HYDRAULIC PERFORMANCE
OF
BERM BREAKWATERS

by

Ole Juul Jensen and Torben Sorensen

Abstract

Rubble mound breakwaters have been used for centuries for the protection of harbours. In many cases breakwaters were built in relatively deep water and exposed to waves too severe in relation to the size of rock used for construction. Furthermore, they were often built with a steep slope, and consequently, severe damage occurred. In some cases, breakwaters have been repaired by a continuous supply of stones until an almost stable equilibrium slope developed. In this way, the breakwaters at Cherbourg, Plymouth and Holyhead, Refs. /2/ & /3/ were developed. At certain places in nature the same may be observed for gravel beaches, where the available material by wave and tidal action is reshaped until an almost equilibrium situation occurs. In recent years, the concept of unconventional rubble mound breakwaters, i.e. berm breakwaters, has gained much attention among researchers and engineers as an economical method to build breakwaters at certain sites. At DHI, the principle of berm breakwaters was first used in 1978 for the Skopun Breakwater Extension, Faroe Islands.

Résumé

Les brise-lames en enrochements ont été utilisés depuis des siècles pour la protection des ports. Dans de nombreux cas les brise-lames étaient construits dans des eaux relativement profondes et exposés à des vagues trop fortes par rapport aux dimensions des roches utilisées pour la construction. De plus, ils ont souvent présenté une pente trop raide et ont en conséquence subi de graves dommages. Dans certains cas les brise-lames ont été réparés au moyen d'un apport continu en pierres jusqu'à ce que soit atteinte une pente d'équilibre presque stable. Les brise-lames construits à Cherbourg, Plymouth et à Holyhead (réf. /2/ et /3/) ont ainsi évolué. Dans la nature à certains endroits on peut observer le même phénomène pour les plages de gravier où les matériaux disponibles sont remaniés par l'action des vagues et des marées jusqu'à ce que soit presque atteinte une situation d'équilibre. Au cours des dernières années le concept du brise-lames en enrochement non classique c.-à-d. les brise-lames à risberme, s'est mérité une grande attention de la part des chercheurs et des ingénieurs à titre de méthode peu coûteuse de construction de brise-lames à certains emplacements. À l'Institut danois d'hydraulique le concept du brise-lames à risberme a été utilisé pour la première fois en 1978 lors du prolongement du brise-lames Skopun aux îles Féroé.

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1. INTRODUCTION

1.1 The Berm Breakwater Concept

Rubble mound breakwaters have been used for centuries for the protection of harbours. In many cases breakwaters were built in relatively deep water and exposed to waves too severe in relation to the size of rock used for construction. Furthermore, they were often built with a steep slope, and consequently, severe damage occurred. In some cases, breakwaters have been repaired by a continuous supply of stones until an almost stable equilibrium slope developed. In this way, the breakwaters at Cherbourg, Plymouth and Holyhead, Refs. /2/ & /3/ were developed. At certain places in nature the same may be observed for gravel beaches, where the available material by wave and tidal action is reshaped until an almost equilibrium situation occurs. In recent years, the concept of unconventional rubble mound breakwaters, i.e. berm breakwaters, has gained much attention among researchers and engineers as an economical method to built breakwaters at certain sites. At DHI, the principle of berm breakwaters was first used in 1978 for the Skopun Breakwater Extension, Faroe Islands.

1.2 Basic Principles

The basic principles of a berm breakwater may be presented as shown in Fig. 1.

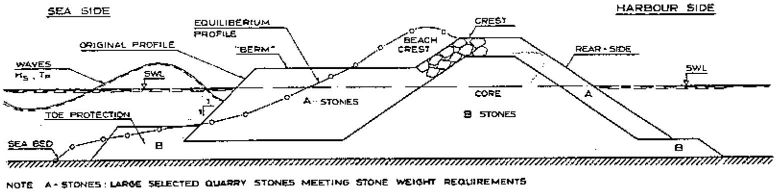

Fig. 1. Principles of a Berm Breakwater.

1. As for a conventional rubble mound breakwater, a berm breakwater requires suitable and proper toe protection if the breakwater is founded on sand in order to avoid excessive scour and sinking of the large stones into the seabed.

2. The main difference from a traditional rubble mound breakwater is with respect to the principal seaward protection. Berm breakwaters use smaller stones relative to the design wave height than traditional rubble mound breakwaters, and consequently the stones would if placed traditionally in two layers have to be placed on a very flat slope to make them stable. This would require heavy equipment, i.e. cranes with long reach and high moment capacity.

   Instead of placing the stones in two layers on flat slope for construction, they are placed in a heap on the seaward face. This requires less heavy equipment. Later the stones are reshaped by wave action until an equilibrium slope is developed. The main objective of the studies of berm breakwaters is thus to determine the necessary size and extent of the heap of stones to make sure that there is enough material in the heap to form the equilibrium slope.

3. The crest and rear side of a berm breakwater behave basically in the same way as on a traditional rubble mound breakwater. Thus, as a minimum requirement, the crest level should be high enough to prevent serious damage due to overtopping during design conditions (waves and water level). Other breakwaters may require a higher crest in order to further reduce overtopping to an acceptable level. This is especially the case if there is a reclamation behind the structure or a harbour basin sensitive to overtopping water.

4. Most berm breakwaters use fewer gradations of quarry stones than traditional rubble mound structues. It is possible (see example in Figs. 5 & 6) to limit the number of gradations to two. In this case, the small stones are used as core material and bed/toe protection, and the larger stones for the berm and armour layer on the crest and rear side.

   It is important, however, to emphasize that for this simple solution, it is of utmost importance that the large stones in the coarse gradation are carefully sorted in the quarry to make sure that this fraction contains no small stones or fines. This is important as the porosity of the large stone material ensures that the breakwater has a high wave energy absorbtion capacity.

5. The breakwater head on berm breakwaters constitute a special problem as the mode of transport of stones on the round-head is different from that on the trunk. This will be further discussed in Section 3. Normally, it will not be feasible to make the round-head on a berm breakwater with a berm profile. It is more advisable to apply either a traditional rubble mound type round-head with larger armour stones or concrete units or instead to use a type of solid breakwater head, i.e. caissons or similar.

2. THE SEAWARD PROFILE UNDER WAVE ATTACK

2.1 The Parameters of Importance

The seaward profile of a berm breakwater requires special attention. The reshaping process develops according to complex "rules" which depend on the following parameters:

1. The stones present on the slope, i.e. the average stone weight, $\bar{w}_s$ or the nominal diameter, D_{n50}, and the density of the stones, ρ_s. The relative stone density (stone density relative to water density) can also be expressed as $\Delta=(\rho_s/\rho_w-1)$. According the Ref. /1/, the gradation of the stones plays a relatively minor role for the stability. Ref. /1/ defines $D_{n50} = (w_{50}/\rho_s)^{1/3}$ as the parameter for the stone sizes. Quarry stones are, however, not cubes, and DHI normally uses $\bar{w} = 0.75 \cdot \rho_s \cdot d^3$. According to Ref. /1/, the reshaping is not very dependent upon the gradation within reasonable variations, if the material is large quarry stones.

2. The amount of material available for the reshaping process, but according to Ref. /1/ not so much the original seaward profile of the material.

3. The wave conditions:

   Wave Height, H_s

   The reshaping does not seem to be very dependent upon the spectral shape (Ref. /1/).

   Wave Period, T_p or T_z

   The wave period is as important as the wave height according to Ref. /1/. Surprisingly, it is reported that the mean period T_z is a better parameter for the wave period influence than the spectral "peak" wave period, T_p.

   The Storm Duration

   The storm duration, i.e. the number of waves during the storm is important for the profile development. After a certain number of waves, the profile will approach its equilibrium.

4. The water level. Different water levels due to storm surge or tides, are of importance for the reshaping process. The influence of the water level may be twofold:

   i. If the breakwater is located in shallow water with the maximum waves limited due to wave breaking, the highest and most severe waves will occur at high water level.

   ii. If the tidal range, TR, is considerable compared to the wave height, say TR ≥ 1/3 H_s, the tide is important as it constantly shifts the water line and thereby the zone of attack of the waves. For otherwise identical wave input, the equilibrium profile will be different for a situation with a tide compared to a situation with constant water level.

2.2 The Reshaping Process

Before describing the reshaping process, it is relevant to outline some important mechanisms about the run-up process on a rubble mound. The forces on the stones or armour units on such a slope consist of the following: 1) hydrodynamic forces (drag, lift and acceleration forces) due to the flow of water, 2) the force of gravity on the unit, 3) reaction forces from neighbouring units. A stone or unit on a slope will be displaced by the hydrodynamic forces if they are exceeding the forces trying to keep the units in place (gravitation and reaction forces). On a relatively steep traditional rubble mound breakwater with slope in the range 1:1.33 to $\sim$ 1:3.0, the armour units will always tend to move downwards on the slope if displaced from their original position. Recent model studies at DHI with measurements of the forces on armour units (Ref. /4/) have shown that the forces during run-up are generally larger than during run-down, but stones are more easily moved downwards on the slope due to gravity.

For flat slopes, 1:4.0 or flatter, the balance between the run-up/ run-down process and gravity effects changes, and the general tendency for a rubble or rip-rap slope is mostly an upwards transport of material resulting in the formation of a "beach crest", see Fig. 1.

With this knowledge, it is more clear what happens when a berm breakwater is exposed to wave action.

The stones on the seaward face can initially be placed in a heap with a large horizontal berm ending seawards in a steep slope with an angle equal to or close to the angle of repose (see Fig. 1). During wave action some of the stones are moved downwards and some upwards forming a more gentle and relatively flat almost S-shaped slope around and somewhat below SWL. This process continues, if there is enough material available for the reshaping process, until a slope is attained that is in equilibrium with the incoming wave conditions. The equilibrium slope may thus be defined as the slope where there is on average an equilibrium between the forces on the stones in the upward and downward direction.

2.3 Profile Development

The ultimate result of the wave reshaping process is thus a profile nearly in equilibrium with the incoming waves. However, for any given sea state, it will require a certain storm duration, i.e. a certain number of waves to accomplish the reshaping process of the seaward stones.

In order to make a preliminary design of a berm breakwater, it is important to have an idea about the profile development and of the equilibrium slope for any given stone size and incident wave conditions. Presently, no formula or theoretical method is available for the prediction of the equilibrium slope and the shape of the stone slope. Ref. /1/ shows an analysis of a long series of model tests of the equilibrium profile which develops on an initially infinite and straight slope of stones when exposed to wave action. Based on empirical analysis of the model tests a model was developed for prediction of the equilibrium slope for given stone material and wave conditions. However, these tests were for a straight infinite slope of stones and relatively small stone sizes relative to the incoming waves, while many practical breakwater projects deal with breaking waves and allow wave overtopping. The examples dealt with in Ref. /1/ are for H_s/D_{n50} in the range 7.4 to 21.5, i.e. very small stones relative to the wave height. The DHI-examples shown later are all for H_s/D_{n50} in the range 4.1 to 4.8. Further, the results in Ref. /1/ are for a slope of homogeneous stone material, while real berm breakwater projects normally have a minimum of two gradations of stones. It is interesting to notice that a traditional rubble mound breakwater typically has $H_s/D_{n50} \cong 3.0$ (see example below).

Example (Traditional Rubble Mound Parameter)

$K_D = 3.0$, $H_s = 4.0\text{m}$, $T_z = 8\text{s}$, $\gamma_s = 2.7\ \text{t/m}^3$, $\gamma_w = 1.03\ \text{t/m}^2$, $\cot\alpha = 2.0$

$$\bar{w} = \frac{\gamma_s \cdot H_s^3}{K_D \cot\alpha \left(\frac{\gamma_s}{\gamma_w} - 1\right)^3} = 6.75\ \text{t},\quad D_{n50} = \left(\frac{w}{\gamma_s}\right)^{1/3} = \left(\frac{6.75}{2.7}\right)^{1/3} = 1.36\ \text{m}$$

$$\frac{H_s}{D_{n50}} = \frac{4.0}{1.36} = 2.94 \sim \underline{3.0}$$

Ref./1/ uses a parameter

$$H_oT_o = \frac{H_s}{\Delta D_{n50}} \cdot \left(\frac{g}{D_{n50}}\right)^{\frac{1}{2}} \cdot T_z$$

being a parameter for the influence of both the wave height and the wave period. The parameter is a measure of the wave height relative to the stone size multiplied with the wave length relative to the stone sizes.

Note, in this example, $H_oT_o = \frac{4.0}{1.62 \cdot 1.36} \left(\frac{9.81}{1.36}\right)^{\frac{1}{2}} \cdot 8 = 39$

In order to compare the profile development, the results of model tests for four different projects are shown in Fig. 2. More details on each project are given in Figs. 4-7. The profiles shown are the ultimate profiles after completion of several test sequences as shown on the figures. Only the wave conditions for the last test sequence are reported here.

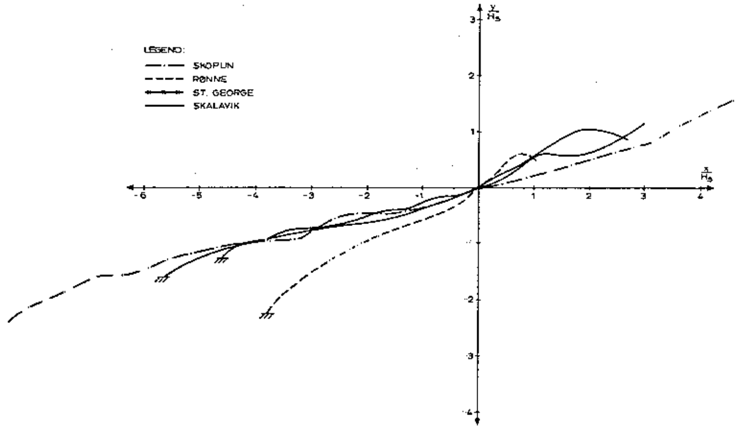

Fig. 2 Comparison of "equilibrium" profiles for four projects.

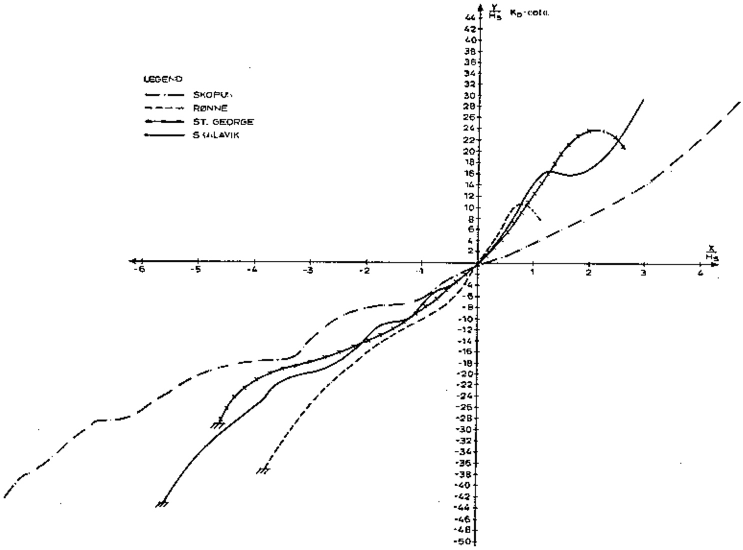

Fig. 3 Comparison of equilibrium profiles after conversion of vertical axis by multiplication with $K_D \cdot \cot\alpha$.

The details for each profile are given in Figs. 4-7 and in Table 1.

Table 1a Data for four berm breakwater projects.

Model Test Year	Project	Depth* h (m)	Stone Charact. W_{50} (t)	W (t)	W (t)	W_{85}/W_{15}	D_{50} (m)	Wave Charact. H_s (m)	T_p (s)	Duration (h)	Steepness $H_s/1.56T_p^2$	Stone/Waves H_s/D_{50}	$H_s/\Delta D_{n50}$	H_oT_o
1978	Skopun Faroe Isl	21 0		12 5	10-15		1 65	7 0	18	3	0.014	4 2	2 56	78
1980	Rønne Denmark	10 0	3.5		2-8		1 09	4 5	10 2	10	0 028	4 1	2 44	52
1983	St. George Alaska, USA	8 2	6.0		1-12	2 45	1 34	6 4	18	5	0 013	4 8	3 35	114
1986	Skalavik Faroe Isl	15 0	8.2		3-15	2.15	1 45	7 0	14	2 5	0 023	4 8	2 86	73

Table 1b Results for four berm breakwater projects.

Model Test Year	Project	Equilibrium Slope Characteristics Slope SWL	Minim. Slope	Depth, DM min. slope (m)	DM/H_s	Crest Height, CH (m)	CH/H_s	Stability Coefficients K_D cotα	K_D, SWL-slope	K_D, min. slope
1978	Skopun Faroe Isl	1 3.1	1 4 5	~ 4 0	0 57	No crest	-	18 0	5.8	4 0
1980	Rønne Denmark	1 1.25	1.3 3	~ 3.0	0 67	2 8	0.62	17 0	13 6	5.1
1983	St. George Alaska, USA	1 2 4	1 6.25	~ 4 4	0 69	6 8	1 06	22 8	9 5	5 1
1986	Skalavik Faroe Isl.	1 2 9	1 5.4	~ 4.5	0 64	No crest	-	27 3	9.4	5.1

* Note: The depth is the actual water depth.

Comparison of the profiles in Figs. 4-7 and the data in Table 1 reveal the following:

i. The $H_s/\Delta D_{n50}$ values did not vary significantly from project to project being in the range of 2.4-3.4. By comparison with Ref. /1/, it is seen that the stones used for these four projects are relatively larger than used for the model tests.

Ref. /1/ uses the following classification:

- statically stable breakwater $H_s/\Delta D_{n50}$ = 1-4
- berm breakwaters and S-shapped profiles $H_s/\Delta D_{n50}$ = 3-6
- dynamically stable rock slopes $H_s/\Delta D_{n50}$ = 6-20
- gravel beaches $H_s/\Delta D_{n50}$ = 15-500
- sand beaches $H_s/\Delta D_{n50}$ > 300

It appears that the four projects have values close to the lower limit reported for berm breakwaters.

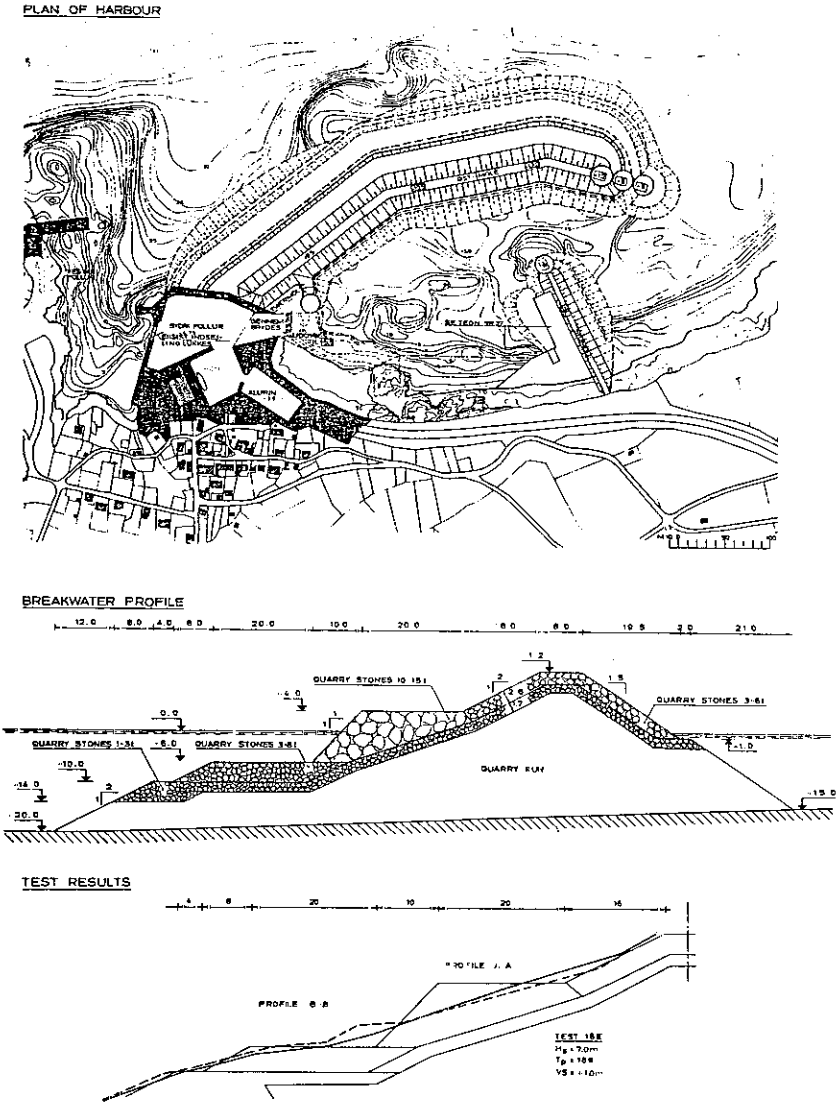

Fig. 4 Results, Skopun.

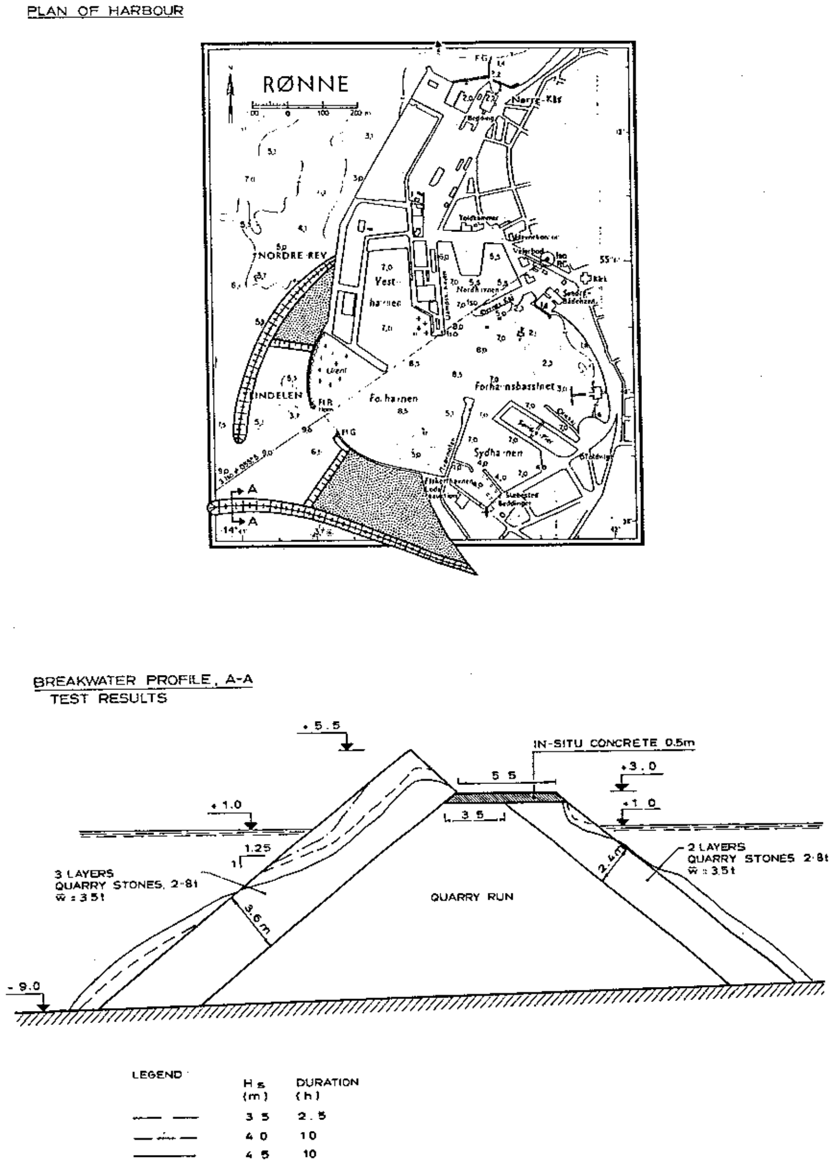

Fig. 5 Results, Rønne.

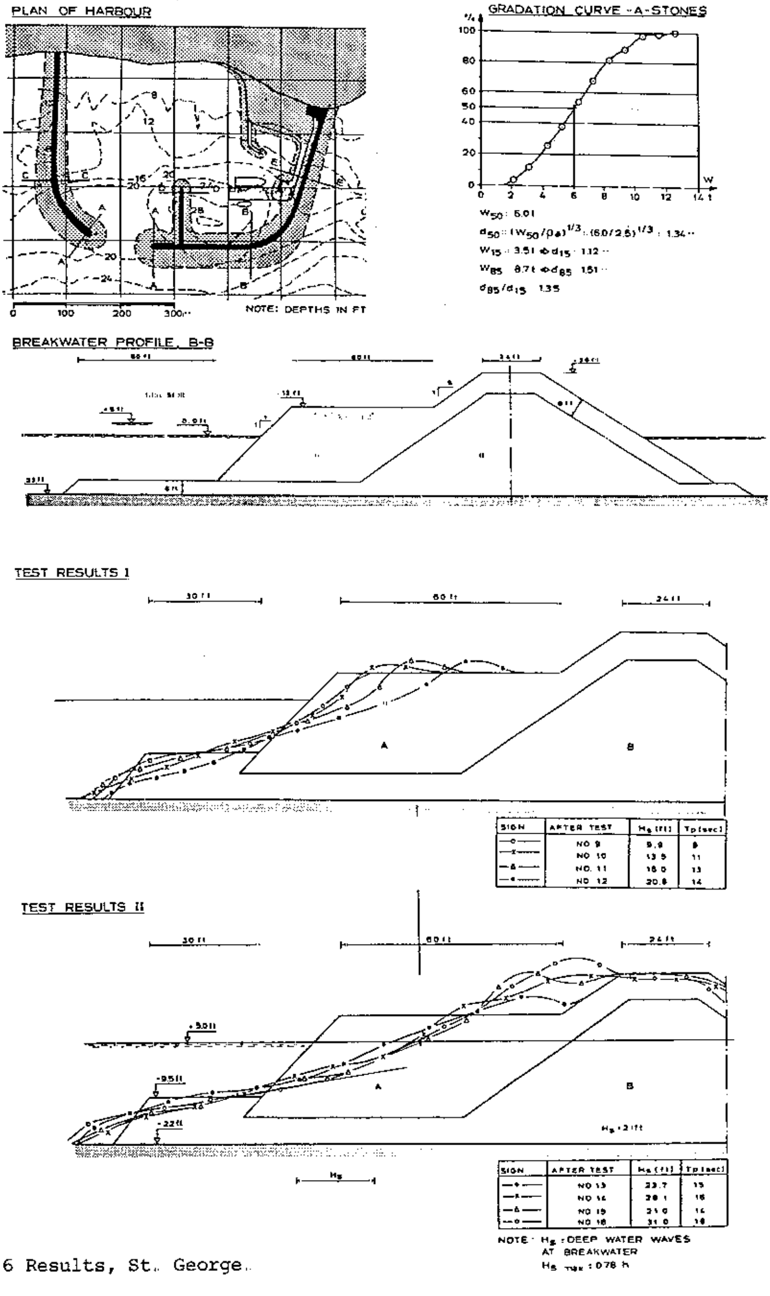

Fig. 6 Results, St. George.

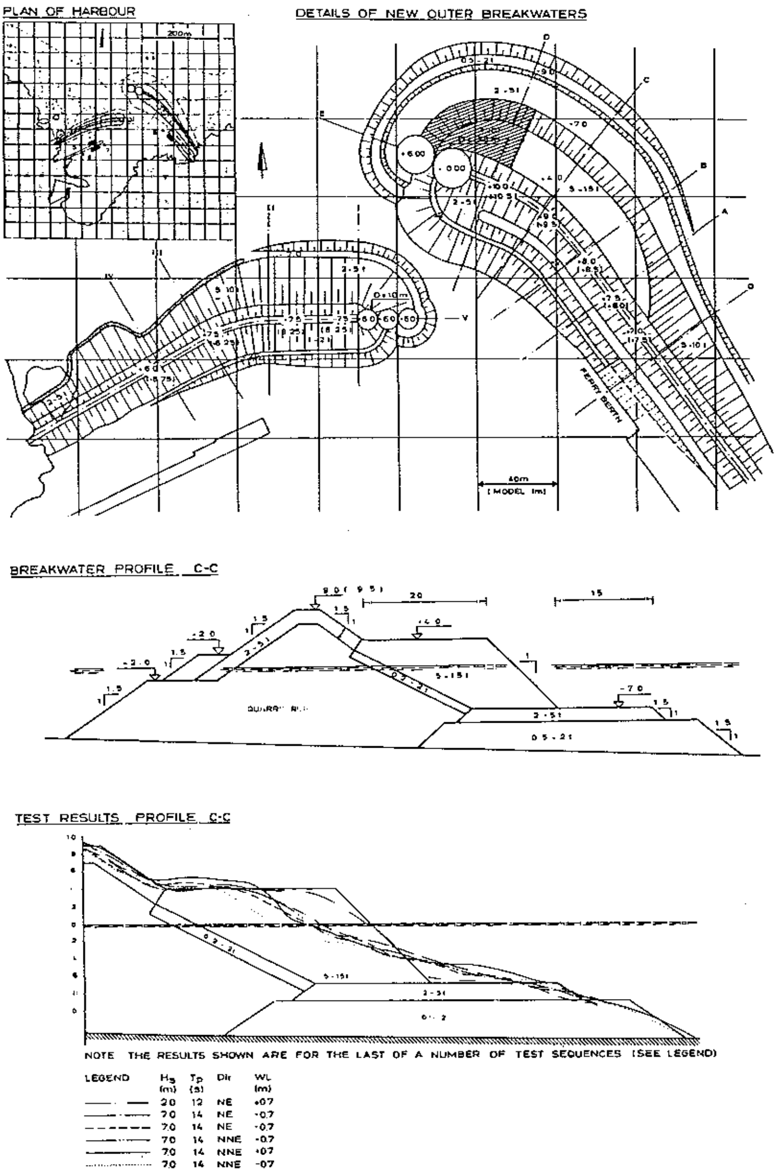

Fig. 7 Results, Skålavik.

ii. An important aspect often discussed for berm breakwaters is the possibility of "long-shore" transport of the stones if the breakwater is exposed to oblique wave attack. Not much information is available in the literature about the lower limit for "long-shore" transport of stones in slopes. Ref. /5/ presents the results of a study dealing with this aspect.

This study deals with more gentle slopes than most berm breakwaters and can consequently only be used as a first rough guideline.

Based on model testing and graphical presentation, the following empirical formula was found as the best fit to the data. S(x) is the material transport. (Note: $H_s = 0.10\text{-}0.21$ m and $D_{90} \sim 5.8 \cdot 10^{-3}$ m).

$$\frac{S(x)}{g\, D_{90}^2\, T_s} = 7.12 \cdot 10^{-4}\, \frac{H_{sd}\, \cos^{\frac{1}{2}} \phi_v}{D_{90}} \left\{ \frac{H_{sd}\, \cos^{\frac{1}{2}} \phi_v}{D_{90}} - 8.3 \right\} \frac{\sin \phi_v}{\tan h(k_s h)_v}$$

h is the depth and $k_s = 2\pi L_s^{-1}$ where L_s is the 15% excess value of the wave length.

It is important to notice that S(x) = 0 for

$$\frac{H_{sd}\, \cos^{\frac{1}{2}} \phi_v}{D_{90}} - 8.3 = 0$$

Note: According to Ref. /1/,

$H_{sd} = H_s$, transformed via the linear theory to deep water,

ϕ_v = angle of wave approach on the foreshore.

Example:

$$\frac{D_{90}}{D_{50}} = 1.2 \text{ (corresponding to the example in Fig. 6).}$$

Note: $\cos^{\frac{1}{2}} \phi_v$ is maximum for $\phi_v = 0$.

Thus

$$\frac{H_{sd}}{1.2\, D_{50}} = \frac{8.3}{\cos^{\frac{1}{2}} \phi_v} \Rightarrow \underline{\frac{H_s}{D_{50}} = 8.3 \cdot 1.2 \cong 10}$$

Although this is a very rough estimate of the lower limit of initiation of "long-shore" transport of material, it appears from the data for the projects, that "long-shore" transport should be insignificant for them all.

It is very important for the design of future berm breakwaters that more basic research is conducted to determine accurately the lower limit for "long-shore" transport as function of stone size, profile slope and wave conditions, etc.

iii. The four projects have H_oT_o values (see Table 1) in the range 52-114. Ref. /1/ states that "for H_oT_o values smaller than 100, the slopes are stable as required in the traditional breakwater design. The formula for H_oT_o does not consider the slope of the structure. For the example given in section 2.3, H_oT_o = 100 appears high.

iv. The four profiles in Fig. 2 show some points of resemblance, especially with respect to the slope from slightly above the SWL to a distance of about H_s below SWL.

The approximate slope around SWL has been determined as shown in Table 1. It appears that the slope ranges from 1:1.25 to 1:3.1. Although Hudsons formula does not normally apply with the same stability coefficient, K_D, independent of the slope, it is interesting to notice that by using the slope found close to SWL, the corresponding K_D-factors are in the range 5.8 to 13.6, i.e. typically an average value of $K_D \cong 10$.

Somewhat below SWL, the minimum slope occurs as the material in the slope is exposed to the largest hydrodynamic forces in this region.

By the same calculation, a minimum K_D value of K_D = 4.0-5.1 is found for this region of the slope. It is interesting to notice that this value is only about 50% larger than would normally be used on a traditional rubble mound breakwater allowing for some displacements of stones during design conditions ($K_D \sim 3.0$), (see the above example). It is of further relevance that the minimum slope occurs at a distance of about 0.65 H_s below SWL. These observations are important as they can be used in a first preliminary assessment of similar berm breakwater structures.

It is important to notice that the four projects all have stones so large relative to the wave height, H_s, that the initial slope has an influence on the equilibrium slope after reshaping by waves.

Ref. /1/ states that "for $H_s/\Delta D_{n50}$ < 10-15, the initial slope has an influence on the equilibrium slope".

According to DHI's experience from other studies not described here, it seems that this limit is slightly on the high side, as model tests have shown that for $H_s/\Delta D_{n50} \gtrsim 6$-7, the waves are able to completely redistribute all stones on the profile.

Data from further projects and more basic research are necessary to more accurately define limits for acceptable values of the parameters of a berm breakwater to define if it has acceptable stability in the direction perpendicular to the slope. As for the "longshore" transport previously discussed, it is of utmost importance that the transport of stones up and down in the slope is at an acceptable level, once the reshaping process is completed. If "too much" movement of stones occur, there is a risk that an excessive wear of the stones will take place. Methods to quantify or estimate the wear of stones would therefore be an improvement of the technology for design of berm breakwaters.

v. As the wave action can most easily move the material on the slope from somewhat above SWL to about H_s below SWL, it is in this region that most of the stone displacement takes place and here the profiles for the four projects are most identical after reshaping. Consequently, Fig. 2 shows the largest differences in the slopes above SWL, i.e. the resultant equilibrium slope is dependent on the initial slope for the range of wave conditions and stone sizes investigated.

vi. In order to verify whether it would be possible to obtain a better comparison and in the light of the fact that the apparent stability factors after reshaping were of the same order of magnitude, an attempt has been made to draw up the profiles by multiplying the vertical axis with $K_D \cdot \cot \alpha = (\rho_s \cdot H_s^3) / (\bar{w} (\rho_s/\rho_w - 1)^3)$. The result is seen in Fig. 3, and it appears that it is mainly the Skopun profile that is different from the others. Note in Fig. 4 that the Skopun profile originally had a very wide berm (20 m) in level -6.0 m, and a flat 1:2 slope above level +4.0 m. It is assumed that these features have contributed substantially to the configuration of the equilibrium profile.

vii. The crest height of a berm breakwater can be slightly reduced compared to a traditional rubble mound breakwater because the run-up is reduced due to the flatter slope. Table 2 shows data for the four profiles in question.

It is clear from the comparison that the necessary crest elevation on a breakwater (relative to the design wave height) depends on a number of parameters, of which the slope and the wave steepness are the most important. The stone size on the rear side and the width of the crest and the slopes on both sides of the crest are important as well. It is further of relevance whether damage is permissible on the rear side of the structure. For example, in the case of Rønne where the crest is very low, the model tests have shown that damage occurred on the rear side, but that this damage did not tend to spread further. The client for this breakwater has accepted this and has easy access to stones and is already practicing regular maintenance and repair work on a similar structure.

Table 2 Crest Elevation Comparison

Project	Crest Elevation (m)	WL (m)	H_s (m)	T_p (s)	Steepness	Freeboard, Δh (m)	$\frac{\Delta h}{H_s}$
Skopun	+11.0	+0.70	7.0	18	0.014	10.3	1.47
Rønne	+ 3.6*	+1.0	4.5	10.2	0.028	2.6	0.58**
St. George	+7.95	+1.55	6.4	18	0.013	6.4	1.00
Skalavik	+9.50	+0.70	7.0	14	0.023	8.8	1.26

* After reshaping (see Fig. 5)

** Crest protected by concrete slab. Damage is permissible on the rear side.

3. ROUNDHEADS ON BERM BREAKWATERS

It is well known from many model studies and from practical experience that the roundhead on a rubble mound breakwater requires heavier protection than the trunk. The reason for this is simple, because the velocities in the wave rushing forward and upward are almost the same on the slope of the trunk and on the cone-shaped roundhead. On the trunk, the uprushing water works against gravity and consequently the stones are not easily moved during up-rush. (Note as previously explained for normal rubble mound breakwaters with slope 1:1.5, 1:2.0 or 1:3.0 that the subsequent down-rush is the critical phase of the wave motion on the slope.) Due to the energy loss during up-rush and in the beginning of down-rush the maximum velocity during down-rush will tend to be smaller than the maximum up-rush velocity. On the roundhead the water washes horizontally over the cone-shaped roundhead and the units protecting the roundhead are more easily moved in the tangential direction where gravity has much less stabilizing effect than on the trunk.

Model tests at DHI (Ref. /6/) have proved that for rubble mound breakwaters, it is normally necessary to increase the weight of the stones with a factor in the range 1.5 to 2.0 relative to the trunk. The factor depends primarily on the size of stones relative to the radius of curvature of the roundhead at the point of wave attack. DHI's experience further shows that for more complicated concrete units such as tetrapodes and dolos, a roundhead requires a larger increase in block weight (assuming the density is maintained) to obtain almost the same stability. Model tests have shown factors of weight increase of 2.3 for tetrapods and up to about 4.0 for dolos. Dolos seem to loose their interlocking effect when placed horizontally. This explains the severe reduction in stability of such units on roundheads. It is interesting to notice that the above observations are in line with the observations in Ref. /7/. This publication presents observations of stone and dolos stability for horizontally placed units exposed to oscillatory flow parallel to the surface. It is concluded that the stability of dolos and stones of the same weight were almost identical. (Note the density of stones is larger than the density of the concrete used for the dolos).

For the above reasons, it is DHI's opinion that for permanent roundheads berm breakwaters will normally need special roundhead protection as the berm profile used for the trunk will not be stable. Note that the fortunate situation on the trunk where the profile develops until an equilibrium situation is reached does not occur on a roundhead. If displacements occur on a roundhead, the stones will be moved backwards along the tangent of the wave direction towards the harbour or inner side of the breakwater. Here the stones are lost and have almost no stabilizing effect contrary to the situation of the trunk. On a roundhead near a harbour entrance it will normally be a requirement that the breakwater configuration is well defined and does not change with time due to depth and navigation reasons. If not very much larger stones than used on the trunk can be made available, it will be necessary to introduce other means to obtain a breakwater head with sufficient stability. In cases where a traditional rubble-mound or berm breakwater is built over more than one season, a roundhead of the berm type can sometimes be used as provisional protection in a season with rough wave conditions where the work is stopped. Whether this is acceptable, depends on the probability of serere damage and on the risk the contractor or the owner is willing to take and whether other possibilities for provisional head protection exist on the actual site. The head solutions for the four projects described in this paper are discussed in the following.

Skopun (See Fig. 4)

For the Skopun project the head consists of three cylindrical timber structures filled with stones. The sizes, foundation, levels and crest elevations appear in Fig. 4. It is important to notice that the project involved the placing of the largest stones available near the timber structures in an attempt to reduce the risk of stones damaging the timber. It is of further importance that the timber structures consist of two rings of timber and that the inner ring is connected with steel members to the outer ring. This arrangement is introduced to make sure that the outer ring does not collapse should some of the timber beams or the steel bands of the outer ring be demolished.

Rønne (See Fig. 5)

In Rønne, good quality granite stones of very large weight (up to 20 to 25 t) are easily available. Consequently the roundhead was originally planned with a traditional rubble mound roundhead. However, the breakwater which is presently under construction will for navigational reasons have a head consisting of concrete caissons.

St. George (See Fig. 6)

For the St. George project a solution for the head was developed using 33 t grooved Antifer cubic concrete units placed in the traditional manner in two layers. The slope of the armour layer was 1:2.5 on the roundhead. Wave basin model tests were performed to study the stability of the roundhead and of the transition from the berm type profile on the trunk to the conically shaped roundhead.

Skalavik (See Fig. 7)
For Skalavik the same solution as for Skopun was used. The stones in the section near the head are specified as larger than 20 t and it has been recommended that the timber beams in the zone most exposed to collision by stones should be further protected by steel pipes surrounding the timber. It is recommended as well that the Contractor makes an effort during construction to place large stones around the toe of the timber structure in such a way that they will reduce the risk of collision of stones during the reshaping process of the berm on the seaward face of the structure.

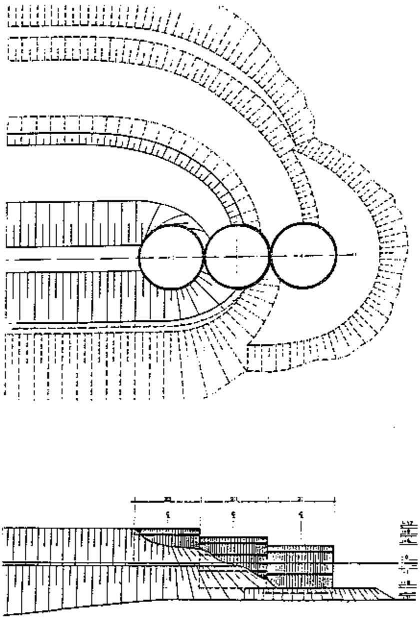

Fig. 8 Head Solution for the Skopun Project.

4. SUMMARY AND CONCLUSIONS

The paper has presented the results of four individual berm breakwater projects and used these practical examples for a more general discussion of the features of berm breakwaters and of appropriate parameters for the description of their behaviour. It is interesting to note that although the four projects discussed are very different, the comparison of the hydraulic features show great ressemblances. Berm breakwaters have attracted the attention of engineers and researchers for almost 10 years, however, the technology for the design of berm breakwaters still requires improvement, and more research is consequently encouraged. The following points are worth noting:

i. The nature of and the parameters determining the lower limit for "long-shore" transport of material on a berm breakwater is of importance and should be investigated.

ii. The possible movement upwards and downwards of stones in the equilibirum slope of a berm breakwater is of significance. More research is required to quantify this aspect and to define acceptable limits and parameters for description thereof. The associated wearing-process is an item for further research as well.

iii. The head on berm breakwaters requires more research to identify for which situations berm-type solutions may also be used for the head, with special regard to the stone size requirements.

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