The proposed structures do not eliminate wave action at the shoreline but act to reduce the wave heights and the resulting wave power. The model test results showed that the wave heights in the lee of the breakwaters varied from 40% to 80% of the incident wave heights depending on the wave conditions. It was determined that the submerged breakwaters did not decrease water quality in front of the swimming area.

Stability tests were performed for the development of the final design of the offshore breakwater. The gap model structure tested showed good overall stability with a minimum of stone motion. A stable cross section was obtained based on the satisfactory performance of the hydraulic model.

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CONCEPTUAL DESIGN OF BERM BREAKWATERS

by

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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 dynamicaly 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-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. Results are presented in this paper. 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 I and II project of the European Union, 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 were treated in detail in Van der Meer and Veldman (1992). The stability of singular points such as 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 were investigated and are summarized secondly in the present paper.

DESCRIPTION OF THE SEAWARD PROFILE

Statically stable structures can be described by the damage parameter S, see Van der Meer (1988) and dynamically stable ones by a profile. Typical profiles, but for different initial slopes, are shown in Figure 1. The main part of the profiles is always the same. The initial slope (gentle or steep) determines whether material is transported upwards to a beach crest or downwards, creating erosion around still-water level.
Based on extensive model tests (Van der Meer (1988)) relationships were established between the characteristic profile parameters and the hydraulic and structural parameters. These relationships were used to make the computational model BREAKWAT, which simply gives the profile in a plot together with the initial profile. Boundary conditions for this model are:
- $H_z/D_{50} = 3-500$ (berm breakwaters, rock and gravel beaches);
- Arbitrary initial slope;
- Crest above still-water level;
- Computation of an (established or assumed) sequence of storms (or tides) by using the previously computed profile as the initial profile.

The input parameters for the model are the nominal diameter of the stone, $D_{50}$, the grading of the stone, $D_0/D_{50}$, the buoyant mass density, $\Delta$, the significant wave height, $H_s$, the mean wave period, $T_m$, the number of waves (storm duration), $N$, the water depth at the toe, $h$, and the angle of wave incidence, $\beta$. The (first) initial profile is given by a number of $(x,y)$ points with straight lines in between. A second computation can be made on the same initial profile or on the computed one.

The result of a computation on a berm breakwater is shown in Figure 2, together with a listing of the input parameters. The model can be applied to:
- Design of rock slopes and gravel beaches;
- Design of berm breakwaters;
- Behaviour of core and filter layers under construction during yearly storm conditions.

Figure 2: Example of a computed profile for a berm breakwater

The computation model can be used as a deterministic design approach with a sensitivity analysis, by making a large number of computations. Aspects which were considered for the design of a berm breakwater (Van der Meer and Koster (1988)) were for example:
- Optimum dimensions of the structure (upper and lower slope, length of berms);
- Influence of wave climate, stone class, water depth;
- Stability after first storms.

An example to derive optimum dimensions for a berm breakwater will be described in the next section. The influence of the wave climate on a structure is shown in Figure 3 and shows the difference in behaviour of the structures for various wave climates. Stability after first (less severe) storms can possibly be described by use of stability equations for straight rock slopes (see Van der Meer, 1988).

Figure 3: Example of influence of wave climate on a berm breakwater profile

Computations with the computational model can, of course, only be made if the model is available to the user. This is often not the case for the reader of a paper and therefore a more simple (and less reliable) method should be given which is able to give the user a first impression (but not more than that!) of the profile that can be expected. This method is described below. Other estimates of profiles...
of gravel beaches are described by Powell (1990) and of berm breakwaters only, by Kao and Hall (1990).

Figure 4 gives the schematised profile simplified from the original profiles in the computational model. The connecting point is the intersection of the profile with still-water level. From this point an upper slope is drawn under 1:1.8 and a lower slope under 1:5.5. The crest of the profile is situated on the upper slope and the transition to a steep slope on the lower part. These two points are given by the parameters \(l_s\) (length of crest) and \(l_b\) (length of step). Of course, a curved line goes through the three points.

![Figure 4: Simple schematised profile for rock and gravel beaches](image)

The connection with the upper part of the profile and the initial profile is given by \(l_i\) (length of run-up). Below the gentle part under still-water level a steep slope is present, and if the initial profile is gentle (\(\cot \alpha > 4\)) again there is a gentle slope which gives the "step" in the profile. The transition from a steep to a gentle slope is given by \(h_i\) (height of transition). If the initial slope is not a straight line, one should draw a more or less equivalent slope, taking into account the area from \(+H_i\) to \(-1.5H_i\), which gives \(\tan \alpha\). The relationships between the profile parameters and the hydraulic and structural parameters are:

\[
\begin{align*}
  l_i = 0.041 \frac{H_i}{T_m} & \sqrt{\frac{g}{D_{50}}} \\
  l_s = l_i + 1.8 l_s & \\
  h_i = 0.6 l_i & \\
  \text{Steep slope below still-water level: } \sqrt{\tan \alpha} & \\
  \text{Gentle slope below still-water level: } 0.5 \tan \alpha
\end{align*}
\]

Finally, the profile must be shifted along still-water level until the mass balance is fulfilled. Figure 4 and Eqs. (1) - (5) give a rough indication of the profile that can be expected. For \(H_i/\Delta D_{50}\) values higher than about 10-15 the prediction is quite reliable. For lower values the initial profile has a large influence on the profile and therefore the given method is less reliable. This also applies to berm breakwaters and in case the method should really be treated as a very rough indication.

OPTIMUM DIMENSIONS FOR A BERM BREAKWATER (example)

A berm breakwater can be regarded as an unconventional design. Displacement of armour rocks in the first stage of its lifetime is accepted. After this displacement (profile formation) the structure will be more or less statically stable. The initial cross-section of a berm breakwater can be described by a lower slope 1:m, a horizontal berm with a length b (just above still-water level in this case) and an upper slope 1:n. The lower slope is often steep and close to the natural angle of repose.

The critical design point in the example of Van der Meer and Koster (1988) was that erosion was not allowed at the upper slope above the berm. The minimum required berm length \(b\) was established for this criterion with the computational model. The berm length \(b\) was determined for various combinations of \(m\) and \(n\). Figure 5 shows the final results. Each combination of \(m\) and \(n\) results in a graph giving more or less the same stability (no erosion at the upper slope). It is obvious from Figure 5 that steep slopes require a longer berm and visa versa.

![Figure 5: Minimum berm length as a function of down slope and upper slope for a specific berm breakwater](image)
The overall conclusion was that the model 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 modeled in the program and this was and is a boundary condition for application of the program.

The combination of statically stable formulae (Van der Meer, 1988) with the dynamically stable model showed to be a good tool for the prediction of the behaviour of berm breakwaters under all wave conditions.

REAR STABILITY OF BERM BREAKWATERS

Van der Meer and Veldman (1992) performed extensive test series on two different berm breakwater designs. A first design rule was assessed on the relationship between damage to the rear of a berm breakwater and the crest height, wave height, wave steepness and rock size.

The boundary condition is that the rock at the crest and rear of the berm breakwater has the same dimensions as at the seaward profile. This means that $H/A_{abc}$ is in the order of 3.0 - 3.5. A further restriction is that the profile at the seaward side has been developed to an S-shape.

The parameter $R/H_s * S_{wp}$ showed to be a good combination of relative crest height and wave steepness to describe the stability of the rear of a berm breakwater. The following values of $R/H_s * S_{wp}$ 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.

\[
\begin{align*}
R/H_s &\leq 0.25: \text{start of damage} \\
R/H_s &\leq 0.21: \text{moderate damage} \\
R/H_s &\leq 0.17: \text{severe damage}
\end{align*}
\]

A lower value of $R/H_s * S_{wp}$ means more overtopping and therefore more damage. Both a lower relative crest height $R/H_s$ and a lower wave steepness give more overtopping, and therefore, more damage.

Andersen et al. (1992) performed basic tests on the stability of the rear of a berm breakwater. They included also not fully developed profiles and (very) long berms.

HEAD OF A BERM BREAKWATER

Burchart and Frigaard (1987) have studied longshore transport and stability of berm breakwaters in a short basic review. The recession of a breakwater head is shown as an example in Figure 7, for fairly high wave attack ($H/A_{abc} = 5.4$). Burchart and Frigaard (1987) give as a first rule of thumb for the stability of a breakwater head that $H/A_{abc}$ should be smaller than 3.

\[
\text{Figure 7 Example of erosion of a berm breakwater head (taken from Burchart and Frigaard (1987))}
\]
Tests on a berm breakwater head by Van der Meer and Veldman (1992) showed 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 AT BERMT BREAKWATERS

Statically stable structures 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 rocks from the still-water level to (in most cases) a location downslope. Movement of rocks 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 may cause transport of material along the structure. This phenomenon is called longshore transport and is well known for sand beaches. Also shingle beaches change due to longshore transport, although the research on this aspect has always been limited.

Rock beaches and berm breakwaters are or can be also dynamically stable under severe wave action. This means that oblique wave attack may induce longshore transport, which can also cause problems for these types of structures. Longshore transport does not occur for statically stable structures, but it will start for conditions where the diameter is small enough in comparison with the wave height. Then the conditions for start of longshore transport are important.

The start of longshore transport is the most interesting consideration for the berm breaker where profile development under severe wave attack is allowed, but longshore transport should be avoided. The berm breaker can roughly be described by 2.5 < \( \frac{H_s}{\Delta D_{50}} < 6 \). Burchart and Frigaard (1987) performed model tests to establish the incipient longshore motion for berm breakwaters and their range of tests corresponded to 3.5 < \( \frac{H_s}{\Delta D_{50}} < 7.1 \). Longshore transport is not allowed at berm breakwaters, and therefore Burchart and Frigaard gave the following (somewhat premature) recommendations for the design of berm breakwaters, which are in fact the criteria for incipient motion:

For trunks exposed to steep waves
\[
\frac{H_s}{\Delta D_{50}} < 4.5
\]

For trunks exposed to oblique waves
\[
\frac{H_s}{\Delta D_{50}} < 3.5
\]

For roundheads
\[
\frac{H_s}{\Delta D_{50}} < 3
\]

Van der Meer and Veldman (1992) tested a berm breakwater under angles of wave attack of 25 and 50 degrees. Burchart and Frigaard (1987, 1988) tested their structure under angles of 15 and 30 degrees. Longshore transport was measured by the movement of rocks 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, \( S(t) \), was defined as the number of rocks that was displaced per wave. Multiplication of \( S(t) \) with the storm duration (the number of waves) in practical cases would lead to a transport rate of total number of rocks displaced per storm. Subsequently, the transport rate can be calculated in m³/storm or m³/s.

Figures 8 and 9 give all the test results on longshore transport, both for the tests of Van der Meer and Veldman (1992) and the tests of Burchart and Frigaard (1988). Both a higher wave height and a longer wave period result in larger transport and therefore a combined wave height-wave period parameter \( H_s T_w \) can be used:

\[
H_s T_w = \frac{H_s}{\Delta D_{50}} T_w \sqrt{\frac{S}{D_{50}}}
\]

(8)

\( H_s \) is defined as the stability number \( \frac{H_s}{\Delta D_{50}} \) and \( T_w \) as the dimensionless wave period related to the nominal diameter: \( T_w = T_p \sqrt{D_{50}} \). With the parameter \( H_s T_w \) it is assumed that wave height and wave period have the same influence on longshore transport. Figs. 8 and 9 give the longshore transport \( S(t) \) (in number of rocks per wave) versus \( H_s T_w \).

Figure 7 gives all the data points. The maximum transport is about 3 rocks/wave for \( H_s T_w = 350 \), which is in fact a very high rate for berm breakwaters. The \( \frac{H_s}{\Delta D_{50}} \) value in that case was 7.1, considerable higher than the design value for berm breakwaters. Figure 7 also shows that quite a lot of tests had a much smaller transport rate than 0.1 - 0.2 rocks/wave.

Therefore, Figure 8 was drawn with a maximum transport rate of only 0.1 rocks/wave. Now only 4 data points remain of Burchart and Frigaard (1988), the others are from the tests of Van der Meer and Veldman (1992). Figure 54 shows that the transport for large wave angles of 50 degrees is much smaller than for the other angles of 15 - 30 degrees. The two lowest points of Burchart and Frigaard show transport for \( H_s T_w = 100 \), where the present tests do not give longshore transport up to \( H_s T_w = 117 \).
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 Van der Meer and Veldman (1992) 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), these are similar to those in Figure 7 they come to a formula for longshore transport:

\[
S(x) = 0 \text{ for } H_{T_{w}} < 100
\]

\[
S(x) = 0.000048 (H_{T_{w}} - 100)^3
\]  

Eq. (9) is shown in Figs. 7 and 8 with the dotted line. The equation fits nicely in Figure 7, but does not fit the average trend for the low \(H_{T_{w}}\) region, see Figure 8. The equation overestimates the start of longshore transport a little (except for 2 points of Burcharth and Frigaard). Therefore Eq. (9) was changed a little in order to describe the start of longshore transport better:

\[
S(x) = 0 \text{ for } H_{T_{w}} < 105
\]

\[
S(x) = 0.00005 (H_{T_{w}} - 105)^3
\]  

This equation holds for wave angles roughly between 15 and 35 degrees. For smaller or larger wave angles the transport will be (substantially) less. Eq. (10) is shown in Figs. (7) and (8) with the solid line and fits better in the low \(H_{T_{w}}\) region. The upper limit for Eq. (10) is chosen as \(H_{L}/\Delta P_{w} < 10\). With Eq. (10) the longshore transport for berm breakwaters has been established.