for all three initial slopes in this indicative figure. The direction of transport of material and the position of the profiles with regard to the initial slope is, of course, largely influenced by the initial slope. The 1 : 1.5 initial slope shows only erosion around the still water level with material transported downwards. The 1 : 3 slope shows material transported upwards and downwards. The 1 : 5 initial slope shows only erosion below the still water level and the material is

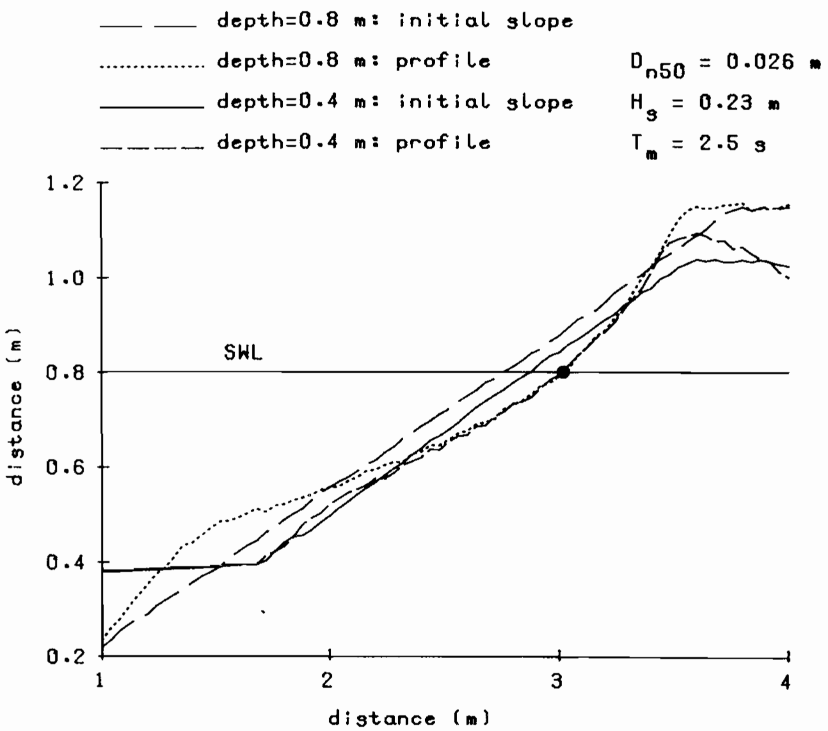


Fig. 13. Influence of water depth.

transported upwards. The profile for the 1 : 1.5 slope is also more or less the same as for a berm breakwater which has a steep seaward slope.

Influence of crest height and water depth

A low crest was investigated in tests 409–415. Tests 409–412 had a small crest width and the rear of the structure was attacked by overtopping waves. The crest disappeared below the still water level and the results can be compared with those of Ahrens (1987) for reef type structures.

Tests 413–415 were performed with a wider crest. Figure 12 shows the comparison of a test with a berm profile and a test with a low crest. A large part of the profile is the same, although the berm profile shows a higher crest and a longer berm. The wave height was also a little higher for the berm profile (0.19 against 0.18 m), however.

Still the same conclusion can be drawn as for the influence of the initial slope. The initial slope (and therefore the crest height) has no or minor influence on a large part of the profile, provided that the crest is wide enough to avoid wave attack at the rear.

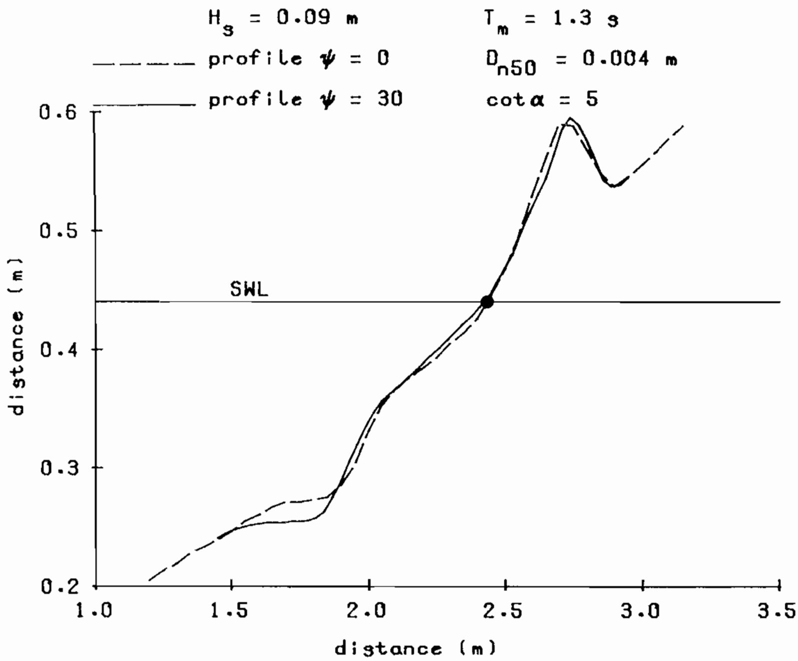


Fig. 14. Influence of angle of wave attack.

A 1:30 uniform foreshore was applied in tests 416–421. The water depth in front of the structure ranged from 0.20 to 0.40 m, causing breaking waves on the foreshore for the smallest water depth due to depth limitations.

Figure 13 shows the comparison of a long 1:3 uniform slope with a short 1:3 uniform slope on a foreshore. A large part of the profile is the same. The length of the profile below the still water level decreases, however, when the length of the slope (or the water depth) is decreased. The effect of a foreshore on the profile below the still water level can not be ignored.

Influence of angle of wave attack

The tests of Van Hijum and Pilarczyk (1982) included both perpendicular wave attack and oblique wave attack ($\psi = 30°$). Two tests are compared in Fig. 14. From this figure it can be concluded that the profile becomes shorter with oblique wave attack. Van Hijum and Pilkarczyk concluded that profile parameters should be reduced by $\sqrt{\cos\psi}$. Re-analysis of the tests with the parameters described in the next section, however, showed that $\cos\psi$ was a better reduction factor.

DEVELOPMENT OF COMPUTATIONAL MODEL ON PROFILE FORMATION

Parameterisation of the profile

Static stability depends largely on the initial slope, as is clearly expressed by the well known Hudson formula. Only little damage, or none at all, is allowed in that case. Of course, for dynamically stable structures which are almost statically stable, the initial slope has also influence on the profile. These structures exceed the limit of "severe damage" for statically stable structures and a clear S-shaped profile is reached. After this reshaping, the structure is more or less statically stable again, due to its more favourable shape to withstand the wave attack. It can be stated that, for $H_s/\Delta D_{n50} = 10-15$, the initial slope has some influence on the profile and that for $H_s/\Delta D_{n50} < 10$ the initial slope has a large influence on the profile. For $H_s/\Delta D_{n50} > 15$ the initial slope has no influence on a large part of the profile.

From the analysis in the previous section it was concluded that the influence of the spectral shape on the profile, can be described by the significant wave height, H_s, and average period, T_m, only. The same conclusion has been found for the influence on stability of statically stable structures (Van der Meer, 1988). No substantial difference was found for various shapes of rock, and it can be concluded that the shape of the rock has no influence on the profile. The grading of the material also has only minor or no influence on the profile, using the nominal diameter, D_{n50}, as reference. Only for very wide gradings a longer profile was found.

A structure with a low crest can be considered as a structure with a non-uniform slope. As already concluded, the initial slope has no influence on a large part of the profile and therefore the influence of a low crest is negligible.

Therefore, the number of governing variables given in Table 1 can be reduced. By virtue of above mentioned conclusions, the following dimensionless variables can be ignored:

- The initial slope (for $H_s/\Delta D_{n50} > 10-15$), but not for berm breakwaters
- The grading of the material, D_{85}/D_{15}
- The shape of the rock
- The spectral shape parameter, κ
- The crest height, R_c/H_s (as no severe overtopping is allowed).

From the qualitative comparison of profiles in the previous section it was concluded that the wave height, H_s, wave period, T_m, the number of waves, N, and the nominal diameter, D_{n50}, all influence the dynamic profile. The water in front of the structure has influence only on the part below the still water level. Finally the angle of wave attack, ψ, influences the profile. The final list of governing dimensionless variables can then be given by:

- The wave height parameter, $H_s/\Delta D_{n50}$
- The wave period parameter (steepness), s_m

– The number of waves, N
– The water depth in front of the structure, $h(x,\mathrm{toe})/H_s$
– The angle of wave attack, ψ
– The profile parameters

On the basis of the conclusions described above a schematized model was developed which describes the dynamically stable profile. Two points on the profile are very important. These are shown in Fig. 15, where profiles for a 1:3 and 1:2 uniform slope are illustrated schematically. The first point, situated above the still water level, is the upper point of the beach crest. The second point, situated below the still water level, is the transition from the gentle sloping part to the steep part.

Figure 16 shows the schematized model for a dynamically stable profile on a gentle initial slope. A 1:5 uniform initial slope is shown with a high beach crest and a step. The profile is schematized by using a number of parameters all of which are related to the local origin or to the water level. The beach crest is described by the height, h_c, and the length, l_c. The transition to the step is described by the height, h_s, and the length, l_s. Curves, described by power functions, start at the local origin and go through these two points. The runup length is described by the length, l_r. The step is described by two angles, β and γ. Finally, the transition from β to γ is described by the transition height, h_t. This transition is not present for steep initial slopes as for berm breakwaters.

Summarizing, the schematized dynamically stable profile is defined by:

– The runup length, l_r
– The crest height, h_c
– The crest length, l_c
– The step height, h_s

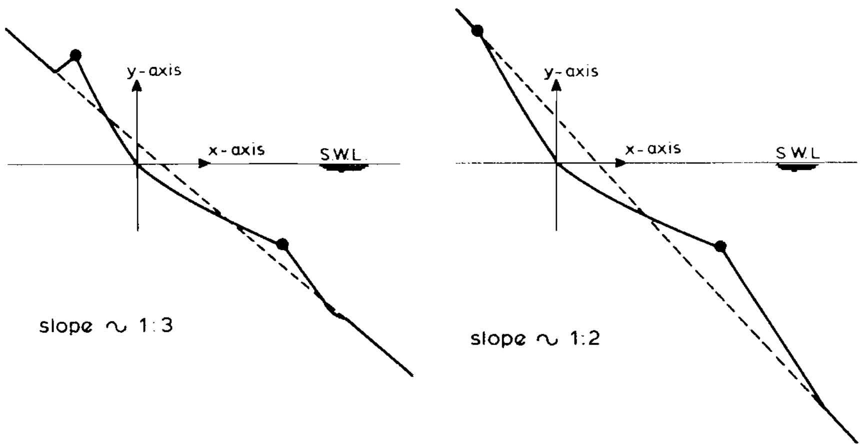


Fig. 15. Schematized profiles on 1:3 and 1:2 initial slopes.

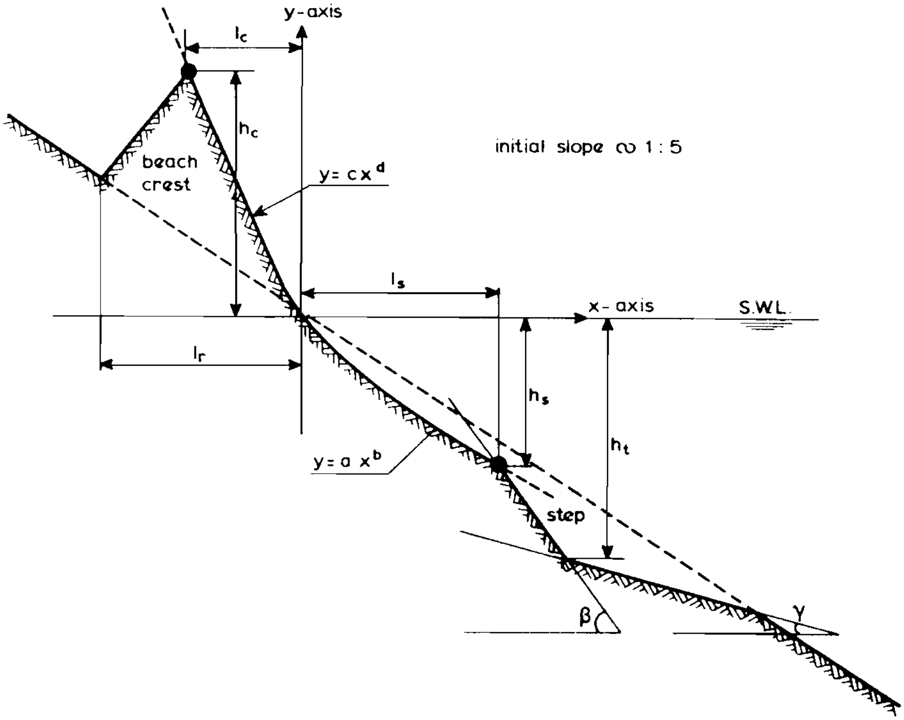


Fig. 16. Schematized profile on 1 : 5 initial slope.

- The step length, l_s
- The transition height, h_t
- The angles, β and γ
- Power functions between h_c and h_s

The schematized profile described above is more or less independent of its location with respect to the initial slope. The location of the local origin (the intersection of the profile with SWL) determines the profile completely. The location of the profile is obtained by means of an iteration process where the profile (the local origin) is moved along the still water level until the mass balance is fulfilled.

Relationships between governing variables and profile parameters

The shape of the dynamically stable profile is given by sets of equations which relate the profile parameters, shown in Fig. 16, to the boundary conditions. A set of equations was developed in Van der Meer (1988) for relatively high $H_s/\Delta D_{n50}$ values of $H_s/\Delta D_{n50} > 10$–20, and a set for lower values, which gives the transition from completely dynamically stable to almost statically stable structures. Both sets of equations are summarized here, with first the equation for completely dynamically stable structures, followed by the one for lower $H_s/\Delta D_{n50}$ values (as berm breakwaters).

The parameters s_m and H_0T_0

The profile parameters were related to the fictitious wave steepness s_m or to the combined wave height-wave period parameter H_0T_0:

$$s_m = 2\pi H_s / g T_m^2 \tag{1}$$

$$H_0 T_0 = H_s/\Delta D_{n50} * \sqrt{g/D_{n50}}\, T_m \tag{2}$$

where $H_0 = H_s/\Delta D_{n50}$ is a dimensionless wave height parameter and $T_0 = \sqrt{g/D_{n50}}\, T_m$ is a dimensionless wave period parameter related to D_{n50}.

The runup length, l_r

$$H_0 T_0 = 2.9 (l_r / D_{n50} N^{0.05})^{1.3} \tag{3}$$

$$H_0 T_0 = (20 - 1.5 \cot\alpha_1) l_r / D_{n50} N^{0.05} - 40 \tag{4}$$

The H_0T_0-intersection between eqs. 3 and 4 gives the transition from one equation to the other, and eq. 3 holds for the highest H_0T_0 region.

The crest height, h_c

$$h_c / H_s N^{0.15} = 0.089\, s_m^{-0.5} \tag{5}$$

$$H_0 T_0 = 33 (h_c / D_{n50} N^{0.15})^{1.3} + 30 \cot\alpha_1 - 30 \tag{6}$$

Equation 5 holds for $H_0T_0 > 900$ and eq. 6 for $H_0T_0 < 900$.

The crest length, l_c

$$H_0 T_0 = 21 (l_c / D_{n50} N^{0.12})^{1.2} \tag{7}$$

$$H_0 T_0 = (3 \cot\alpha_1 + 25) l_c / D_{n50} N^{0.12} \tag{8}$$

The H_0T_0-intersection between eqs. 7 and 8 gives the transition from one equation to the other, and eq. 7 holds for the highest H_0T_0 region.

The step height, h_s

$$h_s / H_s N^{0.07} = 0.22\, s_m^{-0.3} \tag{9}$$

$$H_0 T_0 = 27 (h_s / D_{n50} N^{0.07})^{1.3} + 125 \cot\alpha_2 - 475 \tag{10}$$

Equation 9 holds for $H_0T_0 > 300 \cot\alpha_2$ and eq. 10 for $H_0T_0 < 300 \cot\alpha_2$.

The step length, l_s

$$H_0T_0 = 3.8(l_s/D_{n50}N^{0.07})^{1.3} \tag{11}$$

$$H_0T_0 = 2.6(l_s/D_{n50}N^{0.07})^{1.3} + 70\cot\alpha_2 - 210 \tag{12}$$

The H_0T_0-intersection between eqs. 11 and 12 gives the transition from one equation to the other, and eq. 11 holds for the highest H_0T_0 region.

The transition height, h_t

$$h_t/H_sN^{0.04} = 0.73\, s_m^{-0.2} \tag{13}$$

$$H_0T_0 = 10(h_t/D_{n50}N^{0.04})^{1.3} + 175\cot\alpha_3 - 725 \tag{14}$$

Equation 13 holds for $H_0T_0 > 400\cot\alpha_3$ and eq. 14 for $H_0T_0 < 400\cot\alpha_3$. The transition does not exist if $H_0T_0 < 875 - 125\cot\alpha_3$.

The profile around the still water level

$$y = a_1 x^{0.83} \qquad \text{below SWL} \tag{15}$$

$$y = a_2(-x)^{1.15} \qquad \text{above SWL} \tag{16}$$

where the coefficients, a_1 and a_2, are determined by the values of h_c, l_c, h_s and l_s.

The slope $\tan\beta$

$$\tan\beta = 1.1\tan\alpha_3^A \tag{17}$$

with $A = 1 - 0.45\exp(-500/N)$

The slope $\tan\gamma$

$$\tan\gamma = 0.5\tan\alpha_3 \tag{18}$$

A relatively shallow foreshore

The influence of limited water depth is described by a reduction factor, r, which influences the profile parameters h_s and l_s only. This factor is given by:

$$\begin{aligned} r &= 1 - 0.75(2.2 - h/H_s)^2 && \text{for } h/H_s < 2.2 \\ r &= 1 && \text{for } h/H_s \geqq 2.2 \end{aligned} \tag{19}$$

Oblique wave attack

The influence of oblique wave attack is taken into account when all length and height parameters (except l_c) are reduced by a factor $\cos\psi$.

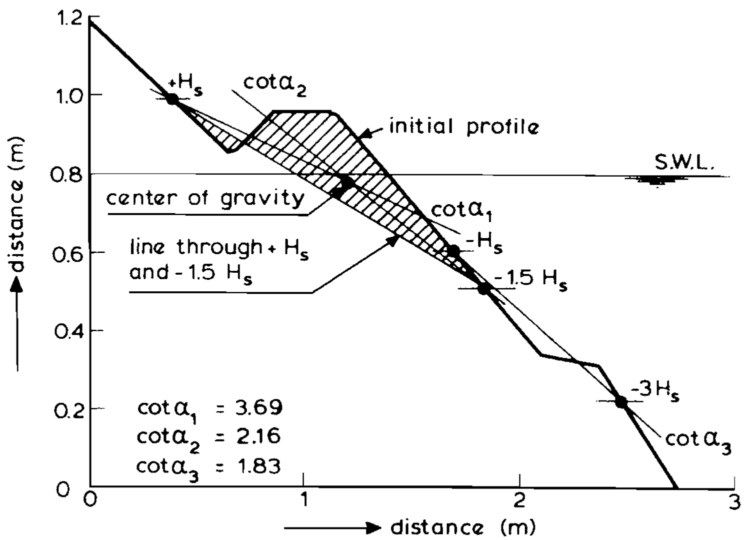


Fig. 17. Equivalent slope angle.

Equivalent initial slope angle

The method to establish equivalent slope angles for an arbitrary initial slope is described now and is shown in Fig. 17.

1. Draw a uniform line through the points $+H_s$ and $-1.5\ H_s$.
2. Establish the center of gravity of the figure between $+H_s$ and $-1.5H_s$ formed by the uniform line and the initial slope (shaded figure).
3. A line through $+H_s$ and the center of gravity gives $\cot\alpha_1$. This equivalent slope angle should be used for l_r, h_c and l_c.
4. A line through $-1.5\ H_s$ and the centre of gravity gives $\cot\alpha_2$ which should be used for h_s and l_s.
5. A line through $-H_s$ and $-3\ H_s$ gives $\cot\alpha_3$. This equivalent slope angle should be used for $\tan\beta$, h_t, and $\cot\gamma$.

VERIFICATION AND APPLICATION OF BREAKWAT

All the relationships for the height and length parameters, the power curves, the two angles β and γ (and the method used to establish the equivalent slope angles for low $H_s/\Delta D_{n50}$ values) were implemented in a computer code. This code is part of Delft Hydraulics' computer program BREAKWAT that can be used for the conceptual design of many types of rubble mound structures. This program can be used to calculate the profile, starting from an arbitrary slope and with varying water levels (tide) and wave conditions.

The input required for the computation can be derived from the relationships developed:

- The mass of the rock M_{50}
- The grading of the rock D_{85}/D_{15}
- The mass density of the rock ρ_a
- The mass density of water ρ_w
- The significant wave height in front of the structure H_s
- The average wave period T_m
- The number of waves N
- The water depth in front of the structure h
- The angle of wave attack ψ

The (arbitrary) initial slope can be given by characteristic points in an x–y plot, connected by uniform lines. It is also possible to use a profile derived from a previous computation as the initial profile for the next computation. In that case a sequence of storms (including water level variations) can be simulated. An example of a calculated profile for a berm breakwater is shown in Fig. 18.

Another part of BREAKWAT can calculate the damage profile of a statically stable straight rock slope. The estimation of this damage profile is made by use of the stability formulae given by Van der Meer (1988) and some additional relationships for the profile. The profile can be schematised to an erosion area around SWL and an accretion area below SWL. The transitions from erosion to accretion, etc. can be described by heights measured from SWL, see Fig. 19. The heights are respectively h_r, h_d, h_m and h_b.

The relationships for the height parameters were based on the tests described by Van der Meer (1988) and will not be given here. The assumption for the profile is a spline through the points given by the heights and with an

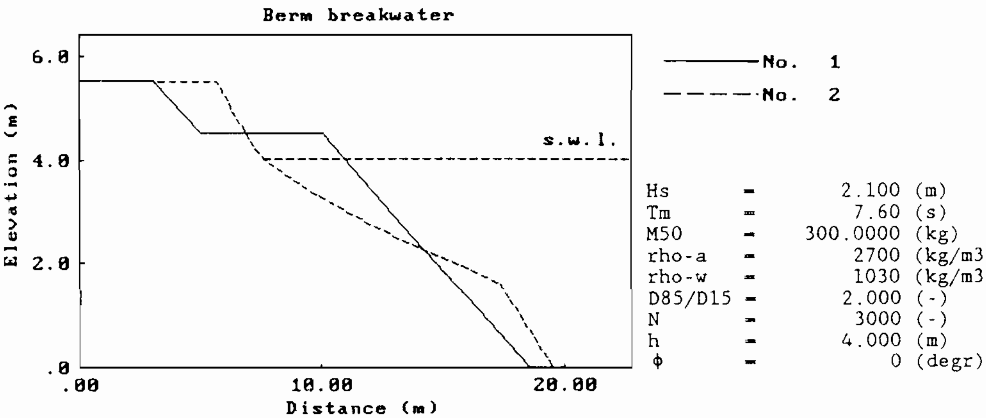


Fig. 18. Dynamically stable profile for berm breakwater as calculated by BREAKWAT.

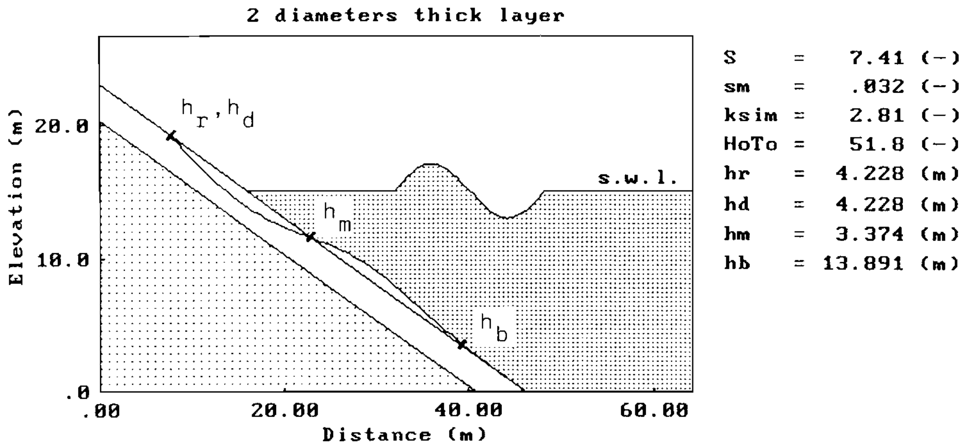


Fig. 19. Statically stable damage profile for a straight rock slope as calculated by BREAKWAT.

erosion (and accretion) area according to the stability mentioned above. The method is only applicable for straight slopes. In case the structure is not really dynamically stable (due to a low $H_s/\Delta D_{n50}$-value), this damage profile may give an estimation of the first damage and profile reshaping. Figure 19 shows an example obtained with BREAKWAT.

The dynamically stable model described in this paper was developed for a wide range of $H_s/\Delta D_{n50}$-values. Berm breakwaters are designed with low $H_s/\Delta D_{n50}$-values in the order of 3–6, and are an important application of the model. Van der Meer (1990) made a verification of BREAKWAT for berm breakwaters and low-crested structures, a report that was prepared for the CUR/CIRIA program on the writing of a manual on use of rock in coastal structures. The results of that verification will be given here.

Data on berm breakwaters and dynamically stable structures in general was asked from various people and institutes. The data sets received and used will be briefly described below.

Ahrens and Heimbaugh (1989) described tests on dumped riprap with a relatively low crest. The three tests with the lowest $H_s/\Delta D_{n50}$-values were selected for verification. These values were 4.4, 7.0 and 3.6.

Hydraulics Research Ltd., Wallingford, UK, tested a berm breakwater during the design stage of a breakwater. Delft Hydraulics was involved in prediction of the behaviour of the structure in order to design the first cross-section for testing. The report is confidential. The maximum $H_s/\Delta D_{n50}$-value was 3.5.

Burcharth and Frigaard (1988) described fundamental tests on a berm breakwater profile in a wave basin. Both perpendicular wave attack and oblique wave attack was used. The $H_s/\Delta D_{n50}$-values were 3.5–7.1.

Four data sets were received from the Danish Hydraulic Institute. The actual reports were confidential and were not revealed. Two cases concerned a berm breakwater, the other two cases were low-crested structures (actually the core of a structure) with the crest around SWL. One of these low-crested structures consisted of a very wide grading of rock. These structures were heavily overtopped.

Tørum et al. (1988) described tests on a berm breakwater.

The general conclusions on the verification of BREAKWAT for each of these data sets are given below.

Ahrens and Heimbaugh (1989)

The calculated profiles were close to the observed profiles. The size of the crest height was a little underpredicted and the length of the profile below SWL a little overpredicted. The relationships for the size of the crest (parameter l_r) are based on only a few tests. In most tests of Ahrens and Heimbaugh a crest was formed above the original crest level. Elaboration of the profiles would give a better relationship for the size of the crest.

Hydraulics research

The calculated profile was very close to the observed one.

Burcharth and Frigaard (1988)

The calculated profiles for the tests of Burcharth and Frigaard were very close to the measured ones. Settlement caused by wave compaction is not simulated by the model. An example of the verification of this data set is shown in Fig. 20.

Danish Hydraulic Institute

The conclusion for the both berm breakwater cases was that the calculations were close to the test results. For the first low-crested core the calculations agree with the test results as long as the condition was dynamically stable and the crest remained above SWL (no severe overtopping). The overall conclusion for the core with the very wide grading was that it had also a very steep slope and that the model overpredicted the erosion for the lowest wave heights very much. The final situation was predicted rather close.

Tørum et al. (1988)

The behaviour of the berm breakwater with wave heights ranging from 2 to 8.6 m was very well predicted by a combined use of the statically stable model and the dynamically stable model.

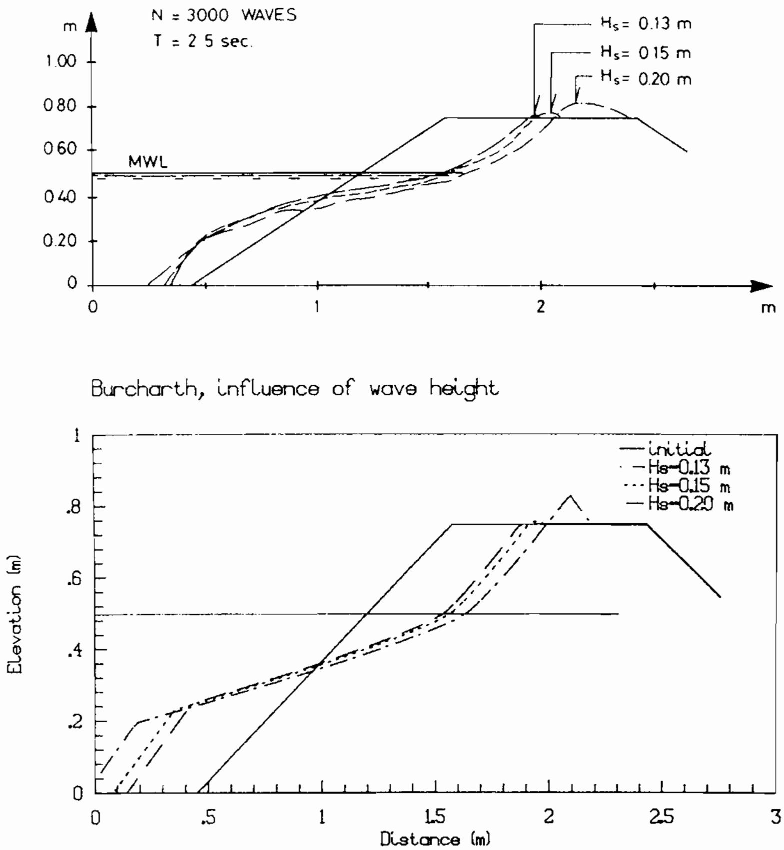


Fig. 20. Verification of BREAKWAT on data of Burcharth and Frigaard (1988).

OVERALL CONCLUSION ON BREAKWAT

The overall conclusion is that the model, including both the dynamically and the statically stable part, never showed large unexpected differences with the test results and that in most cases the calculations and measurements were very close. Compaction of material caused by wave attack and damage to the rear of the structure caused by overtopping, are not modelled in the program and this was and is a boundary condition for use of the program. The combination of the statically stable formulae or model with the dynamically stable model proved to be a good tool for the prediction of the behaviour of the seaward slope of berm breakwaters under all wave conditions.

ACKNOWLEDGEMENT

The writing of this paper was co-sponsored by the Commission of the European Communities under MAST, contract 0032.

REFERENCES

Ahrens, J.P., 1987. Characteristics of reef breakwaters. WES, CERC, Vicksburg, Technical Report CERC-87-17.

Ahrens, J.P. and Heimbaugh, M.S., 1989. Dynamic stability of dumped riprap. Proc. Coastal Zone '89, Charleston, ASCE.

Baird, W.F. and Hall, K.R., 1984. The design of breakwaters using quarried stones. Proc. 19th ICCE, Houston, USA, Ch. 173.

Burcharth, H.F. and Frigaard, P., 1988. On 3-dimensional stability of reshaping breakwaters. ASCE, Proc. 21th ICCE, Malaga, Spain.

Kao, J.S. and Hall, K.R., 1990. Trends in stability of dynamically stable breakwaters. Proc. 22th ICCE, Delft.

Powell, K.A., 1990. Predicting short term profile response for shingle beaches. Hydraulics Research, Wallingford, Report SR 219.

Tørum, A., Naess, S., Instanes, A. and Vold, S., 1988. On berm breakwaters. ASCE, Proc. 21th ICCE, Malaga, Spain.

Van der Meer, J.W., 1987a. Stability of breakwater armour layers — design formulae. Coastal Eng., 11, 219–239.

Van der Meer, J.W., 1987b. Application of computational model on berm breakwater design. In: D.H. Willis, W.F. Baird and O.T. Magoon (Editors), ASCE, Proc. Berm Breakwaters: Unconventional Rubble-Mound Breakwaters, Ottawa, Canada, pp. 93–103.

Van der Meer, J.W., 1988. Rock slopes and gravel beaches under wave attack. PhD. thesis Delft University of Technology. Also Delft Hydraulics Publication No. 396.

Van der Meer, J.W., 1990. Verification of BREAKWAT for berm breakwaters and low-crested structures. Delft Hydraulics Report H986. Prepared for CUR C67.

Van der Meer, J.W. and Pilarczyk, K.W., 1986. Dynamic stability of rock slopes and gravel beaches. ASCE, Proc. 20th ICCE, Taipei, Ch. 125.

Van der Meer, J.W. and Veldman, J.J., 1992. Singular points at berm breakwaters: scale effects, rear, round head and longshore transport. Coastal Eng., in press.

Van Hijum, E. and Pilarczyk, K.W., 1982. Equilibrium profile and longshore transport of coarse material under regular and irregular wave attack. Delft Hydraulics Publication No. 274.