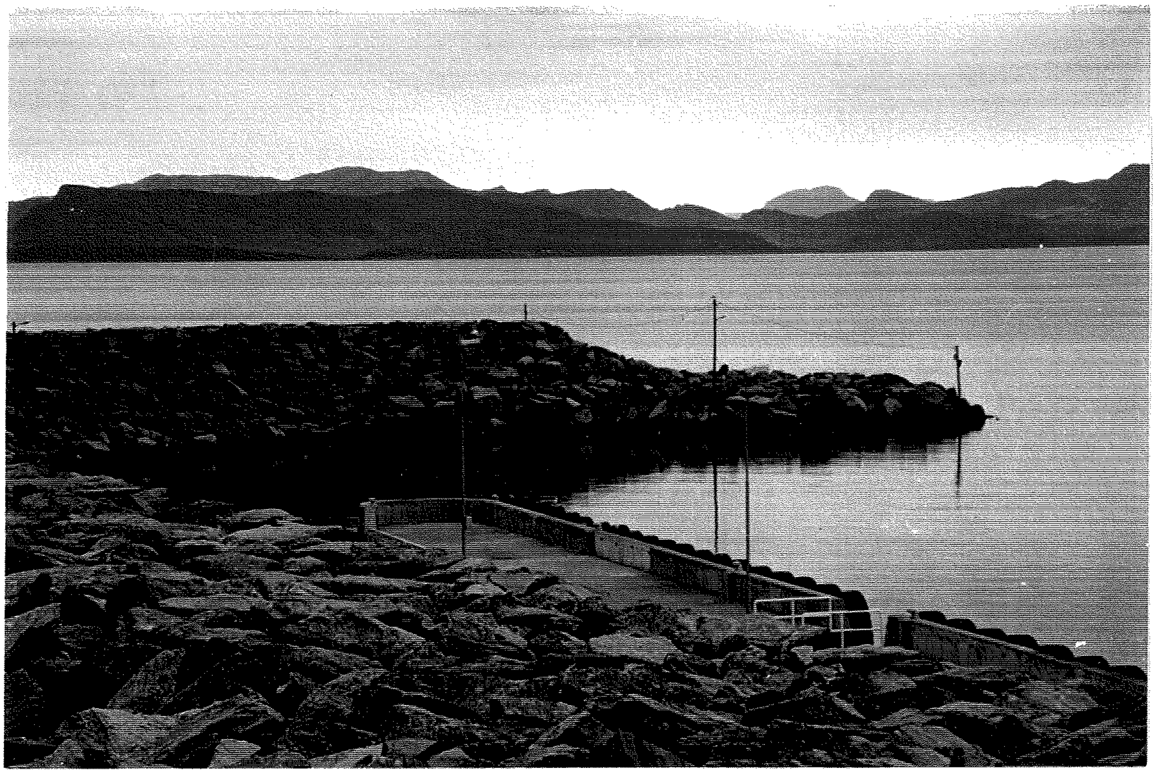
BERM BREAKWATER STRUCTURES

FINAL REPORT

May 1997



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Berm Breakwater Structures - Research Summary

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Abstract

A research project on berm breakwater structures was carried out with the overall objective of arriving at a better design basis. Eight European organisations from Denmark, The Netherlands, United Kingdom, Norway, Italy and Iceland participated in the joint research project co-sponsored by the European Commission under the second research and development programme on Marine Science and Technology, MAST II. Studies were made for understanding the physics of berm breakwaters, profile development, problems related to practical engineering applications and three-dimensional (3D) effects. A combination of theoretical work, physical model tests and numerical modelling was used in the project.

Features of berm breakwaters are described followed by examples of prototype experience. A summary of the main results of the research project is presented, including forces on individual berm stones, numerical modelling of the flow on and in berm breakwaters and of the berm reshaping, parameter analysis, study of the influence of permeability and stone gradation, results of model tests with scour protection, and analysis of 3D model tests carried out in both deep and shallow water for studying the roundhead and trunk stability.

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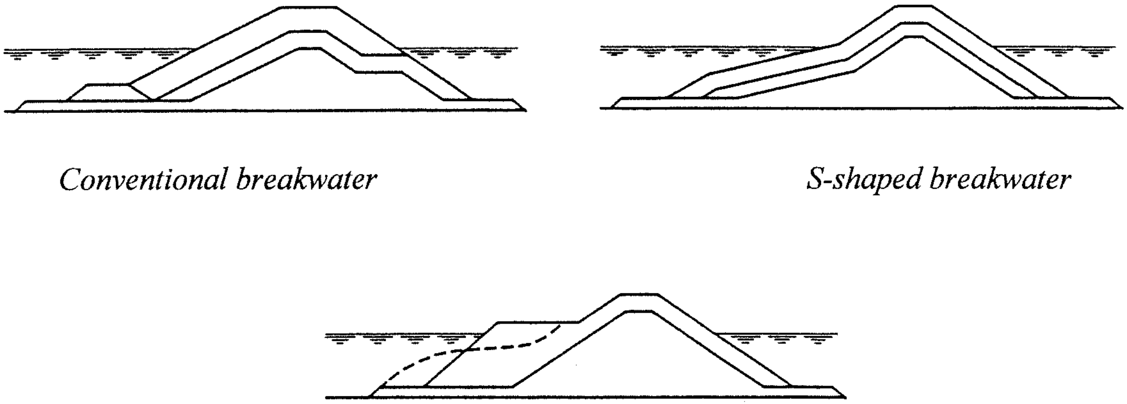
Introduction

In principle, two different types of rubble mound breakwaters exist, ie conventional rubble mound breakwaters with or without a crown wall and berm breakwaters. The main armour layer of a conventional rubble mound breakwater is designed for limited damage (statically stable), whereas for a berm breakwater the berm reshapes into a flatter and more stable profile. The more stable reshaped profile of a berm breakwater is the basic idea of the S-shaped breakwater, which is initially built with a flatter statically stable slope around still water level. In **Figure 1**, typical cross-sections of the three mentioned types of rubble mound breakwaters are shown.



Conventional breakwater *S-shaped breakwater*

Berm breakwater

Figure 1 *Typical cross-sections of three types of rubble mound breakwaters.*

A berm breakwater is a rubble mound breakwater with a berm above still water on the seaward side. During exposure to wave action of a certain intensity, the berm reshapes until eventually an equilibrium profile on the seaward face is reached. For each wave height, there is thus an equilibrium profile corresponding to this wave height. Just below the water level, the reshaped profile has typically a slope of about 1:5. In front of this flat slope, stones are deposited with a steeper slope approaching the natural angle of repose. Wave energy is dissipated in the mass of stones in the flat slope resulting in reduced wave run-up above still water level where the natural equilibrium slope is steeper.

Berm breakwaters should reshape into a statically stable profile where stone movements are only occurring in very severe and rare conditions as frequent movements could result in abrasion and fracturing of stones finally resulting in degradation of the breakwater. A berm breakwater trunk section exposed to oblique waves has to be designed with stones larger than a certain critical size in order to avoid continued movements of stones in the wave direction (longshore transport) which would eventually endanger the breakwater.

Singular points are of special interest for berm breakwaters, eg bends and roundheads. Stone displacements occurring at singular points result in stones being moved in the wave direction and the structure is consequently weakened. A point of special concern is whether, and under which conditions, a berm breakwater roundhead after some initial re-shaping may develop into a stable shape that is not subject to continued erosion, or at least such slow erosion that it may be acceptable for a permanent structure.

The average armour rock size required for a berm breakwater structure is smaller than for a traditional rubble mound structure. This is due to the flatter final slope of the seaward face on which the breaking wave plunges and dissipates energy, and the higher proportion of wave energy dissipated within the porous mound (reducing the hydrodynamic forces acting on the individual stones). Further, wave action causes consolidation of the break-water and nesting of the stones, which increase the stability. Typically, stones with a weight two to ten times smaller can be used for construction of the berm compared to the main armour layer of a conventional breakwater.

Especially when a quarry is available near the construction site and it is not possible to produce a sufficient quantity of large armour stones for a conventional rubble mound breakwater, a berm breakwater can be a feasible solution. Berm breakwaters are presently being considered for more and more applications worldwide, and several berm breakwa-ters have already been constructed. Berm breakwaters can normally be constructed with only two stone gradations as indicated in **Figure 1,** but experience from especially Iceland has shown that in many cases a less expensive structure can be made using the largest stones as an armour layer of the berm. Proper design of berm breakwaters might lead to utilisation of almost 100 per cent of the quarry yield.

The smaller stones to be used for berm breakwaters have also an influence on the con-struction method and equipment to be used. The core can be constructed by end tipping trucks or dumping by barges, whereas the berm can be constructed by cranes with stone grabs, end tipping trucks or excavators. Generally lighter and less specialised construction equipment can be used compared to construction of conventional breakwaters. Even if the construction tolerances are wider for berm breakwaters than for conventional rubble mound breakwaters, fulfilment of the specifications to the stone material (mean weight, gradation, geometrical shape, quality, content of fines, etc), construction method and breakwater profiles is strictly required.

Prototype Experience

Juhl and Jensen (1995) presented a review of selected practical experience with eight berm breakwaters in terms of typical cross-sections and key parameters (two examples from Norway, Iceland and USA, and one from Faroe Islands and Australia). In three of the eight cases presented, prototype measurements of the reshaped profile were made, and it was found that the berm breakwaters in question performed well during wave conditions approximating the design conditions. Further, in two of the cases good agreement for the reshaped profiles was found with measurements from model tests.

The practical experience with the eight berm breakwaters presented showed that the dimensionless stability parameter, $H_o=H_s/\Delta D_{n50}$, varies between 2.5 and 4.1. The Icelandic experience shows a stability parameter in the range from 2.4 to 3.2 for breakwater trunk sections, and in the range of 1.7 to 2.4 for breakwater roundheads where the largest quarry stones are placed as an armour layer of the berm.

The ratio between the crest freeboard and the significant wave height, R_c/H_s, varied between 0.7 and 1.2, which is smaller than for conventional rubble mound breakwaters, as the porous berm reduces wave run-up and overtopping. This range of dimensionless crest freeboard is found to agree well with results from model tests carried out for studying the rear side stability of berm breakwaters, see Andersen et al (1992) and van der Meer and Veldman (1992).

Icelandic Experience

There is 14 years of experience in design and construction of berm breakwaters in Iceland. More than twenty rubble mound structures of the berm type have been constructed since 1983, fourteen of those were new structures whereas the others were improvements or repair of existing breakwaters. The Icelandic Maritime Administration (IMA) has through their involvement in all phases of the design and construction of these projects gained an insurmountable experience with berm breakwaters. Examples of their experience were presented in Sigurdarson and Viggosson (1994), and Sigurdarson et al (1996).

The design philosophy of berm breakwaters aims at optimising the structure with respect to wave load and possible yield from a quarry. The estimated quarry yield is used as an integrated part of the design process in an attempt to optimise the utilisation of the quarry and in many cases a 100 per cent utilisation is achieved. The berm concept has also proved to be very efficient to reduce wave overtopping on existing breakwaters.

A variant of the traditional berm breakwater concept has been developed with all the major advantages of the berm concept and furthermore being more stable. This variant can be described as a "tailor-made size graded berm breakwater" constructed of several stone classes. The largest stones are used on top of the berm and some times also at the front of the berm in order to reinforce the structure. Depending on the quarry yield, the breakwater can be made of more layers. This variant allows the use of even smaller stones inside the berm than in a traditional berm breakwater of only two stone classes. The sorting of armour stones is only increased marginally as all armour stones are weighed as part of the handling in the quarry. The design aims at minimising the deformation of the berm during design conditions, and in many cases leads to a less expensive structure than the traditional berm concept.

The berm concept was introduced in Iceland through the design phase of the Helguvik breakwater which was designed after the original berm concept with only one stone class for the berm (constructed 1986-88). An overproduction of core material was necessary to secure sufficient volume of berm stones larger than 1.7 t.

Bolungarvik

Bolungarvik is an active fishing harbour located on the north-west coast of Iceland. The existing harbour facilities include a 215 m long protecting pier giving little shelter for northerly waves. Due to large wave action at the lee side of the pier and huge wave overtopping, it was frequently closed down during the winter time. Through studies including physical model tests, it was found optimal to construct a berm type breakwater on the ocean side of the existing pier.

The layout of the harbour and typical breakwater cross-sections are presented in **Figure 2**. The armour layer protecting the berm at the trunk section consists of 3 to 8 t stones giving a stability parameter, $H_s/\Delta D_{n,50}$, of 3.1. Stones of 1 to 3 t are used for the berm, ie a stability parameter of 4.3. At the breakwater head, 8 to 14 t stones are used on the top of the berm giving a stability parameter of 2.4.

The total volume of the breakwater is 200,000 m^3 with equal parts of core and berm stones. All stone material was produced in a quarry located 5 km from the construction site. The cost estimate was divided as follows: 25 per cent for the production in the quarry, 50 per cent for transport and 25 per cent for placement of the stones.

After exposure to a storm close to the design conditions, only a very few stones have moved and none more than their diameter.

Bakkafjordur

The breakwater at Bakkafjordur was built in 1983-84 from stones of a very poor quality quarried at the breakwater site. Samples of stones were tested in freeze/thaw tests and gave very poor results, and the stones are expected to have little resistance to abrasion.

In the winter 1992/93, the breakwater is believed to have experienced waves close to the design load. The entire berm was eroded and erosion of the crest had initiated. Deterioration of the stones has accelerated the development of the profile. A typical cross-section of the constructed berm breakwater is shown in **Figure 3** together with comparisons of prototype measurements and results from model tests. Reasonable agreement was found between the profile developments in model and prototype. The possible rounding and breakage of the available poor quality stones were included in the model study by testing with reduced stone size.

Repair took place in 1993, and although the breakwater may need some maintenance every ten years or so, it was considered the most economic solution again to use the poor quality stones from the local quarry.

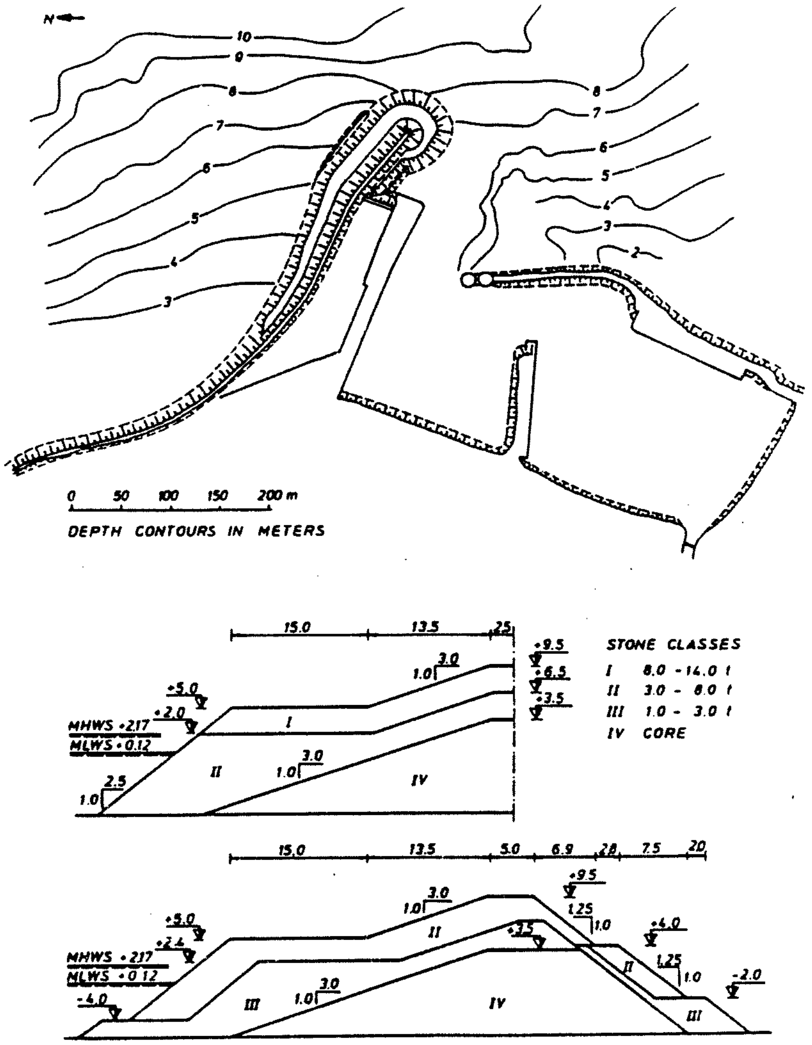


Figure 2 *Layout and cross-section for the Bolungarvik breakwater. The upper cross-section is from the roundhead and the lower from the trunk section.*

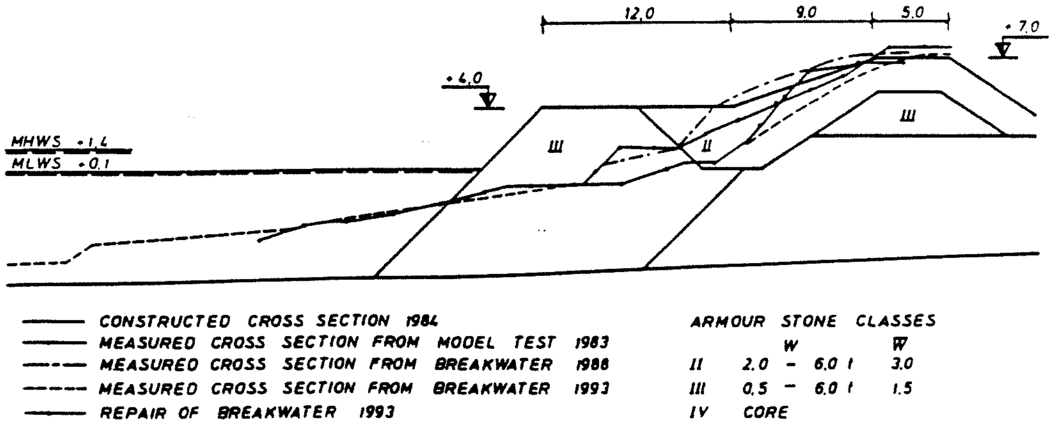


Figure 3 *Measurements of profile reshaping of berm breakwater at Bakkafjordur. All measures are in m.*

Norwegian Experience

Two berm breakwaters have been constructed in Norway: one in Årviksand (1989) and one in Rennesøy (1992). In order to reduce construction costs, both projects include a structural variant to the typical berm breakwater profile. A monitoring programme was established for each of these breakwaters including profiling after severe storms that cause reshaping of the berm. Further, fifty of the armour stones in Årviksand have been marked and it is planned to track their movements after reshaping has taken place. The two breakwaters have not yet been exposed to waves approaching the design wave height, and thus no significant reshaping of the berm breakwaters has taken place until now, except in a small exposed area on the Rennesøy breakwater. However, it is anticipated that future monitoring will give valuable prototype experience on the profile development.

Årviksand

The berm breakwater constructed in Årviksand in northern Norway is an extension of a breakwater at a fishing port, see Tørum et al (1990). Through model tests, it was found economical to use larger stones for the rear side of the breakwater to protect against wave overtopping rather than increasing the crest elevation or extending the berm width. This solution requires an additional stone class to be handled in the quarry. A typical profile of the trunk section including a rear side strengthened against the impact of wave overtopping is shown in **Figure 4**. The upper part of the berm at the breakwater head was armoured with larger stones and the top elevation of the berm increased, as shown in **Figure 5**.

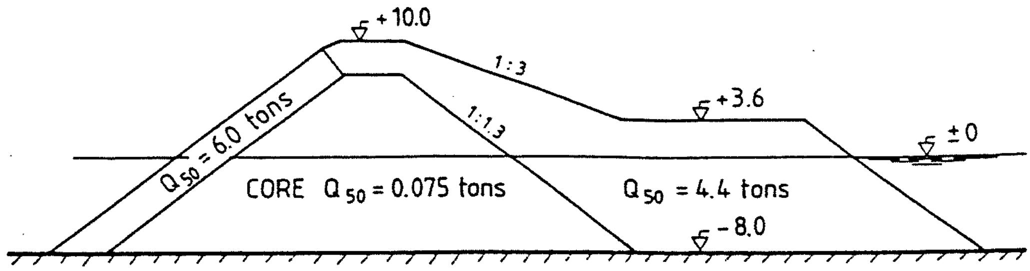


Figure 4 *Extension of breakwater at Årviksand, profile of breakwater trunk section. All measures are in m.*

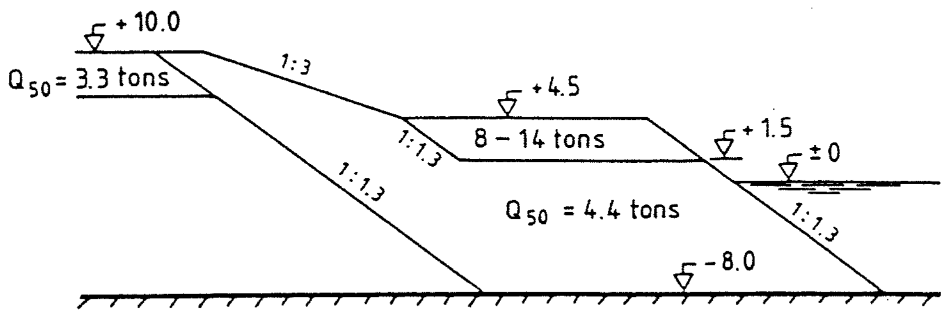


Figure 5 *Extension of breakwater at Årviksand, profile of breakwater head. All measures are in m.*

Rennesøy

A new ferry terminal has been constructed at Rennesøy with a berm breakwater protecting the harbour facilities (Espedal and Lothe, 1994). From an economical point of view, it was desirable to extend the core (0-1.5 t stones) into the berm to make better use of the quarry yield. Based on results from model tests, the profile shown in **Figure 6** was selected for the most exposed parts of the trunk, whereas the roundhead was designed with larger stones on top of the berm and without extension of the core into the berm. A photo of the breakwater head is shown in **Figure 7**.

This structural variant may in many cases be economical due to substitution of berm stones by cheaper core material, which will result in less energy dissipation in the berm. The influence of extending the core into the berm is described in Tørum and Lissev (1996).

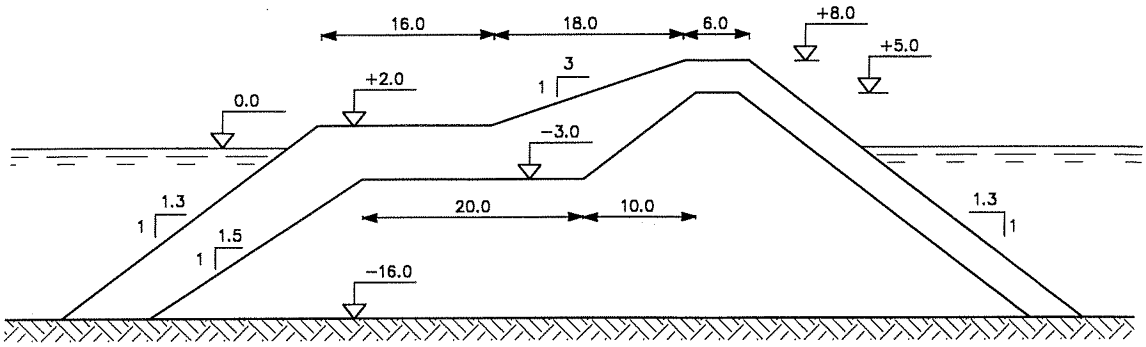


Figure 6 *Typical cross-section for berm breakwater on Rennesøy. All measures are in m.*

Figure 7 *Photo of the Rennesoy breakwater head.*

In 1993, profiling was made of a trunk area in which the berm had reshaped, and it was found that the recession of the berm was significant. The final model tests performed for studying the breakwater stability only included the roundhead, but in the wave disturbance tests in scale 1:100, it was observed that the area in which reshaping actually has occurred was the most exposed section of the breakwater.

Forces on Individual Berm Stones

The different armour stones on a berm breakwater have different sizes and shapes and are randomly located, and thus will be exposed to the flow in a somewhat arbitrary way. The wave forces induced on a certain stone also vary with the sheltering effect from the neighbouring stones. To explore fully the forces on single stones will therefore require tests with several stones placed in different orientations and in different positions. Measurements of forces on a sharp edged stone and associated water particle velocities were first carried out and presented by SINTEF NHL, see Tørum (1994). The measurements covered a single stone placed in one position and were made to gain more insight into the mechanism of regular wave forces on an armour unit on a berm breakwater.

In this research study, irregular waves were used, and the forces were measured on an armour stone placed at different locations along the breakwater slope and with the stone in an embedded and elevated position at each location. The orientation of the stone was the same in each location.

The force measurements were made with a force transducer placed in a cylindrical box and placed in the breakwater as shown in **Figure 8**. To exclude any contact between the force measurement stone and neighbouring stones, a stiff chicken wire mesh was placed around the stone used for the force measurements. The location of the force measurement points are shown in **Figure 9**. The forces in the direction normal as well as parallel to the slope were measured. Further details on the tests and the analysis are described in Westeren and Tørum (1996).

The force data were analysed by fitting the data to a Weibull distribution function. Among the statistical force data, the 90 per cent peak force was picked out for each tested wave condition, ie the force that was not exceeded for 90 per cent of the peak forces. **Figure 10** shows the 90 per cent parallel force for the different measurement points and for the different wave conditions for the elevated stone case. The trend was the same for the other tested conditions, namely that the forces were largest above the still water line. These forces had to some extent the character of slamming forces having a high intensity and short duration. Hence a mechanism for moving the stones may be that they are gradually 'knocked' loose by the slamming forces and then moved down the slope.

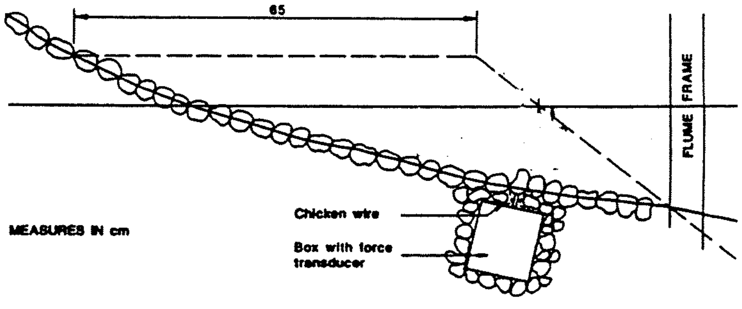


Figure 8 *Force transducer box placed in the breakwater (measurements in cm).*

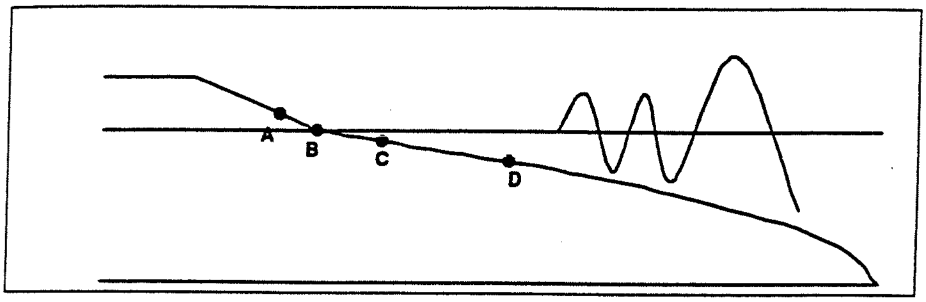


Figure 9 *Location of the force measurement points A, B, C and D.*

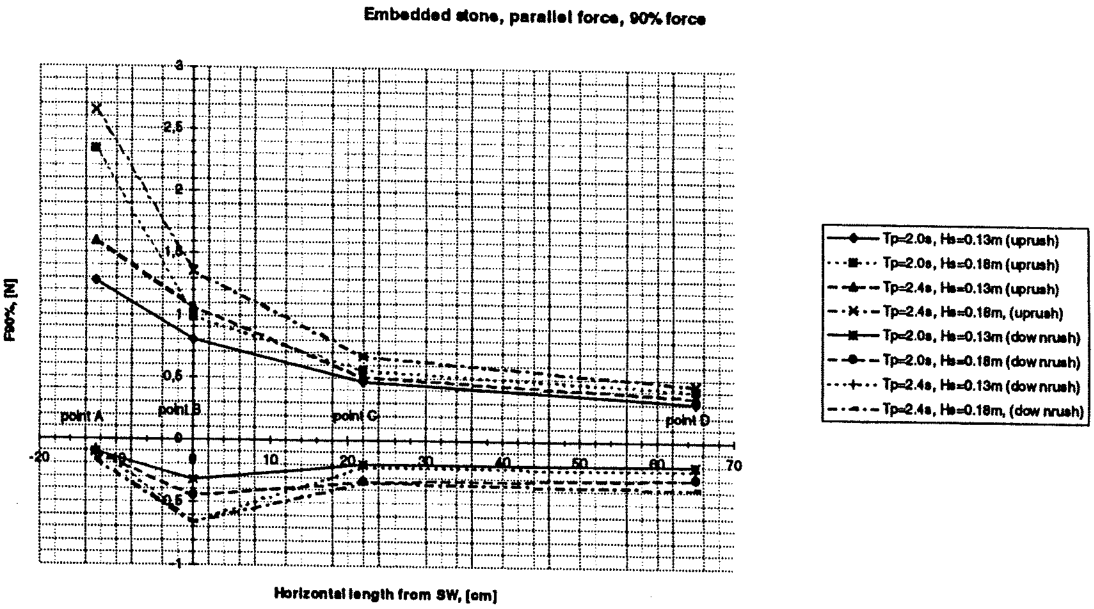


Figure 10 *Parallel forces for the embedded stone case.*

The parallel forces are highest for the largest peak wave periods provided the significant wave heights are the same. This means that the waves with the lowest steepness give the highest forces, which agrees with the findings that waves with smaller wave steepness result in larger reshaping effect on a berm breakwater.

Numerical Modelling

Numerical modelling of the flow on and inside permeable structures and of the reshaping of berm breakwaters was made both by 1D and 2D numerical models. The 1D numerical model developed by Delft University of Technology can after some additional verification of the reshaping part be regarded as an engineering tool, whereas the other models at the moment are research tools which have to be developed further in order to be applicable for normal engineering services.

1D Numerical Model

Wave interaction with permeable coastal structures, with the emphasis on berm breakwaters, was studied by Delft University of Technology (Gent (1995) and Gent (1996)). Physical processes involved in the hydraulic and structural response of berm breakwaters under wave attack were examined, and a predictive numerical model for wave interaction with berm breakwaters developed. Special attention was given to berm breakwaters that reshape under heavy storm conditions by redistributing the stones in the seaward profile. The reshaping of such dynamic structures was modelled numerically.

In MAST I, a 1D model based on shallow water wave equations was developed for simulating wave motion both inside and outside permeable structures. In this project, a procedure was developed to simulate the reshaping of dynamic structures such as berm breakwaters, reef-type structures and gravel beaches. This procedure was incorporated in the 1D wave model. This integrated simulation model can be used to study the interaction of the internal and external wave motion of permeable structures and to study the interaction of the wave motion with the structures themselves.

The reshaping process of dynamic seaward slopes as a result of hydrodynamic loadings was studied. Van der Meer (1992) performed many model tests to study the stability of the seaward slope of berm breakwaters. The results were summarised in empirical relations determined by for instance wave height, wave period, stone diameter, number of waves and initial slope. The relations are valid for a wide range of parameters. However, it can be useful to study the reshaping process in a more detailed way. This can be done by modelling the reshaping process numerically by using hydrodynamic properties such as velocities and accelerations, ultimately leading to a more general applicable method and a complementary design tool.

A new approach to simulate the reshaping process as a result of the hydrodynamic loadings is presented. The initiation of movement is based on a Morison-type of equation including drag, inertia and lift forces. The new position of unstable stones is determined using the hydrodynamic forces which vary in time and space. The new position of the moved stones is immediately incorporated in the wave model. Verification and sensitivity analyses showed that the model can provide rather accurate results for quite some practical cases. **Figure 11** shows a comparison with prototype measurements described by Montgomery et al (1987).

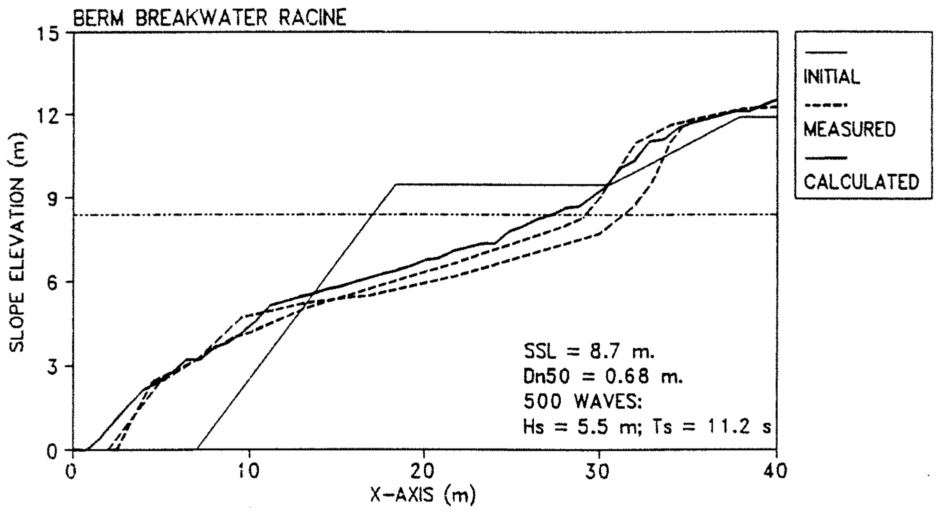


Figure 11 *Comparison between measured and computed reshaped profiles of a prototype berm breakwater.*

Several computations were made to show the dependencies on the reshaped profiles of wave height, wave period, stone diameter and initial slope. **Figure 12** shows that the affected reshaped profile becomes longer for longer wave periods. This was also observed in the experiments by van der Meer (1992). Also the influence of the size of the stones is represented as detected in physical model tests, see **Figure 13**. Similar conclusions can be drawn for variations of the wave height and of the initial slope.

A set of physical model tests were run at Delft University of Technology and used for additional validations of the numerical modelling of the wave interaction with berm breakwaters.

The numerical reshaping model was extended by Delft Hydraulics to include the effects of stone gradation.

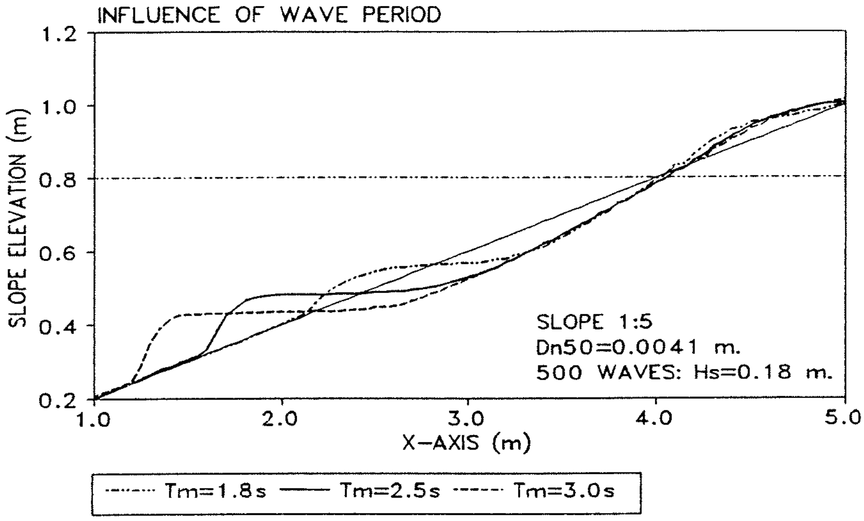


Figure 12 *Influence of wave period on calculated profiles*

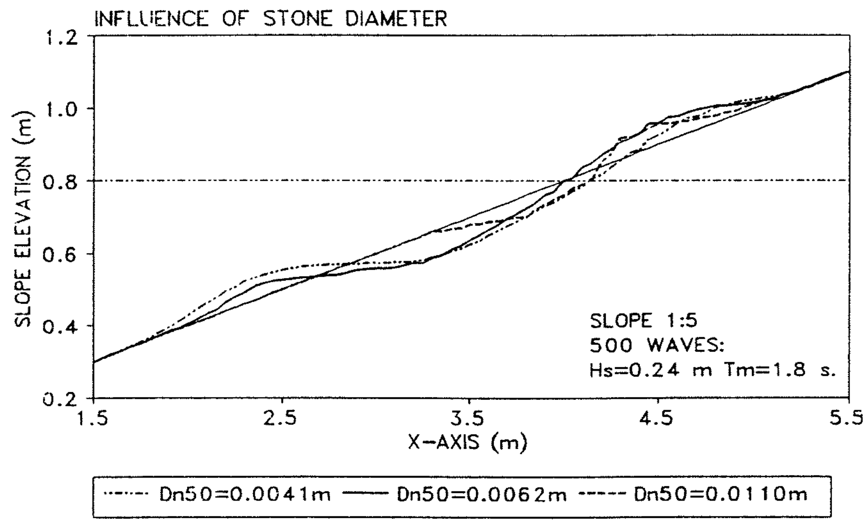


Figure 13 *Influence of stone diameter on calculated profiles*

1D Numerical Sphere Model

A numerical model for reshaping of berm breakwaters has been further developed within the project by Imperial College. The model involves three programmes: Solution of the nonlinear shallow water wave equations (**NSW**), a numerical model for the construction of a berm breakwater structure (**BOX**) and a 2D model for the reshaping of dynamically stable berm breakwaters (**RSHP**). The validity of the **NSW** model was accepted as adequate based on comparison with the model developed at Delft University of Technology. The **BOX** model was found to work well, providing a simulation of the packing of spherical 'rocks' drawn from a defined rock-size grading. The main effort within the project was devoted to the **RSHP** programme because of its complexity.

The wave model, NSW

This model predicts the flow characteristics on the face of impermeable or permeable slopes subjected to regular, periodic wave attack, specified by a wave height and wave period. It is based on theoretical models of Kobuyashi et al (1990) with a restriction that the depth of a permeable face of a berm breakwater must be limited to a thickness of the order of the incident wave height, below which the structure is assumed to be impermeable.

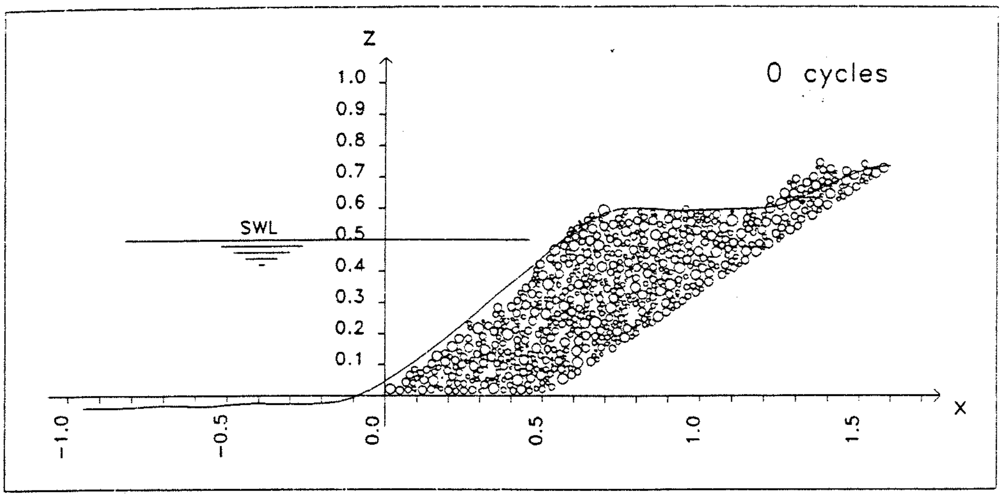
The structure model, BOX

This model provides a numerical description of the berm breakwater structure in which individual armour units are represented by spheres. The spheres are drawn randomly from a source of known unit diameter grading and placed on the structure in a predetermined geometrical pattern. The model then allows the units to be displaced until their static stability is satisfied. In the present work, the flexibility of the numerical model has been improved significantly so that a wide range of structural profiles can be generated.

The assumption of spherical units is not as restrictive as might appear because their freedom to move under wave-induced loading can be controlled in the reshaping model in terms of the moment required to reduce their stability to zero. This is an indirect representation of the angularity of rock armour units.

The reshaping model, (RSHP)

This component of the total programme is complex. It takes the hydrodynamic field predicted by the **NSW** model (the programme has been written to accept data from other models of the hydrodynamics, if required) and applies this successively to the most exposed units on the breakwater face. Hence, the wave-induced loading (drag, lift and inertia) on each unit is calculated. This allows the stability of the units to be evaluated via the moments due to hydrodynamic, gravity and buoyancy forces taken about the contact point axis of each unit - noting that there is a random, but known, geometrical arrangement of the units. If the particle is unstable, it is allowed to move in the general direction of the applied hydrodynamic force, subject to the local geometry of the particles on which it is initially resting. This results in a reshaping of the breakwater profile, the calculation being repeated until there is negligible change. **Figure 14** illustrates a comparison with reshaping in the deep water 3D tests at Danish Hydraulic Institute. The comparison with laboratory results is good considering the complexity of the numerical simulation, but several key factors within the model will require further work before it can be used as a design tool.



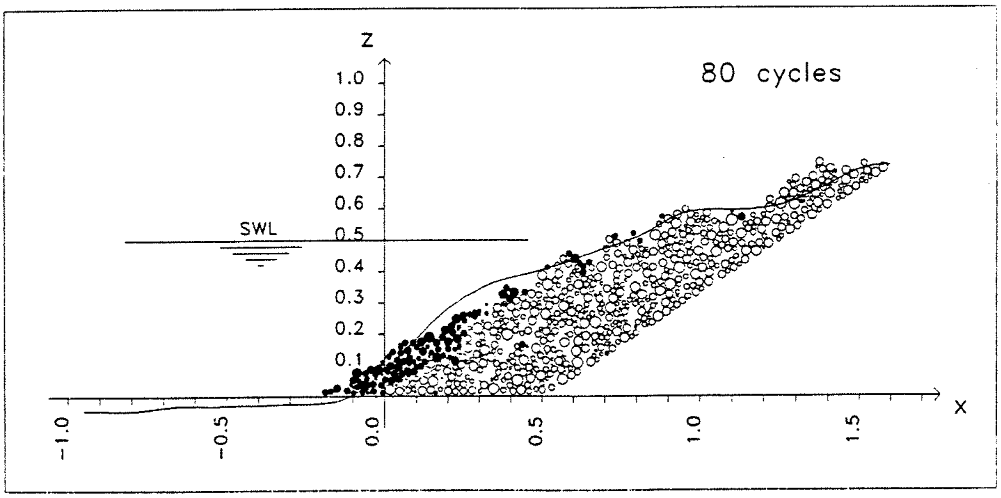


Figure 14 *Numerically constructed and reshaped berm compared with the initial and final profile after reshaping from the 3D tests at Danish Hydraulic Institute.*

2D Numerical Model

A sensitivity analysis was performed with the 2D numerical model, Skylla. The physical model layout of the tests performed at Delft University of Technology was used. Parameters that have been varied are the size of the top layer, the size of the core material, the layout of the core and the viscosity. The sensitivity of the maximum velocities along the seaward slope (just outside) to these parameter variations was studied. A homogeneous berm breakwater, a berm breakwater with a traditional core and a berm breakwater with an impermeable core were studied; as expected, the highest velocities occur for the structure with the smallest core material. Comparison between computations with a traditional core and computations with an extended core underneath the horizontal berm showed little differences in the velocities outside the structures. Similar results were also obtained with the 1D numerical model (van Gent, 1996). In general, it can be concluded that the computational results indicate that an extension of the core might have little effect on the reshaping process.

Parameter Analysis

In order to develop a set of parameters to describe the berm profile development due to wave attack, a database with results from about 550 model tests was established by the University of Bologna.

All the information regarding the 2D and 3D tests are reported in a database organised as follows:

- reference to the source of data
- reference to the initial profile
- id number of the test
- structural parameters related to the rock: D_{n50}, D_{n15}/D_{n85}, Δ
- parameters related to waves: $H_s, H_{2\%}, H_0, T_m, H_0T_0, s_m$ and the shoaling coefficient
- number of waves
- parameters related to the cross-section: crest freeboard, berm width, angle of structure slope, and depth in front of structure
- parameters characterising the profile after each wave attack

The berm breakwater profiles were schematised by using the six parameters previously established by van der Meer (1992), all of which are related to the local origin placed at the intersection between the initial profile and the water level ($l_r, h_c, l_c, h_s, l_s, tg\beta$).

All the wave heights were transferred to the water depth in front of the structure using the formula of Goda. Previous analysis by Lamberti et al (1994) showed that $H_{2\%}$ was the best value for the design, rather than H_s, so the non-dimensional parameters related to the waves (H_0, H_0T_0, s_m) were re-arranged with $H_{2\%}$.

In order to examine the relationship among a set of six correlated variables (profile parameters), it was useful to transform the original set of variables into a new set of uncorrelated variables called principal components. The technique for finding this transformation is called Principal Component Analysis. The usual objective of the analysis is to see if the first few components account for most of the variation in the original data and thus can be used in describing the phenomena without any significant loss of information and bringing the highest simplification compatible.

The first component found from the available set of data is highly and positively correlated to the five lengths describing the profile, and represents the general erosion process and is a function of the intensity of the wave attack. The second component is highly correlated to the slope of the step and is a function mainly of the depth at the toe. The third, which describes, together with the second, the difference in the two parts of the active profile, above and under the SWL, is a function of the initial slope and of the intensity of the wave attack. The first component describes 69 per cent of the variance, and the first with the second and the third describe 95 per cent of the variance. The conclusion of the factor analysis is that the variables describing the phenomena can be reduced to three components.

The three components are a function of the number and the intensity of the waves (H_0 and s_m), of the water depth and of the initial slope of the berm. When the three components are evaluated, it is possible to estimate the six parameters describing the evolution of the profile through an inverse transformation. Knowing the parameters, the reshaped profiles can be reconstructed with respect to the conservation of volume during the wave attacks.

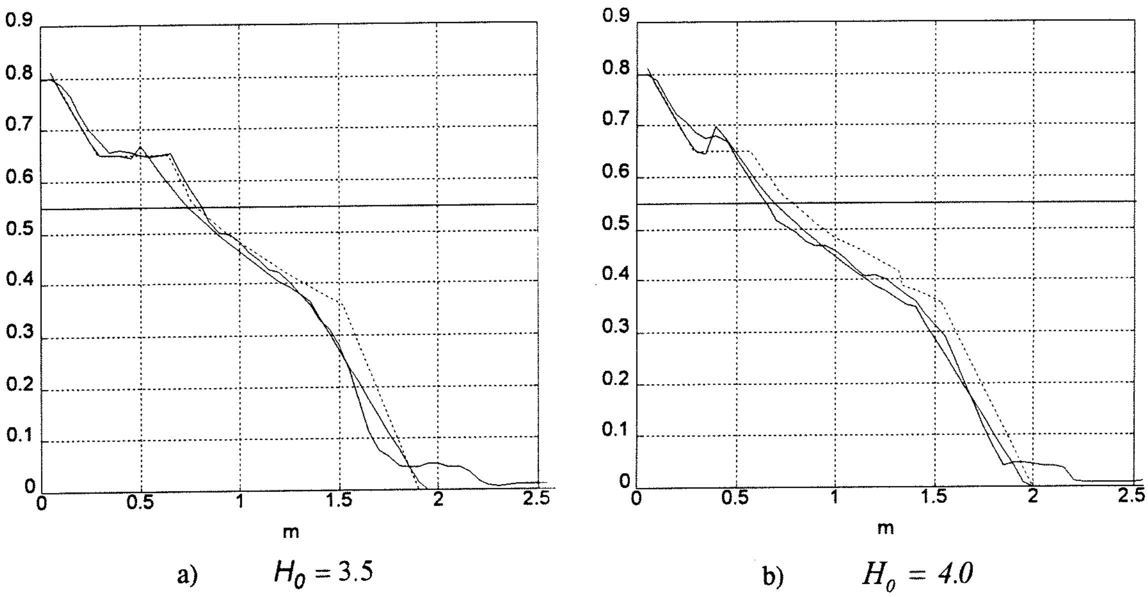


Figure 15 *Results from parameter analysis compared to results from model tests (curve including breakwater toe) and the predictive method of van der Meer (dushed line).*

The reshaping of the breakwater based on the PCA was compared with experimental profiles from the 3D model tests run at Danish Hydraulic Institute in 1995, see Juhl et al (1996). The test programme carried out was a sequence of six wave attacks with increasing intensity (from H_o=1.5 to H_o=4.0, s_m=0.05). The profiles were compared also with the predictive method based on the equations derived by van der Meer (1992), see **Figure 15**.

The output from the principal component analysis is satisfactory for initial berm reshaping occurring for small wave heights and when significant reshaping takes place for large wave heights. In cases in which the berm still exists and is wide, the proposed method does not properly describe the dependence of the shape of the initial profile and thus overestimates the reshaping for modest wave attack.

Influence of Permeability and Stone Gradation

The influence of permeability and stone gradation on the reshaping and wave overtopping of berm breakwaters was studied by flume model tests at the Danish Hydraulic Institute, see Juhl (1997). Another aspect studied was the difference in reshaping comparing a traditional berm breakwater constructed of two stone classes and a more stable Icelandic type of berm breakwater with the largest stones used as an armour layer covering a part of or the entire berm.

A total of twelve series of model tests were carried out in a wave flume covering studies of the effect of two stone gradations, three permeabilities, two crest and berm elevations, three berm breakwater solutions based on the Icelandic experience, and two wave steepnesses.

The profiles used in the 12 test series are shown in **Figure 16**. The water depth in front of the berm was 0.25 m for all tests. Test series 1 to 8 were carried out for comparing the reshaping of a berm breakwater constructed of two stone classes with the reshaping of a more stable Icelandic type of berm breakwater with the largest stones armouring the berm. Test series 1 to 4 were made with relatively high-crested breakwaters not allowing wave overtopping. Three alternative berm breakwaters of the Icelandic type were tested and compared to tests for a berm breakwater consisting of two stone classes (reference profile). For the subsequent test series, the Icelandic type studied was a profile similar to profile 1. Test series 5 to 8 were made with more low-crested breakwaters with the crest elevation and berm width adjusted to the difference in wave steepness (S_{om}=0.03 and 0.05). Test series 9 and 10 were made for studying the influence of stone gradation, and test series 11 and 12 for studying the influence of permeability (25 per cent of fines added to the berm material).

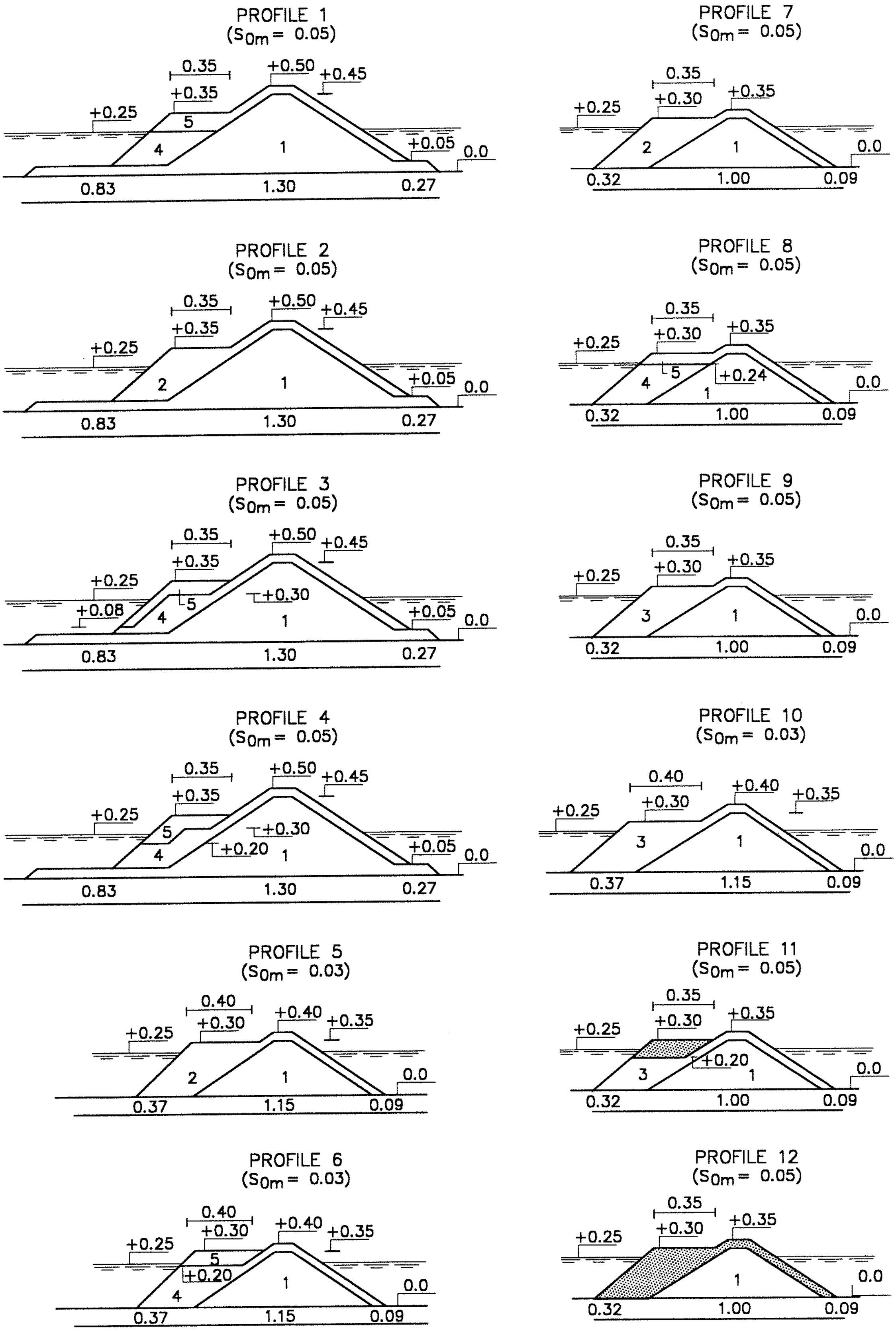


Figure 16 *Tested berm breakwater profiles.*

The berm breakwaters were constructed of two or three stone classes, ie one for the core and scour protection and one or two for the berm, crest and rear side protection. The reference case is a berm breakwater constructed of two stone classes, a relative wide stone gradation for the berm, $D_{n,85}/D_{n,15}$=1.80 (stone class 2) and a core (stone class 1). In testing of the Icelandic type of berm breakwaters, the berm stones were separated into two classes, the lower fraction to be used for the berm (stone class 4) and the higher fraction to be used as an armour layer (stone class 5). Stone class 3 was a more narrow stone gradation with $D_{n,85}/D_{n,15}$=1.40.

Influence of introducing an armour layer (Icelandic type)
Three alternative Icelandic type berm breakwater profiles were tested and compared to tests with a traditional berm breakwater profile consisting of two stone classes (test series 1 to 4). **Figure 17** shows the reshaped profiles after testing with H_o=4.0. All three Icelandic type breakwaters showed significant less erosion volume and berm recession compared to the traditional berm breakwater. The profile resulting in the smallest erosion volume and berm recession was profile 3 (an armour layer at the top and at the front of the berm) followed by profile 4 (armour layer placed as a hammer head), whereas profile 1 (armour at the top of the berm) showed a little less effect, but has an advantage in construction.

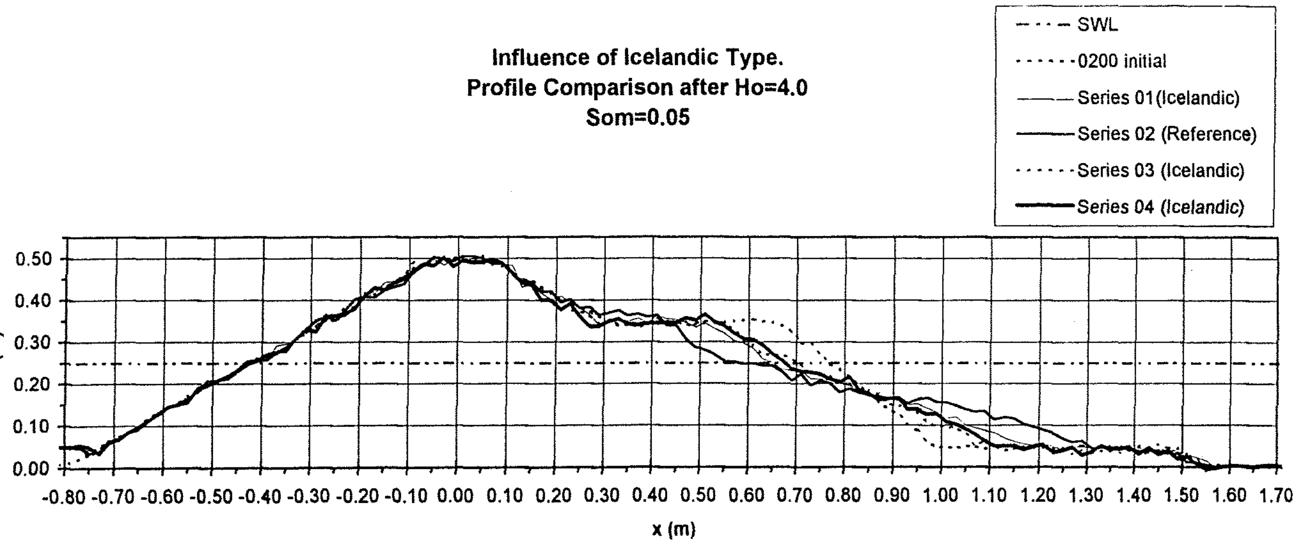


Figure 17 *Berm reshaping for a traditional berm breakwater and three alternative Icelandic type of berm breakwaters.*

Also the other tests made for studying the influence of an armour layer showed a reduction in the erosion volume and recession of the berm. The increased stability implies that the overall dimensions can be reduced.

An increase in the berm freeboard is associated with an increased berm volume and was found to result in less recession of the berm.

Influence of stone gradation

Model tests were carried out with two stone gradations, a wide stone gradation having $D_{n,85}/D_{n,15}$=1.80 (test series 5 and 7) and a more narrow having $D_{n,85}/D_{n,15}$=1.40 (test series 9 and 10). Results of test series 7 and 9 are presented in **Figure 18**. It is found that the wider stone gradation resulted in larger erosion volume and berm recession. Also the rear side stability is less for the wider stone gradation. In a wide stone gradation, the smaller stones will partly fill the voids between the larger stones resulting in a reduced permeability, which for the considered stone gradations are expected to cause increased erosion volume and berm recession due to decreased energy dissipation in the berm.

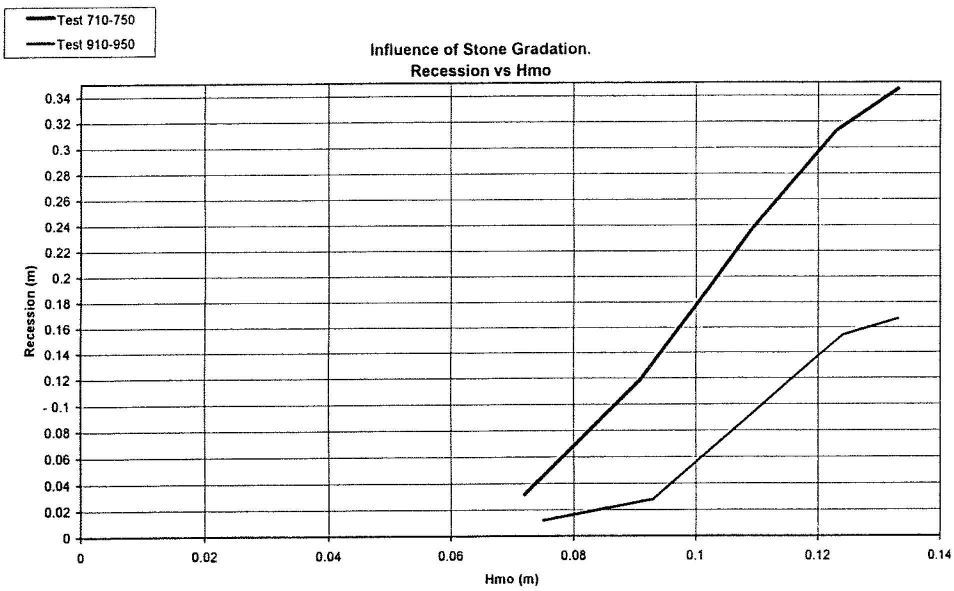


Figure 18 *Influence of stone gradation on berm recession for a wave steepness of S_{om}=0.05.*

Influence of permeability

Wave run-up and overtopping conditions are significantly influenced by the presence of fine material in the berm material reducing the permeability and thus the energy dissipation, which is one of the main features of berm breakwaters. The influence of the permeability was studied in two test series with finer material added either to the top of the berm or to the entire berm constructed of stones with the narrow gradation, $D_{n,85}/D_{n,15}$=1.40. An increase in the erosion volume and berm recession was observed by adding the finer material to the top of the berm (test series 11). Adding finer material to the entire berm (test series 12) lead to a further increase in the erosion volume and berm recession, which exceeded the berm width. Further, a significant increase in overtopping was found, resulting in severe damage to both crest and rear side. **Figure 19** shows the influence on the berm recession by reducing the permeability of the berm.

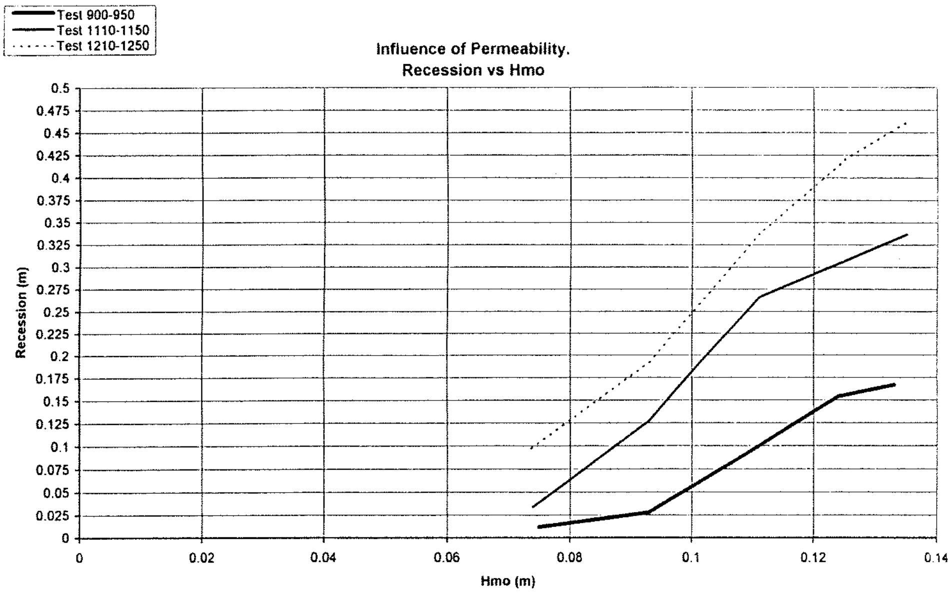


Figure 19 *Influence of permeability on berm recession.*

Wave overtopping and reflection conditions

The waves generated by wave overtopping were found to slightly decrease by going from a stone gradation with $D_{n,85}/D_{n,15}$=1.8 to a more narrow gradation with $D_{n,85}/D_{n,15}$= 1.4, and to increase by reducing the permeability.

The reflection from a berm breakwater is dominated by the slope of the reshaping profile, and thus the reflection from the Icelandic type of breakwaters was larger than for a traditional berm breakwater.

Scour Protection

Local scour can occur at a breakwater constructed on a sandy seabed and may endanger the overall stability due to sliding of the main armour layer if the toe and scour protection are failing. The scouring pattern is a function of the water depth, wave conditions, sediment characteristics, and breakwater configuration and reflection characteristics as described by Arneborg et al (1996). A simultaneous current flow at the breakwater will significantly influence the scouring.

Scouring in front of a berm breakwater constructed without a sufficient scour protection layer may result in berm stones to be moved into the scour hole, which will lead to further reshaping of the protecting berm. Physical 2D model tests were made for qualitatively studying the scour development in front of a berm breakwater. A total of four test series were carried out in order to study the influence on the scouring and breakwater performance of varying wave steepness and of two types of scour protections, see Juhl and Archetti (1997).

A berm breakwater with a high crest and a wide berm was used for all four test series. The tests were made with a water depth of 0.25 m (and a few tests with a water depth of 0.20 m), ie the largest waves were breaking in front of the breakwater. Test series 1 and 2 were made for studying the scouring for a profile without any scour protection exposed to two wave steepnesses. Test series 3 was made with a profile including a scour protection layer extending 0.50 m in front of the berm, and test series 4 was made with the scour protection material placed as a 0.10 m extension of the berm, the idea being that the material under the exposure of the first waves will reshape into a toe and scour protection.

A summary of the findings is presented below:

- The scouring was found to increase by decreasing the wave steepness from 0.05 to 0.03. A significant increase in the extent of the scour hole was found by decreasing the wave steepness from 0.03 to 0.02.
- Subsidence of berm stones into the sandy seabed was found for the profiles without a scour protection layer.
- Introduction of a scour protection layer moved the scour hole out in front of this and no subsidence of berm stones into the sandy seabed was found. Consequently, the reshaping of the berm and thus also the berm recession were reduced.
- During reshaping of the scour protection material placed as an extension of the berm, some of this finer material was mixed into the berm material. The resulting reduced permeability led to increased wave run-up and overtopping.

3D Model Tests

The major task within the research project was the 3D model tests for studying the stability of the breakwater roundhead and adjacent trunk section. Deep water tests were carried out at Danish Hydraulic Institute and shallow water tests at SINTEF NHL. Model tests for studying the influence of storm duration, short-crested waves and rock shape on berm reshaping and longshore transport were made at Aalborg University.

Deep Water Tests, Roundhead

For berm breakwaters as compared with traditional rubble mound breakwaters, special measures have to be taken for the breakwater roundhead. If stone displacements occur on a roundhead, the stones will be moved in the wave direction and loose most of their stabilising effect.

A total of six test series were carried out in a wave basin at Danish Hydraulic Institute for studying the development of berm breakwater roundheads, see **Figure 20**. Five wave incidence angles were tested together with two wave steepnesses. All tests were carried out with irregular long-crested waves, and each test series consisted of five tests for reshaping of the breakwater (H_0=2.0, 2.5, 3.0, 3.5 and 4.0).

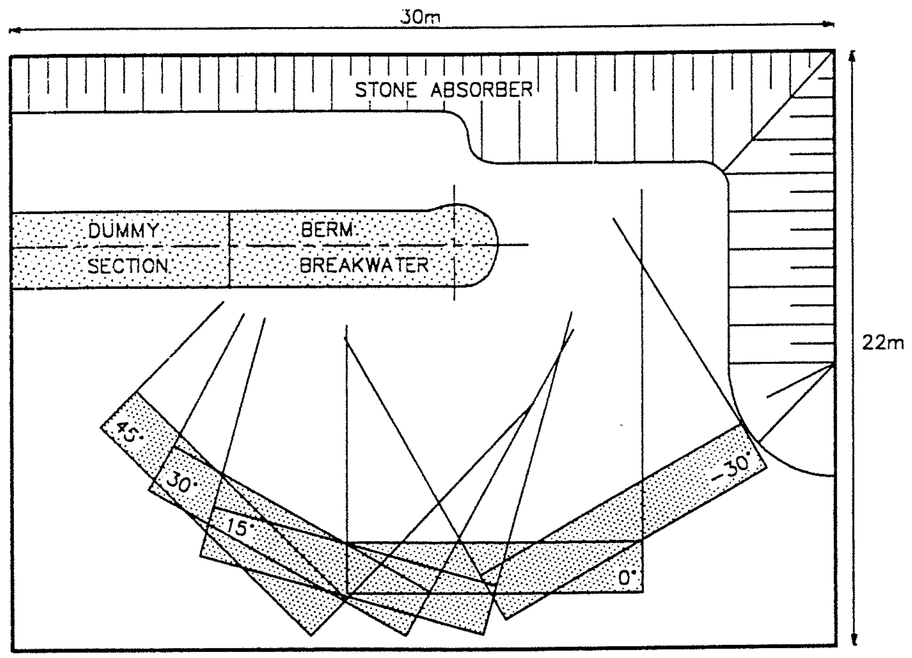


Figure 20 *Model plan, including positions of the wave generators.*

The berm breakwater was constructed with two stone classes, ie one for core and scour protection and one for berm, crest and rear side protection as shown in **Figure 21**.

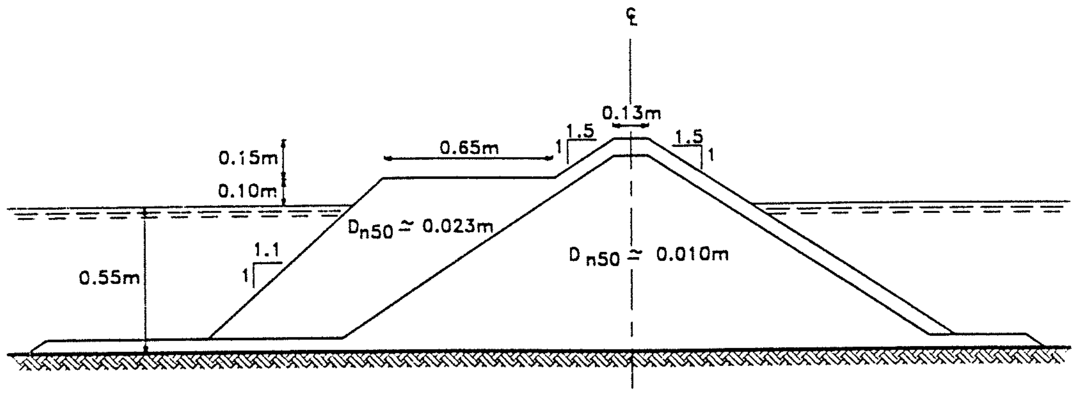


Figure 21 *Cross-section of the initial profile for the trunk. The roundhead was made by rotating the profile around the centreline.*

The measurements included a total of 38 profiles along the breakwater which were subsequently interpolated into a 3D representation of the breakwater. This formed the basis for detailed calculations on the profile development, recession of the berm, eroded and deposited volumes and transport of stones in arbitrary sections of the breakwater.

Recession

Plotting the recession measured after H_0 =4.0 for all wave directions as function of the relative angle, see **Figure 22**, shows that the recession pattern follows the wave direction, and that maximum recession occurs at an area directly exposed to the waves. It is found that the maximum recession on the head occurs for wave direction 0°, where the recession reaches the initial berm width for H_0 between 3.5 and 4.

Comparing the maximum recession found at the head with the recession on the trunk, see **Figure 23**, it is found that only in the case with wave direction 45^0 the maximum recession on the head is equal to or less than the recession on the trunk section. In the case with wave direction -30^0, the recession on the head is up to 75 per cent higher than for the trunk section. For head on waves, the maximum recession on the head is 50 per cent higher than the recession on the trunk.

When analysing the maximum recession as function of the wave direction, it can be seen that the maximum recession on the head is increased by a factor of up to 2 comparing the results for wave directions 45° and 0°, whereas the recession on the trunk is increased by a factor of about 1.4. The recession on the head is thus more sensitive to changes in the wave direction than the recession on the trunk.

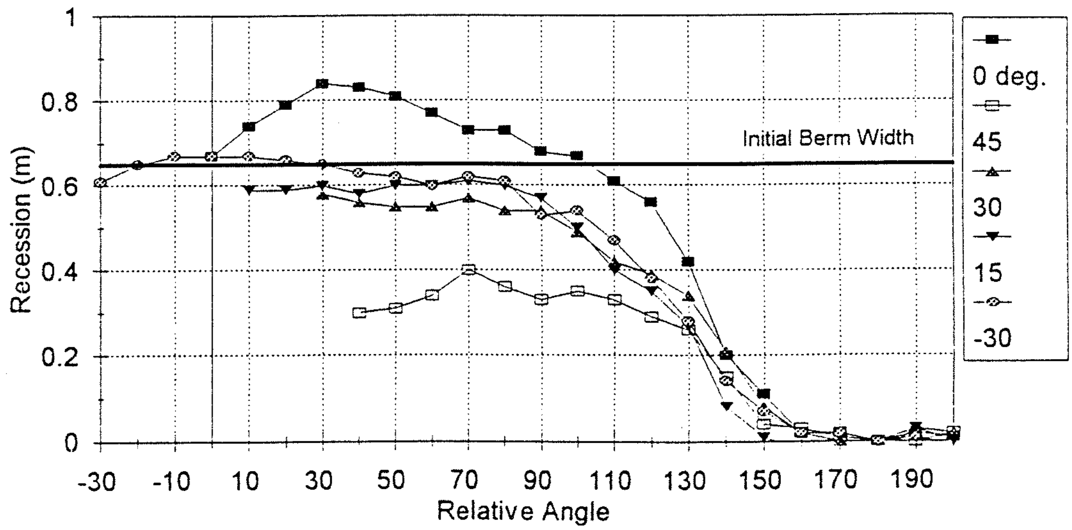


Figure 22 *Recession. All wave directions. After H_0=4.0, S_m=0.05.*

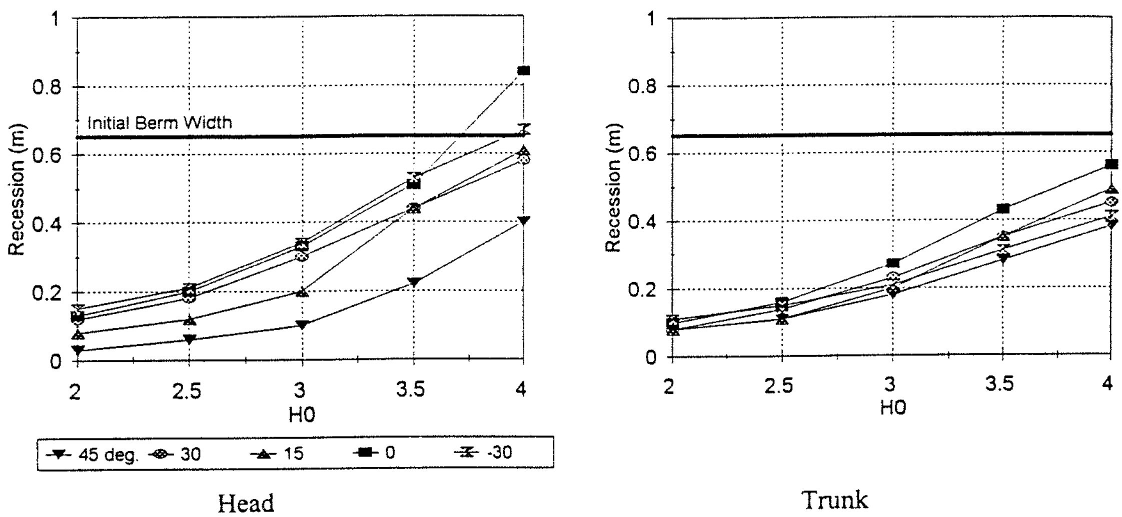


Figure 23 *Maximum recession on head, respectively trunk*

The influence of the wave steepness was investigated for wave direction -30°. The results showed a strong influence by the wave steepness, as the maximum recession on the head for S_m=0.03 was almost twice the recession found for S_m=0.05. On the trunk, the influence was less pronounced. Analysis showed that a good agreement between the two data sets (S_m=0.03, respectively 0.05) was obtained when plotting the recession as function of H_0T_0, especially for the trunk section. T_o is given by $T_o=T_m \sqrt{g / D_{n,50}}$.

Erosion and deposition

Figure 24 presents the erosion and deposition pattern around the breakwater head, after H_0=4.0 for all wave directions as function of the relative angle, again showing that the reshaping pattern follows the wave direction closely. A pronounced deposition area at the rear of the roundhead and a relative wide area of erosion are found. The reshaping of the head accelerates for H_0T_0>80-100.

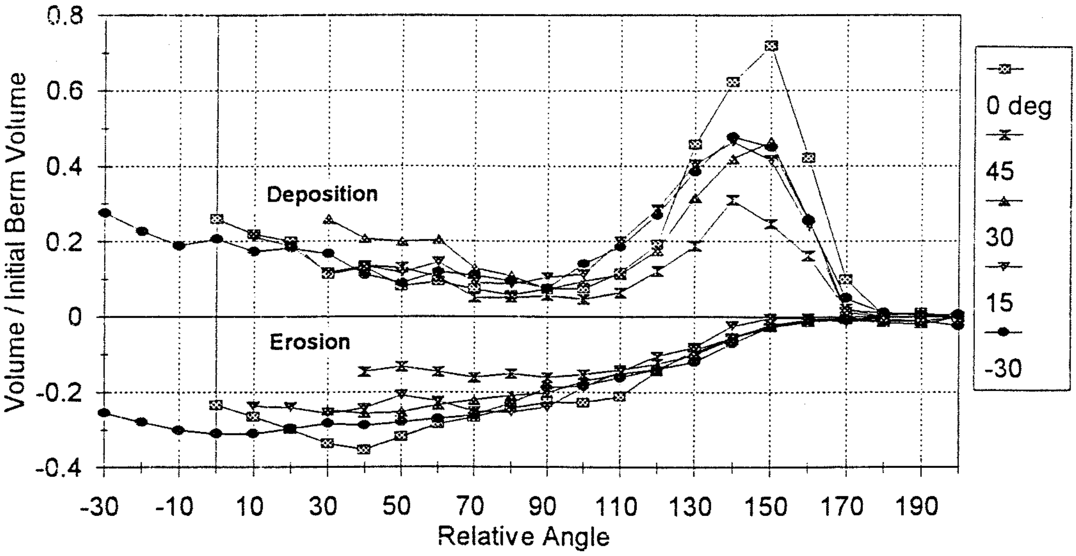


Figure 24 *Erosion and deposition after H_0=4.0. All wave directions.*

Stone transport

In **Figure 25**, the stone transport for H_0=4.0 is plotted for all wave directions as function of the relative angle, showing that the maximum transport takes place at the sections 110°-130° relative to the wave direction. The maximum stone transport on the head occurs with head on waves (0°) being up to three to four times higher than for wave direction 45°.

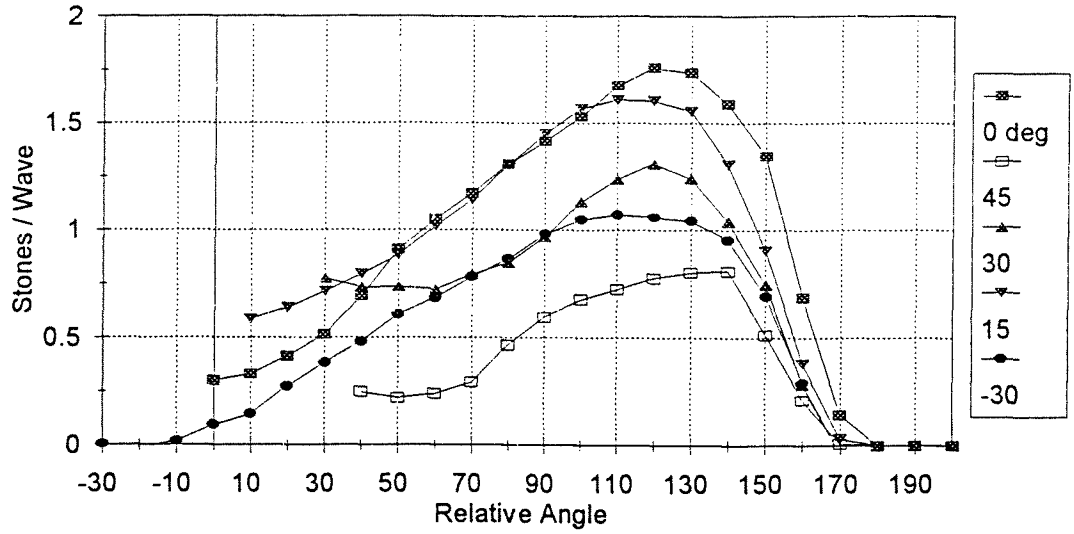


Figure 25 *Stone transport for H_0=4.0. All wave directions.*

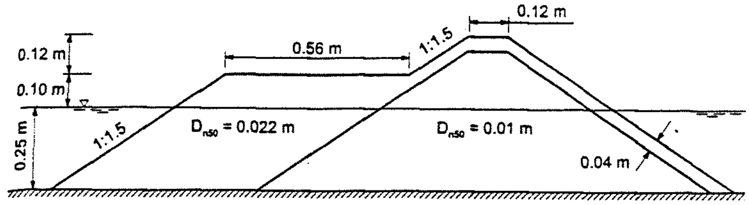
Shallow Water Tests, Roundhead

SINTEF NHL carried out 3D tests on a berm breakwater head in shallow water. These tests were supplementary to the deep water 3D tests carried out at Danish Hydraulic Institute. The SINTEF tests were carried out for the three different cross-sections shown in **Figure 26.**

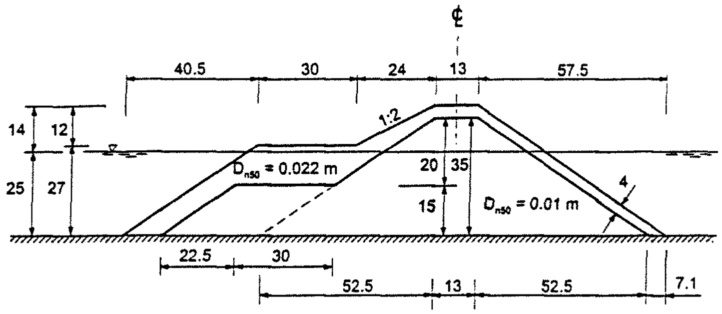
Breakwater head A is the closest to the section tested at Danish Hydraulic Institute, except that the outer slope of the initial berm was 1:1.1 for the DHI tests and 1:1.5 for the SINTEF tests.

The SINTEF tests were carried out with a target wave steepness in deep water of $s_{0m}=2\pi H_s/gT_z=0.05$. Wave measurements in shallow water showed that the zero upcrossing period became smaller in shallow water than in deep water due to wave breaking, and hence the wave steepness became larger in shallow water than in deep water.

In general, the recession in shallow water was much less than in deep water. **Figure 27** shows the cross-section tested, Breakwater head A, before and after completing the tests with a maximum value of $H_s/\Delta D_{n50}=3.85$ (the maximum value obtainable in shallow water due to wave breaking). While during the deep water tests at Danish Hydraulic Institute the whole width of the berm was reshaped, only a fraction of the berm breakwater was reshaped during the shallow water tests at SINTEF NHL. One reason for this is the breaking of the largest waves in shallow water, which leads to smaller wave forces and thus less reshaping. Another reason is the deposition volume's dependency of the water depth, and finally the flatter initial berm slope of the shallow water profiles tested.



Berm breakwater head A.



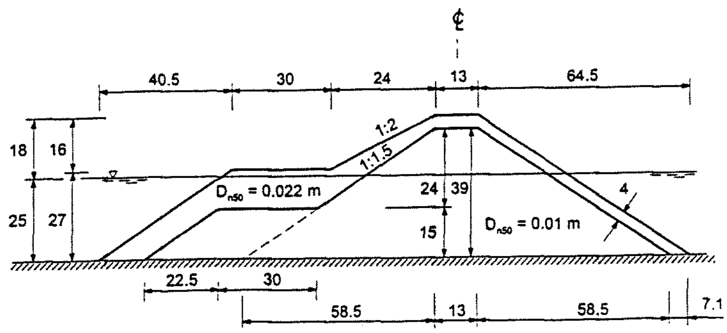


Figure 26 *Breakwater cross-sections used in the 3D tests at SINTEF.*

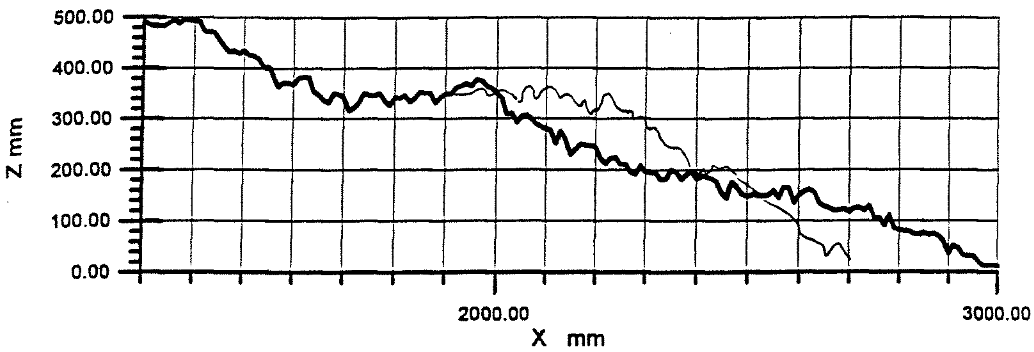


Figure 27 *Breakwater head A. Cross-section before and after testing.*

Trunk Section

A berm breakwater trunk section exposed to head-on waves can hardly be destroyed unless it is damaged by overtopping waves, whereas for oblique waves the stones can move along the breakwater. A study was made to describe the influence of wave obliquity on the profile shape, on the initiation of longshore transport and on the longshore transport rate at the trunk section both during and after the profile reshaping, see Alikhani et al (1996).

Model testing of a trunk section exposed to oblique waves were carried out at the Danish Hydraulic Institute together with the model tests for studying roundhead stability. Basin tests were made at Aalborg University for studying the influence of storm duration and of short-crested waves. Assessment of the stone movements was made based on visual observations and photos taken after each test run. Observations were made both during the reshaping process involving a large number of stone movements and after the reshaping. In order to facilitate observations of the threshold conditions and the longshore transport rate, all the berm stones in a 1.0 m and 0.6 m, respectively for the two test series, wide section of the trunk were painted.

Threshold of stone movement
In general, the model tests showed that the largest transport distance occurs for an angle of wave attack, β, of 45° with a decreasing tendency for smaller and larger angles. It was found that the threshold values for longshore transport can be expressed as $H_o T_{op} > 50/\sqrt{(\sin 2\beta)}$ during the reshaping phase, and $H_o T_{op} > 75/\sqrt{(\sin 2\beta)}$ after the reshaping phase.

Longshore transport
An equation for calculation of the longshore transport rate under oblique wave attack was established considering that the longshore component of the incident wave energy is one of the most important parameters:

$$S = 0.8 \cdot 10^{-6} \sqrt{\cos\beta} \left(H_o T_{op} \sqrt{\sin 2\beta} - 75 \right)^2$$

The equation is calibrated to give the maximum longshore transport for a wave angle of 45° and zero transport for head-on waves and waves propagating parallel to the breakwater axis. A graphical presentation of the longshore transport rate after reshaping found from the tests made at Danish Hydraulic Institute is shown in **Figure 28** together with the derived equation. The test results and the equation show comparable relationships of the wave conditions and angle of wave attack.

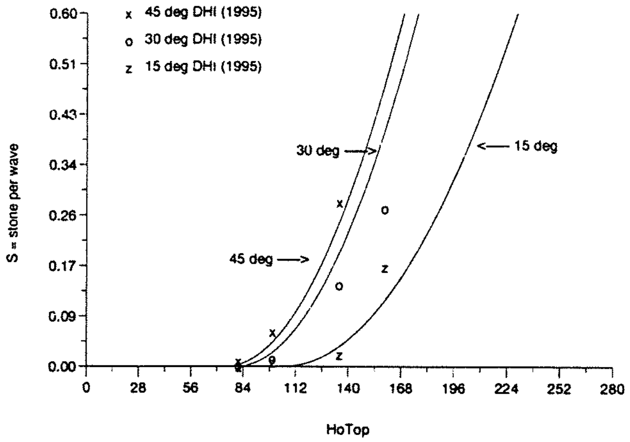


Figure 28 *Longshore transport, in the form of stones per wave, S, as function of* H_oT_{op}.

Tests with directional waves have shown that the longshore transport is much lower than for long-crested waves. For example for s=10, where s is a parameter that controls the angular distribution in the cosine power spreading function, the longshore transport was ¼ of the case for long-crested waves (wave angle 60^o and H_o=4.0).

Long duration tests with registration of the number of moved stones after each 1,000 waves showed that the transport rate decreases in an exponential way with increasing number of waves.

Influence of stone shape

A series of wave basin tests were made with a berm breakwater trunk for studying the influence of stone shape on longshore transport, see Frigaard et al (1996). All tests were run with an angle of wave attack of 30^o, and the breakwater was built as a homogeneous structure. Four type of stone shapes were used, ie rounded, normal, flat and mixed stones. All four types followed the same gradation curve and had a nominal diameter of $D_{n,50}$=17.9 mm.

Longshore transport was initiated for a stability number, N_s, of approximately 3.0, and it was found that the transport for the flat stones was three to five times higher than for the rounded stones and the transport for the normal stones being in between these. Any measurable differences in the reshaped profiles with different stone types were not observed in the tests.

Acknowledgements

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Roundhead Stability of Berm Breakwaters

Jørgen Juhl[1], Amir Alikhani[2], Peter Sloth[1], Renata Archetti[3]

Abstract

Three-dimensional (3D) model tests were carried out for studying the stability of a berm breakwater roundhead and the adjacent trunk section. The present paper describes the influence of the wave incidence angle and wave steepness on the roundhead stability. The test results are described in terms of profile development, recession of the berm, eroded and deposited volumes and the transport of stones during reshaping. Results from analysis of the influence of wave obliquity on profile shape, initiation of longshore transport and longshore transport rate at the trunk section are presented in Alikhani et al (1996).

Introduction

For berm breakwaters as compared with traditional rubble mound breakwaters, special measures have to be taken for the breakwater roundhead. If stone displacements occur on a roundhead, the stones will be moved in the wave direction and will loose most of their stabilising effect. A point of special concern is whether, and under which conditions, a berm breakwater roundhead after some initial reshaping may develop into a stable shape that is not subject to continued erosion, or at least such slow erosion that it may be acceptable for a permanent structure.

The major part of the research on berm breakwaters has concentrated on the reshaping of the seaward side of the trunk under perpendicular wave attack. In 1988, van der Meer pre-

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sented results from a series of flume model tests with gravel beaches and rock slopes which led to a set of parameters equations for assessing the profile development of berm breakwaters. Also Kao and Hall (1990) presented results on various aspects influencing the stability of berm breakwaters. Analysis of flume tests concentrating on the rear side stability was presented by Andersen et al (1992).

Only a little research has been made to study the stability of berm breakwater roundheads. Burcharth and Frigaard (1987) described results from model tests with reshaping breakwaters exposed to head on waves and waves having a wave incidence angle of 15^{o} and 30^{o}. The analysis concentrated on the stability of roundheads and trunk erosion in oblique waves, and some preliminary recommendations for berm breakwater trunks and heads were given.

Jensen and Sørensen (1992) presented results from a 3D model study of a berm breakwater including trunk and roundhead. Comparison of the profile development on the trunk and head was made.

Van der Meer and Veldman (1992) made a discussion on roundhead stability based on analysis of results from a series of wave basin tests.

Description of Model Tests

Model Set-Up

The model tests were carried out in a 22 x 30 m wave basin at the Danish Hydraulic Institute, see Figure 1, which also shows the various positions of the two 5.5 m wide movable wave generators for generating long-crested irregular waves. A stone absorber was placed along the basin boundary in order to minimise wave reflections. The berm breakwater profile used for the trunk section is shown in Figure 2. The roundhead was constructed by rotating the profile around the centreline.

The berm breakwater was constructed from two stone classes, ie one for the core and the scour protection and one for the berm, the crest and the rear side protection. The core material had a nominal diameter of $D_{n,50}$=0.010 m. A relative wide stone gradation was used for the berm, ie $D_{n,85}/D_{n,15}$= 1.80 with a nominal diameter of $D_{n,50}$=0.023 m. The density of the stone material was ρ_s=2.68 t/m^3 and the density of the water was ρ_w= 1.00 t/m^3.

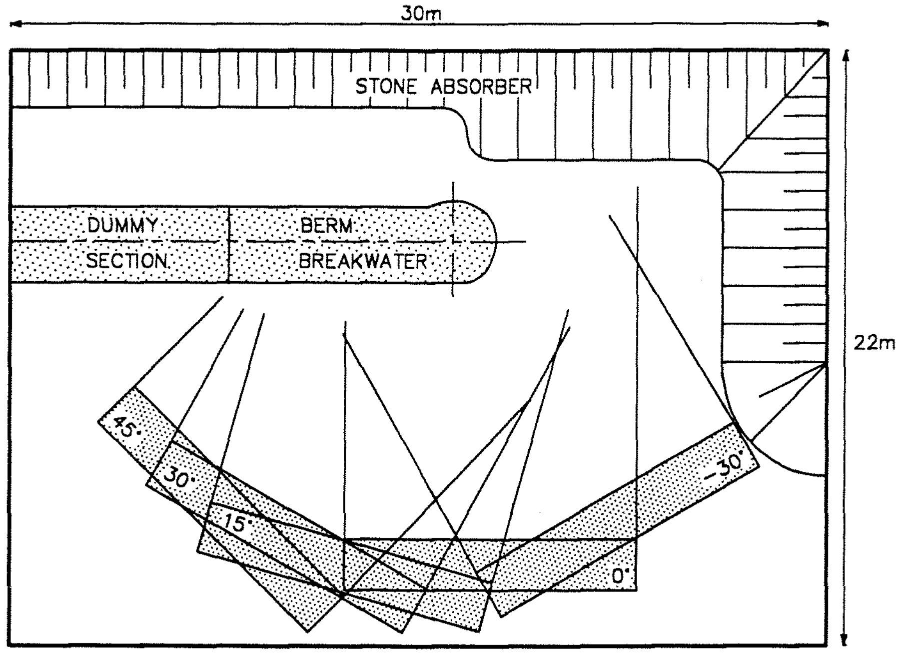


Figure 1. Model plan, including positions of the wave generators.

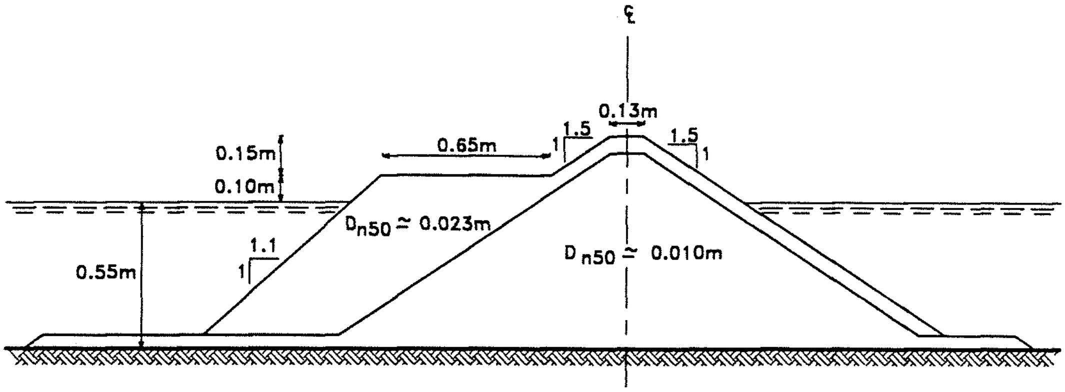


Figure 2. Cross-section of the initial profile for the trunk.
The roundhead was made by rotating the profile around the centreline

Test Programme

A total of six test series were carried out with irregular waves, covering five angles of incident waves with a wave steepness of S_m=0.05 (45°, 30°, 15°, 0° and -30°; where 0° is perpendicular to the trunk) and two wave steepnesses for -30° (S_m=0.03 and 0.05). The wave steepness, S_m, is given by $H_{m0}/L_{m0}=H_{m0}/(g/2\pi \cdot T_m^2)$, where H_{m0} is the wave height, L_{m0} is the deep water wave length, and T_m is the mean wave period.

Each test series consisted of five tests with a duration corresponding to 2,000 waves for reshaping of the berm breakwater ($H_0=H_{m0}/\Delta \cdot D_{n,50}$=2.0, 2.5, 3.0, 3.5, and 4.0; where Δ is the relative density, and $D_{n,50}$ is the nominal stone diameter) followed by four tests with a duration of 1,000 waves for studying the stone movements on the reshaped profile (H_0=2.5, 3.0, 3.5, and 4.0).

All waves were generated on basis of a Pierson-Moskowitz spectrum.

Measurements

The waves were measured in 14 positions in the wave basin by use of resistance type wave gauges. The incoming wave conditions were checked by five reference wave gauges placed in a way to minimise the effect of reflections. Spectral analysis and zero-crossing analysis were carried out.

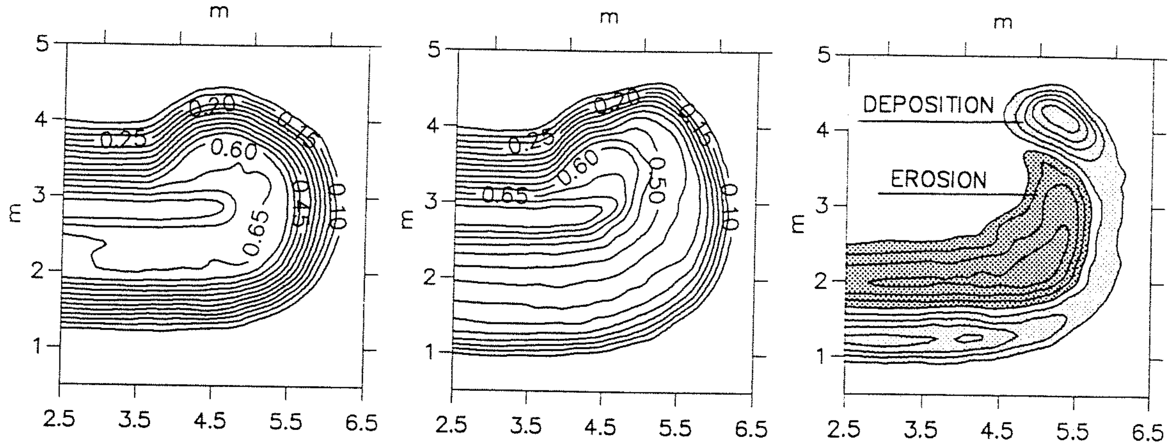
A total of 38 profiles along the 8.5 m long breakwater were measured after construction of the breakwater (initial profile) and after each test run. The profiling was made with a laser running on a beam across the breakwater trunk. The horizontal position of the laser running on the beam was measured by another laser, whereas the position along the breakwater was fixed manually. The profiles were measured for each 0.5 m along the trunk and for each 0.1 m at the roundhead.

The data from the 38 profile measurements were interpolated into a 3D representation of the breakwater from which it was possible to extract profiles in arbitrary cross-sections and to make contour plots of the breakwater as shown in Figure 3.

Definitions and Analysis

Sections and Wave Directions

The 3D representation of the test results was used to calculate the results for 10° sections on the breakwater head and 10 cm sections on the trunk with the convention as shown in Figure 4, which also shows the wave direction convention. This means that section 0 is the section covering 10° in continuation of the breakwater centreline and -90 is the section perpendicular to the centreline, where the trunk meets the roundhead.



Before testing

After testing (H_0=4.0)

Difference plot after testing (H_0=4.0)

Figure 3. Contour (m) plots of breakwater. Wave direction 0^o

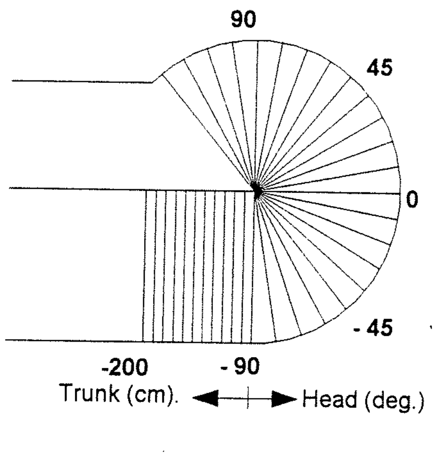
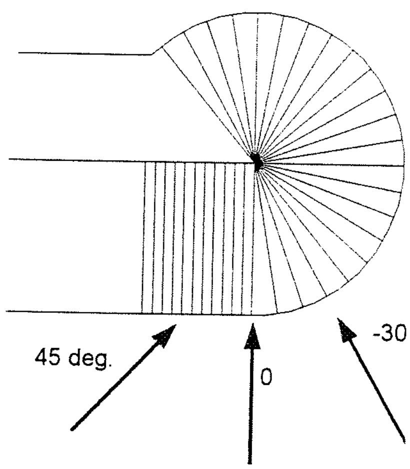


Figure 4. Profile definition.



Definition of wave direction

Relative Angle

When comparing results at the roundhead for different wave directions, a relative angle is introduced, defined as the angle between the incident waves and the considered section on the head, see Figure 5. For example, a relative angle of 0° corresponds to the section pointing directly towards the waves, whereas a relative angle of 90° corresponds to the section perpendicular to the wave direction.

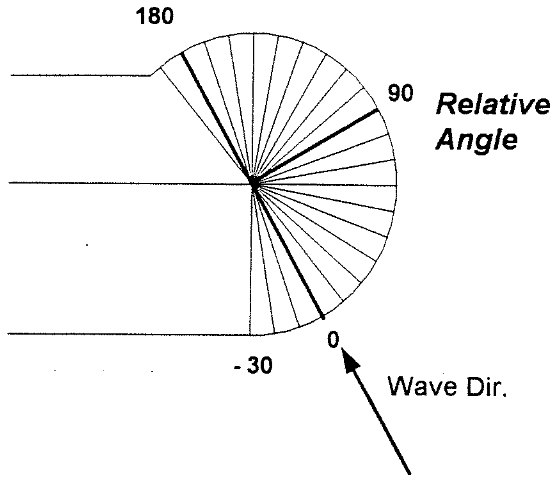


Figure 5. Definition of relative angle

Investigated Parameters

During the analysis, the following parameters have been investigated:

Recession

The recession is defined as the width of the berm eroded, see Figure 6. For the investigated breakwater profile with an initial berm width of 0.65 m, a recession of 0.65 m thus corresponds to a fully eroded berm with a directly exposed breakwater crest.

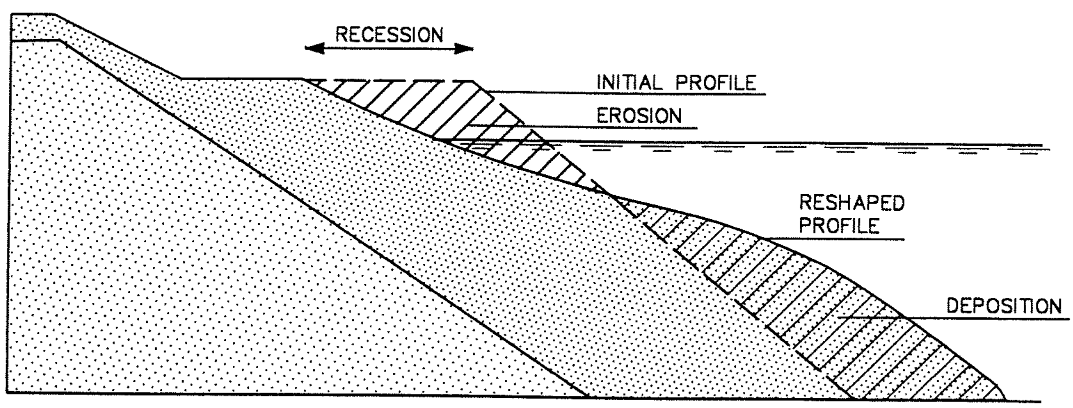


Figure 6. Definition of recession, erosion and deposition

Erosion
The erosion is defined as the volume eroded for the individual sections as shown in Figure 6. The erosion is presented as the eroded volume relative to the initial volume of berm material for the section. An erosion equal to 0.2 thus corresponds to the case where 20 per cent of the initial berm volume is eroded at the considered section. The erosion is presented as negative values.

Deposition
The deposition is defined as the deposited volume at the individual sections, as shown in Figure 6. As for the erosion, the deposition is presented as the deposited volume relative to the initial volume of berm material for each section. The deposition is presented as positive values.

Transported volume
This parameter is defined as the volume which has passed a given section since the start of the test series. The transport is defined positive in the anti-clockwise direction on the head (towards the rear of the head). On the trunk, the transport is positive towards the head. The volume is calculated directly from the measured profiles (ie including voids).

The transported volume is calculated by accummulating the volume changes from the rear of the roundhead - where no changes occur - clockwise around the head.

Stone transport
The stone transport is the estimated number of stones per wave passing a given section for a given H_0. The number of stones is estimated by correcting the calculated transported volumes for porosity and dividing by the average stone weight.

Presentation of Results

This section presents the results of the analysis of the profile measurements and concentrates on recession, erosion and deposition, transported volume and finally stone transport during the reshaping process.

Recession

Figure 7 presents the recession at the head found for wave direction 45°, respectively 0°, for increasing H_0.

It is seen that the recession for head on waves (wave direction 0°) is significantly higher than for 45°.

Plotting the recession measured after H_0 =4.0 for all wave directions as function of the relative angle, see Figure 8, shows that the recession pattern follows the wave direction, and that maximum recession occurs at an area directly exposed to the waves. It is found that the maximum recession on the head occurs for wave direction 0°, where the recession reaches the initial berm width for H_0 between 3.5 and 4.

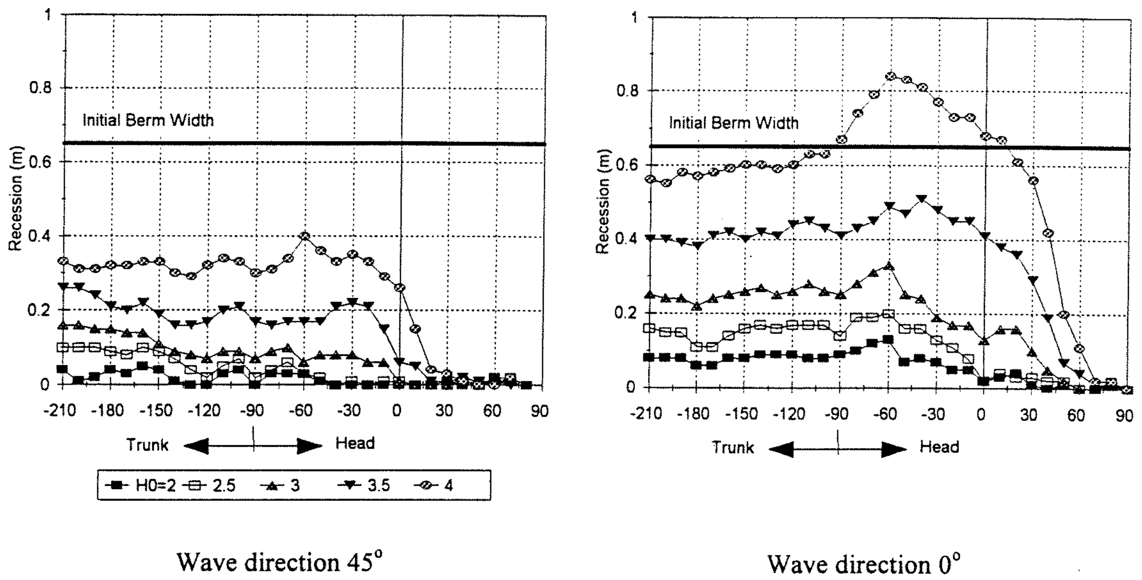


Figure 7. Recession of head and adjacent trunk

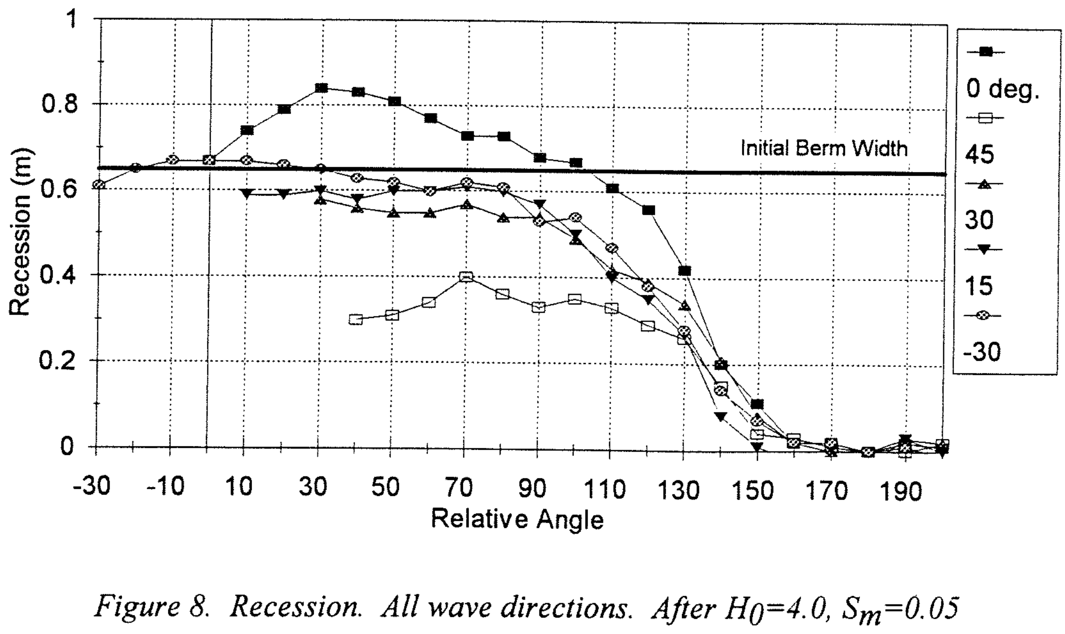


Figure 8. Recession. All wave directions. After H_0=4.0, S_m=0.05

Comparing the maximum recession found at the head with the recession on the trunk, see Figure 9, it is found that only in the case with wave direction 45° the maximum recession on the head is equal to or less than the recession on the trunk section. In the case with wave direction -30°, the recession on the head is up to 75 per cent higher than for the trunk section. For head on waves, the maximum recession on the head is 50 per cent higher than the recession on the trunk.

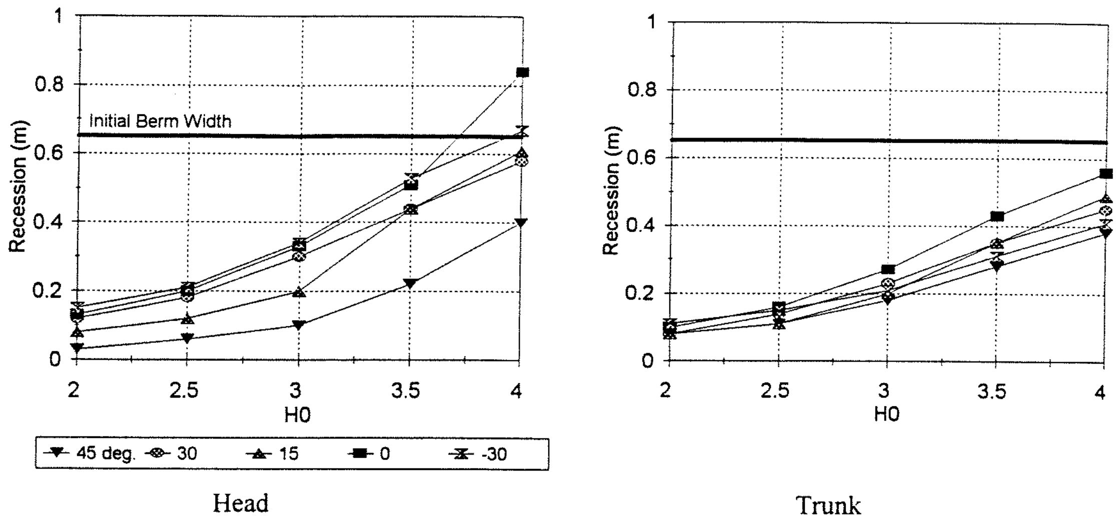


Figure 9. Maximum recession on head, respectively trunk

When analysing the maximum recession as function of the wave direction, it can be seen that the maximum recession on the head is increased by a factor of up to 2 comparing the results for wave directions 45° and 0°, whereas the recession on the trunk is increased by a factor of about 1.4. The recession on the head is thus more sensitive to changes in the wave direction than the recession on the trunk. For comparison, a cosine (cos) and a cosine^2 (cos^2) distribution of the recession found for wave direction 0° for H_0=4 were compared to the results, showing that the variation of the maximum recession with the wave direction, on the head, can be approximated by a cos^2 distribution for the higher H_0's. At the trunk, the results show a distribution closer to a cos distribution.

The influence of the wave steepness was investigated for wave direction -30°. The results showed a strong influence by the wave steepness, as the maximum recession on the head for S_m=0.03 was almost twice the recession found for S_m=0.05. On the trunk, the influence was less pronounced. Analysis showed that a good agreement between the two data sets (S_m=0.03 respectively 0.05) was obtained when plotting the recession as function of H_0T_0, especially for the trunk section. T_0 is given by $T_0=T_m \sqrt{g / D_{n,50}}$.

Erosion and Deposition

Figure 10 presents the erosion and deposition pattern around the breakwater head, after H_0=4.0 for all wave directions as function of the relative angle, again showing that the reshaping pattern follows the wave direction closely. A pronounced deposition area at the rear of the roundhead and a relative wide area of erosion is found.

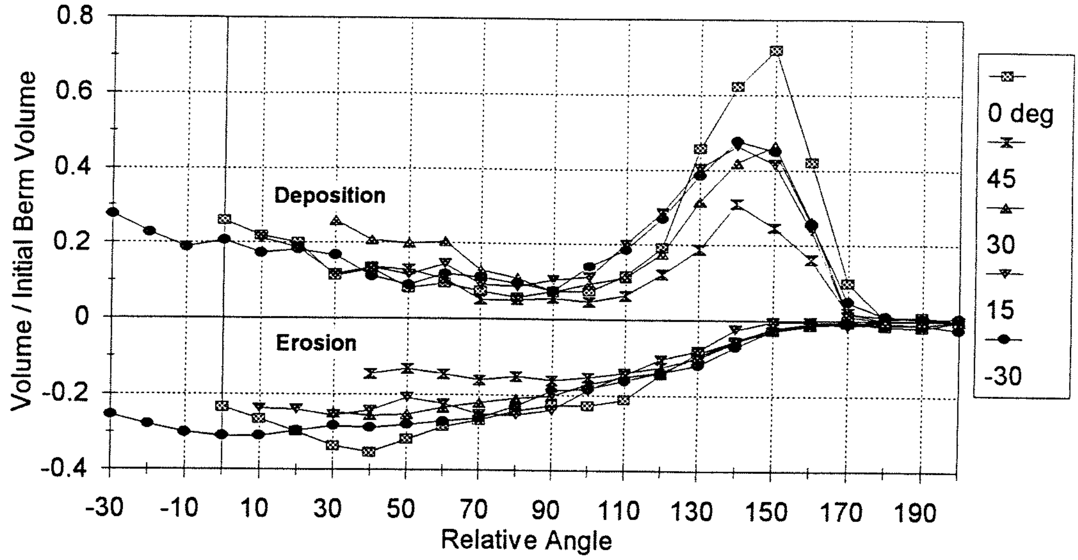


Figure 10. Erosion and deposition after H_0=4.0. All wave directions

For an infinitly long trunk, the erosion and deposition areas are equal (disregarding compaction), whereas for a roundhead stones will be transported towards the rear side of the roundhead. At the directly exposed part of the head, the erosion will be larger than the deposition, whereas the opposite will be the case at the rear of the head. This can also be seen from the contour plots presented in Figure 3 showing the height contours of the breakwater before and after the test series with wave direction 0^o together with a difference plot.

As seen from Figure 11, the erosion at the head accelerates for H_0>2.5. The influence of the wave steepness can be seen from Figure 12 presenting the maximum erosion and deposition as function of both H_0 and H_0T_0 for the two wave steepnesses, S_m=0.03 and 0.05, respectively (wave direction -30^o). Plotting the maximum erosion and deposition as function of H_0T_0 shows a better agreement between the two data sets than when plotting against the wave height parameter, H_0.

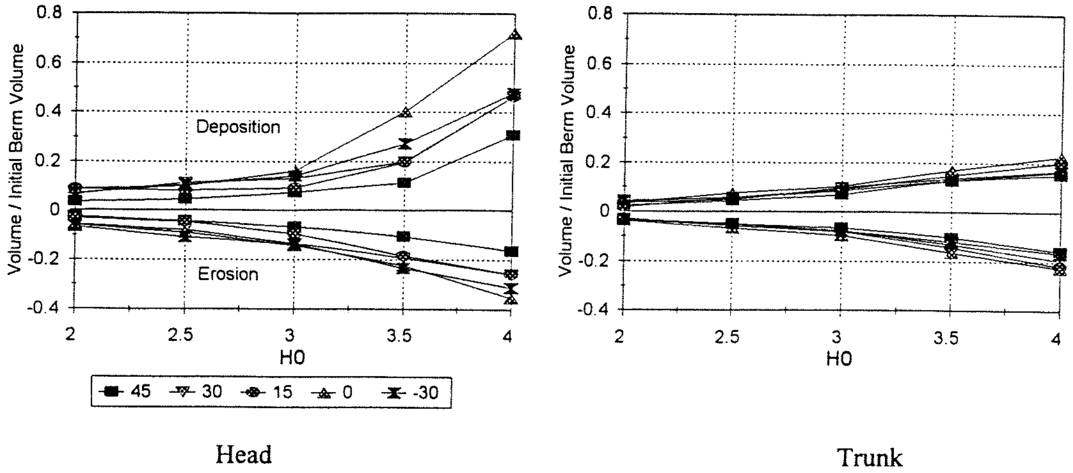


Figure 11. Maximum erosion and deposition

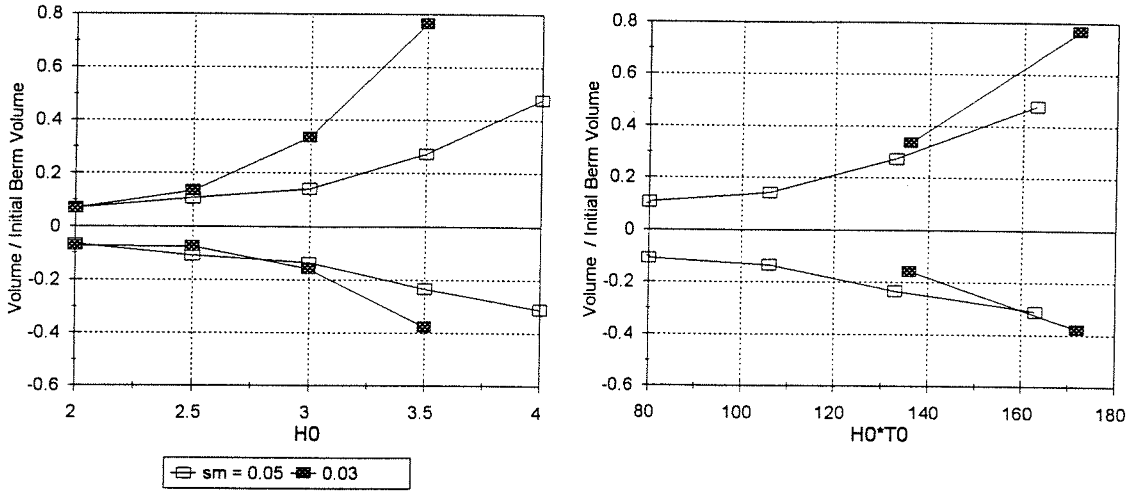


Figure 12. Influence of wave steepness on erosion/deposition.
Wave direction -30°

Transported Volume

Figure 13 presents the transported volume at the roundhead for all wave directions as function of the relative angle after H_0=4.0 (S_m=0.05), showing that the section which experiences the heaviest traffic during the test series is located 110°-130° anti-clockwise from the angle of wave attack.

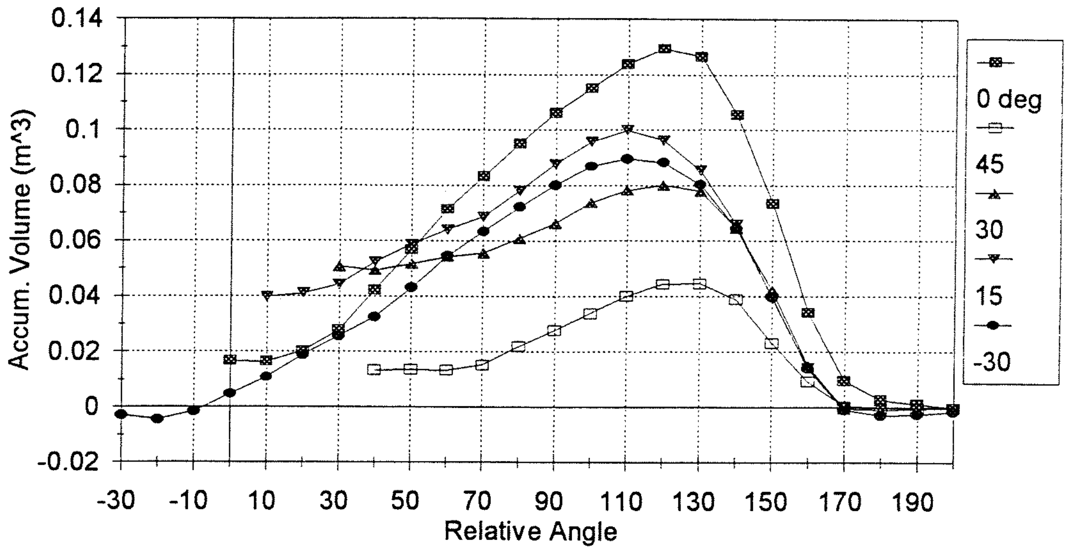


Figure 13. Transported volume on head after H_0=4. All wave directions

Figure 14 shows the maximum transported volume on the roundhead as function of H_0 revealing that the transport accelerates for $H_0>2.5$.

As for the recession on the head, the sensitivity to the wave direction can clearly be seen on the transport. The results indicate that the decrease in transported volume with the wave direction can be estimated by a $\cos^3$ distribution for the higher H_0's comparing with the transport for wave direction 0^o.

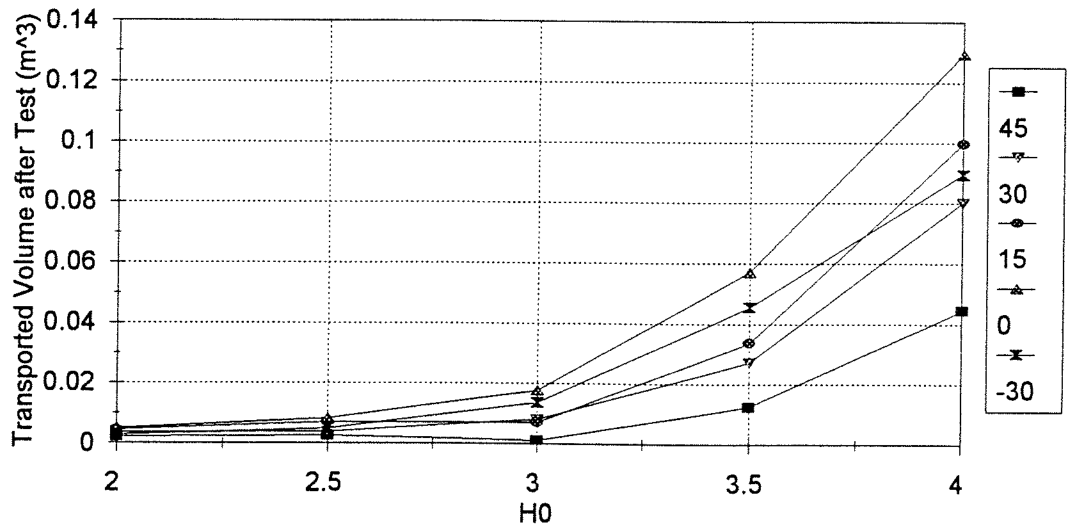


Figure 14. Maximum transported volume on head

Stone Transport

In Figure 15, the stone transport for H_0=4.0 is plotted for all wave directions as function of the relative angle, again showing that the maximum transport takes place at the sections 110°-130° relative to the wave direction. The maximum stone transport on the head occurs with head on waves (0°), being up to three to four times higher than for wave direction 45°.

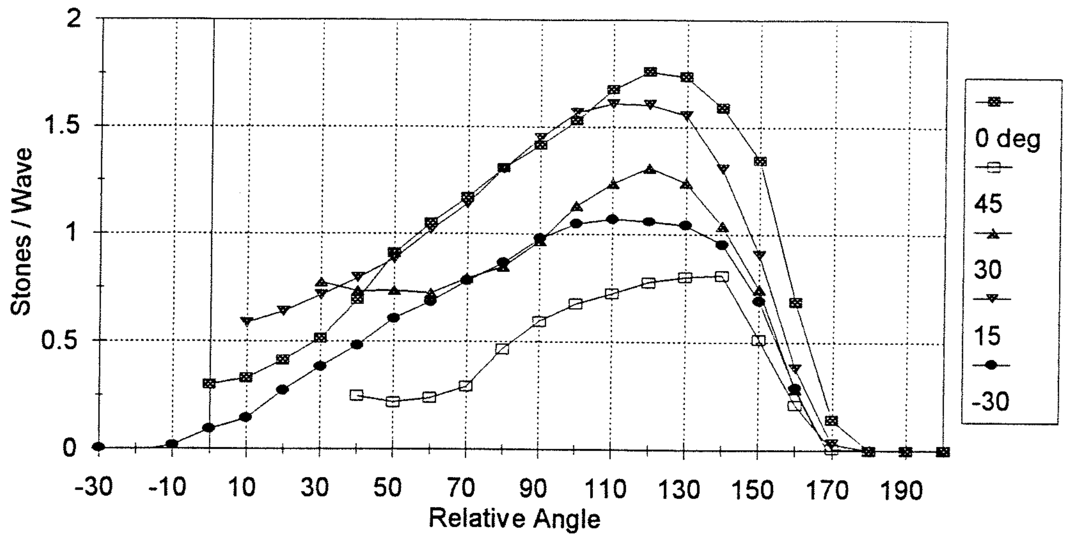


Figure 15. Stone transport for H_0=4.0. All wave directions

Conclusion

A total of six test series were carried out in a wave basin for studying the development of berm breakwater roundheads. Five wave incidence angles were tested together with two wave steepnesses All tests were carried out with irregular long-crested waves, and each test series consisted of five tests for reshaping of the breakwater (H_0=2.0, 2.5, 3.0, 3.5 and 4.0).

The measurements included a total of 38 profiles along the breakwater which were interpolated into a 3D representation of the breakwater. This formed the basis for detailed calculations of the profile development, recession of the berm, eroded and deposited volumes and transport of stones in arbitrary sections of the breakwater.

The main findings of the work are summarised below :

- The maximum recession occurs at the area directly exposed to the waves.
- The recession/erosion pattern follows the wave direction.
- On the head, the maximum recession was observed for head-on waves (0°) being up to two times higher than for 45°. For the trunk section, the increase was less than 1.5. The recession on the head is thus more sensitive to changes in the wave direction than on the trunk.
- For 45° wave direction, the maximum recession on the trunk and on the head is of the same magnitude, whereas the maximum recession on the head is 75 per cent higher than on the trunk for wave direction -30°. For 0°, the maximum recession on the head is about 50 per cent higher than on the trunk.
- The results for the two wave steepnesses, $S_m = 0.03$ and 0.05, showed that the wave steepness is of paramount importance, and that H_0T_0 is a good parameter when comparing the results - better than H_0.
- The reshaping of the head accelerates for H_0>80-100.
- The maximum transport of stones takes place 100°-130° anti-clockwise from the angle of wave attack.
- The maximum stone transport rates were found for wave direction 0°, being up to three to four times higher than for wave direction 45°.

Acknowledgements

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