INTERNATIONAL NAVIGATION ASSOCIATION

STATE-OF-THE-ART OF DESIGNING AND CONSTRUCTING BERM BREAKWATERS

Report of Working Group 40
of the
MARITIME NAVIGATION COMMISSION

INTERNATIONAL NAVIGATION
ASSOCIATION

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This Report has been produced by an international Working Group convened by the Maritime Navigation Commission (MarCom). Members of the Working Group represent several countries and are acknowledged experts in their profession.

The objective of this report is to provide information and recommendations on good practice. Conformity is not obligatory and engineering judgement should be used in its application, especially in special circumstances. This report should be seen as an expert guidance and state of the art on this particular subject. PIANC disclaims all responsibility in case this report should be presented as an official standard.

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CONTENT

FOREWORD

Manuals and guidelines for different types of breakwaters and coastal structures have been issued before this time, e.g. the Shore Protection Manual (1984), CIRIA/CUR (1991), CUR (1995), and PIANC (1992). The types of breakwaters treated in these manuals and reports have mainly been conventional rubble mound breakwaters and caisson type breakwaters.

The berm breakwater concept is fairly old, but was not used very much until the 1980's when it was "reinvented" to provide wave protection for an airport runway extension into the sea in Dutch Harbor, Alaska in the Aleutian Islands (Rauw, 1987). The concept was also used later for the design of the berm breakwater at Keflavik, Iceland in 1983 (Baird and Woodrow, 1987). Since that time, many berm breakwaters have been built in Iceland and throughout the world. The primary advantage of the berm breakwater is that the armour stones are smaller than in a conventional rubble mound breakwater. Hence, the berm breakwater can be constructed with commonly available heavy construction equipment and from local quarry sites at a cheaper cost.

Along with the experience gained in the construction of the berm breakwaters now built, a substantial amount of research on different aspects of the concept has been carried out. However, the results of this research and practical experience is scattered throughout the literature. PIANC decided, therefore, to form a Working Group under MarCom to formulate guidelines for the design of berm breakwaters.

The following terms of reference were given to the Working Group by PIANC MarCom in 1998:

• "Background

Berm breakwaters have become in many cases an attractive, both technically and economically, rubble mound breakwater in exposed locations. This type of breakwater has up to now been mainly used in Canada, Iceland, Norway and Denmark (Faero Islands). The main advantage of the berm breakwater is that smaller stones can be used on this dynamic stable berm breakwater than on a static stable conventional rubble mound breakwater. Hence, conventional contractors equipment can be used to move and place the cover stones rather than heavy specialty equipment, which has to be used for lifting heavy cover blocks for the static stable rubble mound breakwater.

• To be investigated

Considerable research has been carried out on berm breakwaters through the recent years (e.g. EU-MAST I and MAST II projects, etc.) covering theoretical and experimental work as well as compilation of practical experience. There are however no general guidelines on the design of this type of breakwater taking recent research results and practical experience into account.

The task of the Working Group will be to study the different research results and compile all relevant information into practical guidelines for the design of berm breakwaters."

Although some 60 berm breakwaters have been built throughout the world and considerable research and practical experience have been compiled for berm breakwaters, there is still a need for additional research. The berm breakwater offers great flexibility for the designer. The design should be "supply based" and not necessarily "demand based". Hence, the specifications should be "functional specifications" and not "demand specifications".

The designer will not necessarily find "easy" guidelines in this report or answers to his or her practical questions, e.g. the width of the berm or the crest height of a berm breakwater vs. significant wave height, because there is no "universal" answer to these questions. On the other hand the experienced designer will find helpful information (with some detailed background) on the issues he or she has to address during the design process. For these reasons, then, the title of this report is "State of the Art on the Design and Construction of Berm Breakwaters" and not "Guidelines for the Design and Construction of Berm Breakwaters".

Because of the restrictions on report lengths set by PIANC, emphasis in this report is put on the items specific to the design and construction of berm breakwaters. Details of those issues common to all breakwaters is left out, i.e. environmental conditions, soil stability, etc.

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Meetings of PIANC WG 40

PIANC WG 40 has had the following meetings:

24 June 1998.
Meeting during the International Conference on Coastal Engineering, Copenhagen, Denmark.

7 June 1999.
Meeting during the international conference on "Coastal Structures'99", Santander, Spain

INTRODUCTION

Berm breakwaters are different from ordinary rubble mound breakwaters as indicated in Figure 1.1

Conventional rubble mound breakwater.

Berm rubble mound breakwater

Figure 1.1 Conventional vs. berm rubble mound breakwaters.

A conventional rubble mound breakwater is required to be almost statically stable for the design wave conditions, while the berm breakwater has traditionally been allowed to reshape to a statically stable or a dynamically stable profile as indicated in Figure 1.1, although recently non-reshaping statically stable berm breakwaters have also been considered. Thus, we may divide the berm breakwater into three categories:

- **Statically stable non-reshaped.** In this condition few stones are allowed to move, similar to the condition for a conventional rubble mound breakwater.
- **Statically stable reshaped.** In this condition the profile is allowed to reshape into a profile, which is stable and where the individual stones are also stable.
- **Dynamically stable reshaped.** In this condition, the profile is reshaped into a stable profile, but the individual stones may move up and down the front slope.

The berm breakwater has normally been constructed with a berm that has been allowed to reshape instead of constructing it with the reshaped profile directly. This is so because it has been considered cheaper to construct the breakwater with a reshaping berm. In recent years, there has been a drive to design the berm breakwater in such a way that it will not reshape at all, because the reshaping process may eventually lead to excessive breaking and abrasion of individual stones. However, many of the "old" reshaped berm breakwaters have functioned quite well without excessive breaking and/or abrasion of the stones. Obviously, the question of allowing reshaping or not has to do with stone quality and the stones' ability to withstand impacts leading to breaking and/or abrasion.

In many cases the necessary armour stone weight on conventional rubble mound breakwaters is so large that concrete armour blocks are required. This is illustrated in Table 1.1 which shows the block weight on the Sirevåg berm breakwater, Norway (further description follows) and the recession for the 100-year wave height $H_{s,100}$ = 7.0 m and for the 10.000-year wave height $H_{s,10,000}$ = 9.3 m. The Sirevåg berm breakwater has been designed as a reshaping (slight) berm breakwater. For comparison, Table 1.1 also shows the necessary armour block weight for a conventional two-layer rubble mound breakwater with different slopes and with different degrees of "damage" S. The necessary stone weights for the conventional rubble mound breakwater has been calculated with the formulas of van der Meer (1987) with the assumption for the porosity parameter P = 0.3, wave steepness s_m = 0.04 and the number of waves N = 2000.

Table 1.1 also shows the required armour weight of cubes and tetrapods for a rubble mound breakwater with a slope of 1:1.5. The cube and tetrapod weights have been calculated from van der Meer formulas (van der Meer 1988a) with wave steepness s_m = 0.04, slope of 1:1.5, damage level N_{od} = 0.35 and N = 2000. N_{od} is the number of blocks moved in a strip with a width of D_n. Van der Meer et. al. (2001) state that N_{od} = 0.2 – 0.5 means low, acceptable damage in a lifetime.

Table 1.1 indicates that it is probably not possible to construct a conventional rubble mound breakwater with a reasonable slope from quarried stone for wave conditions similar to those at Sirevåg. If a rubble mound breakwater is still required, then concrete cover units will have to be used. However, the weight of these concrete units has to be larger than the weight of the stones in the berm breakwater. Table 1.1 also shows how "tough" the Sirevåg berm breakwater is, since it can easily withstand the 10.000-year wave event and still be a reshaped statically stable berm breakwater.

It should be added that smaller stones than those used for the Sirevåg berm breakwater may be used (several berm breakwaters have been constructed with relatively smaller stones), and still be considered reshaped statically stable for the 100-year design waves. But the berm breakwater might then become reshape dynamically stable for the 10.000 year waves. In this case, the possibility of stone breakage and abrasion would have to be looked into more carefully.

A berm breakwater generally presents a voluminous permeable berm. It is clear, though, that even a non-reshaped statically stable berm breakwater requires cover stone with less weight than that for conventional rubble mound breakwaters. In cases where not enough large cover stone can be provided for a conventional rubble mound breakwater, a berm breakwater would be an alternative to a conventional rubble mound breakwater with large concrete cover blocks.

Which type of breakwater should be chosen should always be based on cost when all other technical and functional requirements have been satisfied. The present "guidelines" are meant to guide the designer through the design of a berm breakwater. The "guidelines" do not always offer clear guidance, but do provide some background information for the designer to help him make his or her own choice. A chapter on costs has also been included, but since costs depend on locality and when built it is difficult to give any specific guidelines on costs.

Many aspects have to be considered when designing a berm breakwater:

- Environmental conditions
- Preliminary design
- Quarry yield
- Stone breaking strength
- Final design taking into account:
  - Reshaping
  - Lateral transport of stones
  - Wave overtopping
  - Scour and scour protection
  - Soil stability
  - Construction methods
  - Cost evaluation

Table 1.1. Comparison of armour block weights.

Column (1) gives significant wave heights, breakwater slopes and damage levels.
Column (2) gives the required stone weight for Ho = 2.7 (see later)
Column (3) gives the mean recession for the Sirevåg berm breakwater for H_s = 7.0 m and H_s = 9.3 m
Columns (4) –(6) give the required armour unit mass for armour layers of rock, concrete cubes and tetrapods respectively.

(1)	(2) Ho = 2.7 (upper criteria for reshaped static stable) W_{50}, tons	(3) Sirevåg berm breakwater W_{50} = 25 tons Berm width = 19.5 m Recession, m	(4) Conventional two-layer rubble mound breakwater. W_{50}, tons	(5) Rubble mound. Concrete cubes, two layers W, tons	(6) Rubble mound. Tetrapods, two layers. W, tons
H_s = 7.0 m	11	4.3			
Slope 1:1.5				42	30
S = 2			65		
S = 3			55		
Slope 1:2					
S = 2			43		
S = 3			35		
H_s = 9.3 m	26	8.3			
Slope 1:1.5				98	69
S = 2			156		
S = 3			118		
Slope 1:2					
S = 2			112		
S = 3			78		

2. PRACTICAL EXPERIENCE WITH BERM BREAKWATERS

Altogether some 60 berm breakwaters have so far been built throughout the world, according to Sigurdarson et al (2001). Table 2.1 gives an overview of where the berm breakwaters have been built. Generally speaking the berm breakwaters perform very well and no major damage has been reported. A berm breakwater is considered to be a very "tough" rubble mound breakwater, while a conventional rubble mound breakwater is more "brittle".

Table 2.1 List of built berm breakwaters. Sigurdarson et al (2001)

Country	Number of built berm breakwaters	The year the building of the first berm breakwater was completed
Iceland	27	1984
Canada	5	1984
USA	4	1984
Australia	4	1986
Brazil	2	1990
Norway	4	1991
Denmark (Faroe Islands)	1	1992
Iran	8	1996
Portugal (Madeira)	1	1996
China (Hong Kong)	1	1999
Total number	57	

Table 2.2 shows a compiled list with some more details of some major berm breakwaters that have been built throughout the world.

3. ENVIRONMENTAL CONDITIONS

The waves are the most important environmental elements to consider when designing a berm breakwater, although coastal currents and ice may also be important in some areas. Since the consideration of design waves, tsunamis, ice forces, geotechnical conditions etc. is common for most breakwater types, these items are not dealt with in this report due to space limitations. Reference for these items is made to other sources, e.g. CIRIA/CUR (1991), OCDI (2001), CERC (2001).

It is emphasised that every possible item of data and all manner of data analysis should be used to establish design environmental conditions. Wave data, for example, could include measured waves, hindcast waves and visually observed waves. The designer should also consider and evaluate the uncertainties with relation to the environmental conditions.

4. STABILITY AND RESHAPING OF BERM BREAKWATERS

4.1 Governing parameters.

The most used parameters in relation to the stability of berm breakwaters are the following:

- $N_s = Ho = \dfrac{H_s}{\Delta D_{n50}}$, stability number
- $HoTo \; \dfrac{H_s}{\Delta D_{n50}} \sqrt{\dfrac{g}{D_{n50}}} \; T_Z$, period stability number
- $\Delta = \dfrac{\rho_s}{\rho_w} - 1$
- $f_g = \dfrac{D_{n85}}{D_{n15}}$, gradation factor

Table 2.2 List of some major berm breakwaters.

Harbour/ location	Country	Construct. finished	H_s m	T_p s	Angle (°)	Total depth on design WL (m)	Stone class at the edge of the berm on trunk section					Experienced storm (m or % of DW)	Reshaping	
							Rock property			W_{min} - W_{max} (t)	W_{50} (t)		(% of length of trunk)	(% of width of trunk)
							Type	Quality	(t/m^3)					
Akranes	Iceland	1991	3,8	19		11,7	Basalt	Good	2,8	4,0 - 8,0	6,0	80%	0%	0%
Olafsvik	Iceland	1995	4,4	10		10,7	Basalt	Good	2,8	4,0 - 8,0	5,3	70%	0%	0%
Bolungarvik	Iceland	1993	6,3	17		11,5	Basalt	Good	2,8	4,0 -10,0	6,0	100%	5%	10%
Blonduos	Iceland	1994	4,8	12		9,5	Basalt	Good	2,8	1,0 - 6,0	2,7	90%	5%	10%
Skagastrond	Iceland	1992	3,5	15		8,0	Basalt	Good	2,8	5,0 - 8,0	6,0	80%	0%	0%
Dalvik	Iceland	1995	2,5	8		10,3	Basalt	Excellent	2,8	1,5 - 4,0	2,3	70%	0%	0%
Husavik	Iceland	1989	4,0	16	45	9,7	Basalt	Good	2,9	1,0 - 5,0	2,3	90%	0%	0%
Bakkafjordur	Iceland	1983	4,8	12		10,5	Basalt	Poor	2,85	2,0 - 6,0	3,0	100%	100%	100%
Helguvik	Iceland	1986	5,9	10	45	28,0	Basalt	Good	2,85	1,7 - 7,0	3,2	80%	0%	0%
Keflavik	Iceland	1996	3,7	10		19,5	Basalt	Good	2,85	5,0 - 8,0	6,0	70%	0%	0%
Årviksand	Norway	1991	6.4[1]	15[1]	~ 0	12	Granoblastic gneiss	Poor, fissures	2.8	2.5 - 10[2]	4.4[2]	Unknown[3]	0	0
Mortavika	Norway	1992	6.5[4]	15.6[4]	0 - 45	22	Tonalitic gneiss	Good	2.8	W_{min} = 5.5[5]	8.0[5]	4.6 - 5.8[6]	0%	0%
										W_{min} = 3.3	5.5		~100 m~ 30%	~13 m ~ 85%[7]
Sirevåg	Norway	[8]	7.0	14.0	0	17		Excellent	2.7	20 - 30				
St.Georg	USA	1987	6.0-6.4	16.9	0	8.5	Basalt	Good	2.7	1.7 – 10.,	4.0	100%	100%	100%
Sergripe	Brazil	1994	4.0	10		12.7	Gneiss/grani	Good	2.8	1.0-4.0	2.5	~70%		

1 At 20 m water depth where the model test wave data were referred to - design waves at the location of the breakwater is not exactly known.
2 Used during model tests.
3 The model tests showed that the reshaping started for H_s~4.0 m in 20 m water depth. Since no reshaping has taken place it is inferred that H_s has not exceeded approximately 4.0 m
4 100 year design wave estimated to H_s = 5.7 m and T_p = 13.7 - raised to H_s = 6.5 m and T_p = 15.6 for safety reasons
5 Differs along the trunk section. At the reshaped section, approximately halfway along the breakwater, W_{50} = 5.5 tons
6 The wave conditions during the major storm that reshaped the breakwater is estimated to have been somewhere in this range.
7 Approximately 10,000 m^3 of rock filled into the reshaped area.
8 April 2001: Construction almost completed.

$$\bullet\ N_s^* = \frac{(H_s^2 L_o)^{1/3}}{\Delta D_{n50}} \quad ,$$

$$\bullet\ N_s^{**} = \frac{H_k}{C_k \Delta D_{n50}} \left(\frac{S_{mo}}{S_{mk}}\right)^{-(1/5)} (\cos\beta_0)^{2/5} \approx \frac{0.89 H_{bk}}{C_k \Delta D_{n50}} \quad ,$$

modified stability number, Lamberti and Tomasicchio (1997)

where

H_s	= significant wave height
H_k	= characteristic wave height, set to the average of the 1/50 highest waves
C_k	= set to H_k/H_s = 1.55 for deep water when the wave heights are Rayleigh distributed
D_{n50}	= $(W_{50}/\rho s)^{1/3}$
L_o	= deep water wave length based on mean wave period.
W_{50}	= median stone weight
T_z	= mean wave period
g	= acceleration of gravity
s_{mo}	= $2\pi H_s/(gT_z^2)$
s_{mk}	= characteristic wave steepness, set to 0.03
β_0	= angle between the mean wave direction and the normal to the longitudinal axes of the breakwater trunk
ρ_s	= density of stone
ρ_w	= density of water

4.2 Reshaping of berm breakwaters for waves normal to the trunk

An important measure for the reshaping is the recession of the berm, Figure 4.1.

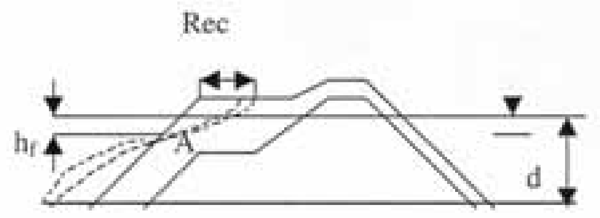


Figure 4.1 Sketch of recession, Rec.

The reshaping or recession can be obtained by the procedures developed by van der Meer (1990), van Gent (1995) and Archetti and Lamberti (1996). But then the accompanying computer programs must be available.

Hall and Kao (1991) investigated the influence of rounded stones on the reshaping of the berm. They arrived at an equation for the recession of a homogenous berm as follows:

$$\frac{\mathrm{Re}\ C_{KH}}{D_{50}} = (-10.4+0.51 \left[\frac{H_s}{(\frac{\rho_s}{\rho_w}-1)D_{50}}\right]^{2.5})+7.52\ (\frac{D_{85}}{D_{15}}) - 1.07(\frac{D_{85}}{D_{15}})^2 + 6.12 P_R \qquad (4.1)$$

where

D	= sieve diameter ≈ 1.2 D_n (Tvinnereim (1981))
D_{85}	= 85% of the stones have a diameter less than D_{85}
D_{15}	= 15% of the stones have a diameter less than D_{15}
P_R	= percentage per number of rounded stones in the armour

Tørum (1998) analysed the dimensionless recession Rec/D_{n50} as a function of HoTo for several scale model test projects in different laboratories (Danish Hydraulic Institute (DHI), Denmark, and SINTEF, Norway) on berm breakwaters with a homogenous berm. The data were given as the mean recessions for several profiles for each test run. There was a considerable scatter in the recession results of different projects in the same laboratory and between results obtained in the different laboratories. Tørum (1998) could not find any explanations for the differences in the test results and attributed the differences to unknown differences in the test set-ups, test procedures etc. There is also an inherent scatter in the test results due to local variations of the stone diameters along the trunk, Tørum and Krogh (2000). Tørum (1998) fitted a second order polynomial and later a third order polynomial to the data, Tørum et al (1999). Later on Menze (2000) and Tørum and Krogh (2000) added terms to take into account the gradation of the stones and the water depth. The recession equation arrived at is then:

$$\frac{\mathrm{Re}\ c}{D_{n50}} = 0.0000027(HoTo)^3 + 0.000009(HoTo)^2 + 0.11(HoTo) - (-9.9 f_g^2 + 23.9 f_g - 10.5) - f_d \qquad \text{Eq. (4.2)}$$

where

f_g	= D_{n85}/D_{n15}, gradation factor. Eq. (4.2) is valid for $1.3 < f_g < 1.8$.
f_d	= depth factor.

The depth factor has been analysed for two dimensionless depth, d/D_{n50} = 12.5 and 25, and is set preliminary to:

$$f_d = -0.16\left(\frac{d}{D_{n50}}\right)+4.0 \quad \text{within the range } 12.5<d/D_{50}<25 \qquad (4.3)$$

where

d is the water depth in front of the berm breakwater.

It has been observed that the reshaped profiles go through the intersection with the original profile at a depth h_f, Figure 4.1. As an approximation h_f can be obtained from:

$$\frac{h_f}{D_{n50}} = 0.2\frac{d}{D_{n50}} + 0.5\text{ , within the range } 12.5<d/D_{n50}<25 \qquad (4.4)$$

Tørum (1998) analysed also the scatter of the dimensional recession data in the following way:

$$\frac{f - f_k}{f_k} = f(HoTo)$$

where

f = data point Rec/D_{n50} for a given HoTo value.
f_k = value after second degree polynomial fit
f(HoTo) = function of HoTo.

For the data at hand the scatter of the data was apparently independent of HoTo. The standard deviation of $(f-f_k)/f_k$ was 0.337.

It is not possible to directly compare the recession given by Hall and Kao, Eq. (4.1) and Tørum (Eq. (4.2). However, if it is assumed that H_s = 6.0 m, T_z = 10 s, D_{n50} = 1.5 m (D = 1.25 m), D_{85}/D_{15} = 1.8, P_r = 0 and assuming "deep" water, Eq. (4.1) gives Rec_{KH} = 4.9 m while Eq. (4.2) gives Rec = 10.0 m.This difference was noticed by Tørum (1997) and may be due to differences in running the tests. The Hall and Kao data may be another set of data differing for unknown reasons from other data similar to the previous mentioned differences between data obtained from different test series at SINTEF and DHI.

Menze (2000) carried out laboratory tests for multi layer-berm breakwaters. The berm breakwater model was a model of the recently constructed Sirevåg berm breakwater in Norway, Figure 4.2, but the results have been analysed from a general point of view.
Figure 4.3 shows the dimensionless recession relation vs. HoTo for homogenous berm together with data for multi-layer berm breakwaters. The multilayer data are taken from tests at DHI, Juhl and Sloth (1998), Profile 1, Profile 2 and profile 3. The set-up 1 (ρ_s = 2700 kg/m^3) and set-up 2 (ρ_s = 3100 kg/m^3) data are from Menze (2000) on tests on the Sirevåg multilayer berm breakwater. The dimensionless recession and HoTo for multilayer berm breakwaters have been based on D_{n50} for the largest stone class.

The multi-layer berm breakwater allows a better use of the quarry stone material than the homogenous berm breakwater. The dimensionless recession for multi layer berm breakwaters is to some extent larger than for the homogenous berm breakwater when the D_{n50} for the largest stone class is used to calculate HoTo and Rec/D_{n50}. An equation for the recession of multi layer berm breakwaters has not yet been developed, but the results obtained by Menze (2000) indicates that the recession will be larger for a multi layer berm than for a homogenous berm, provided the same gradation of the cover stones

Menze (2000) also carried out tests with cover stones with two densities, ρ_s = 2700 kg/m^3 and 3100 kg/m^3 with about the same gradation, f_g = 1.14. Figure 4.4 shows the results of Menze (2000).

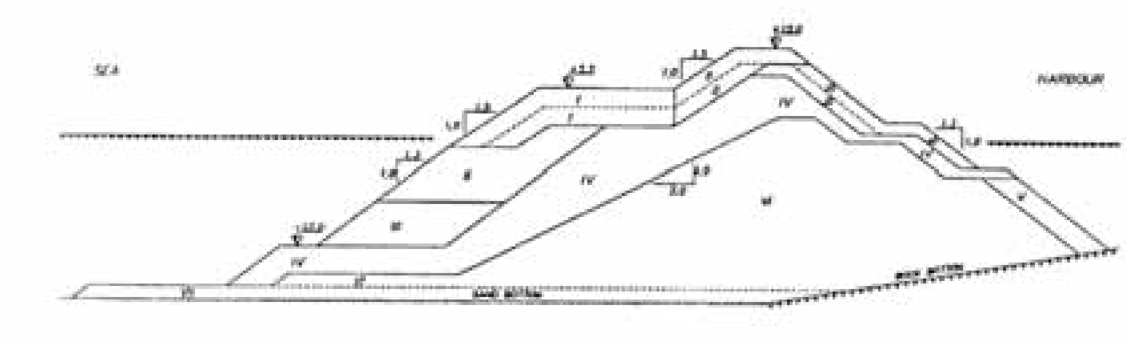


Figure 4.2. Multi layer berm breakwater. Sirevåg berm breakwater, Norway.
Cross section from outer part, water depth d = 17 m. Design 100-year waves H_s = 7.0 m, T_z = 10.6 s.
Class I stone W = 20 – 30 tons, gradation factor f_g = 1.1. Class II stones 10 –20 tons,
Class III stones 4 – 10 tons and Class IV stones 1 – 4 tons. Sigurdarson et al (2000).

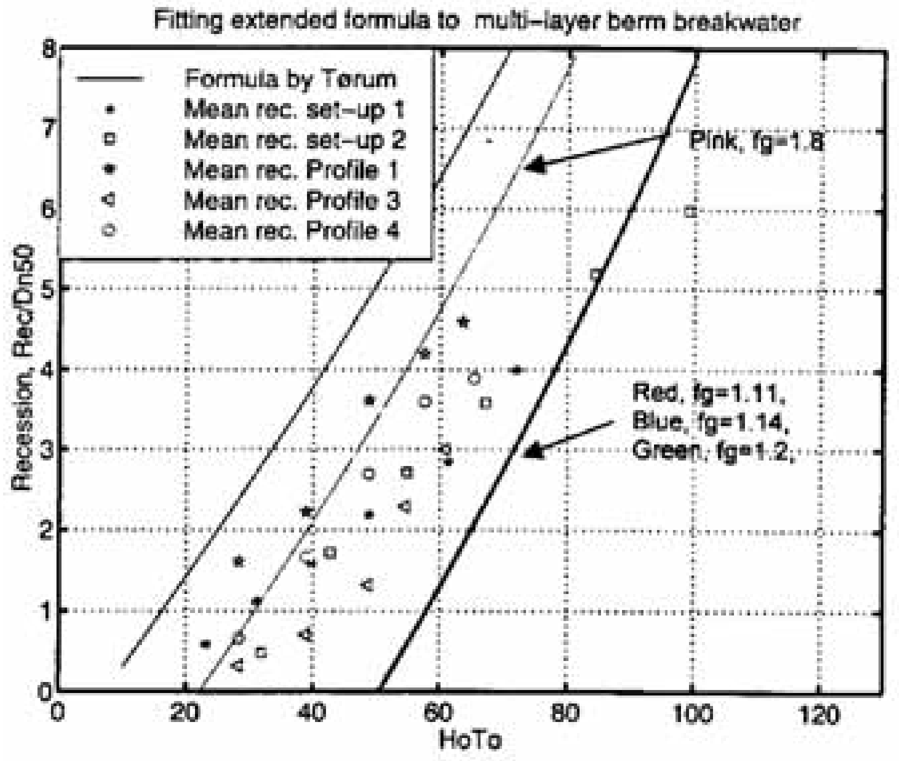


Figure 4.3 Recession of a multi layer berm breakwater. "Formula by Tørum" is Eq. (4.1) without depth correction and f_g = 1.8. "Pink f_g=1.8" is Eq. (4.1) with depth correction and fg=1.8. "Red, fg = 1.11 etc" are Eq (4.2) with depth and gradation corrections. Menze (2000). Note that Eq. (4.2) is not strictly valid for $fg_g \approx 1.14$, the gradation for Class I stones of the Sirevåg multi layer berm breakwater.

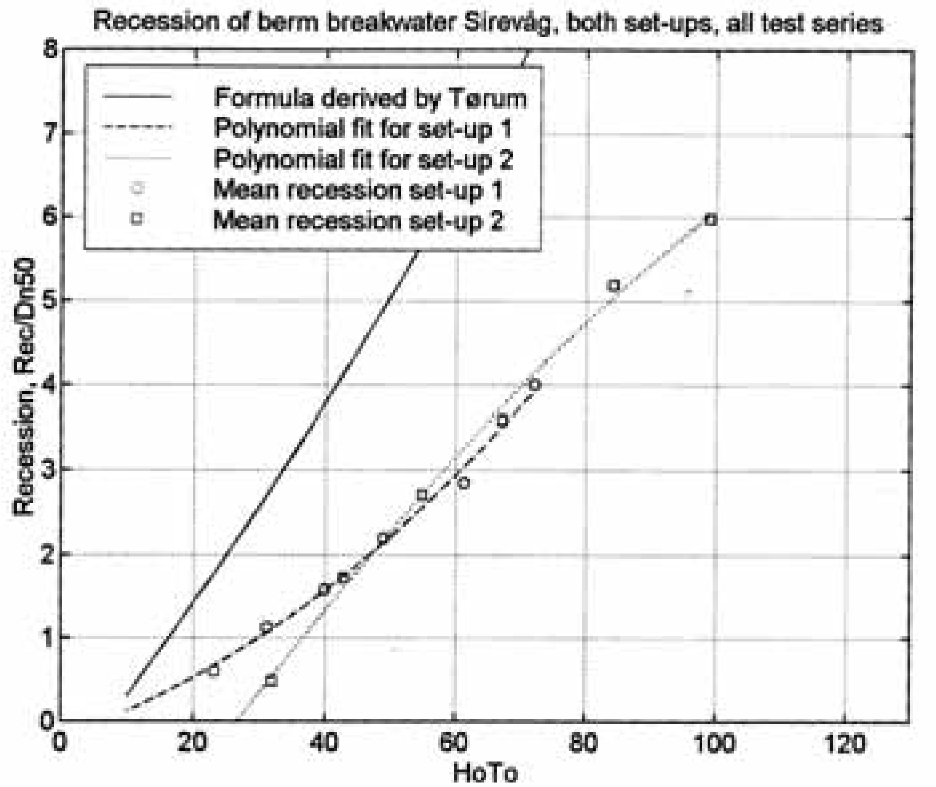


Figure 4.4. Recession of multilayer berm breakwaters for different stone densities.
Set-up 1 is for ρ_s = 2700 kg/m^3 and Set-up 2 is for ρ_s = 3100 kg/m^3. "Formulae derived by Tørum" is eq. (4.1) with depth corrections, d/D_{n50} = 25, but without gradation correction. Menze (2000).

4.3 Long shore transport for oblique waves

Stones on a berm breakwater start to move when $N_s \cong 1.5$-2.0; mobility is low when $2<N_s<3$ and when $N_s>3$ mobility increases very rapidly. A berm breakwater reshapes into a statically stable profile if Ho< ≈ 2.7. For Ho> ≈ 2.7 the berm breakwater reshapes into a dynamically stable berm breakwater, e.g. the cross-section remains stable, while the individual stones move up and down the breakwater slope.

The stability number N_s characterises wave intensity only by its height. Ahrens (1975), van der Meer (1988), Vrijling et al. (1991) and van der Meer and Veldman (1992) point out the relevance of wave period in mound stability and stone movements, particularly when horizontal movements are included. They propose a mobility index

$$N_s^*=(H_s^2 L_0)^{1/3}/\Delta \cdot D_{n50} \text{ and}$$
$$H_o T_o = H_s/\Delta \cdot D_{n50} \,.\, T(g/D_{n50})^{1/2}$$

Since $L_0=gT^2/2\pi$ both depends on product H_sT and provides the same information if Δ does not vary significantly. In fact the three mobility indexes (N_s, N_s^* and HoTo, where $Ho \equiv N_s$) are related by:

$$HoTo = N_s^{2/3}\sqrt{2\pi/(\Delta s_p)} = \sqrt{2\pi\Delta}(N_s^*)^{3/2} \tag{4.5}$$

Lamberti et al. (1994), Lamberti & Tomasicchio (1997) and Archetti & Lamberti (2000) conducted extensive research to obtain detailed information on armour stone movement along the developed profile of a reshaping breakwater for its typical mobility range $1.5<N_s<4.5$. An appropriate mobility index was defined, empirical correlation between mobility and displacement frequency and step-length were given and a conceptual longshore model was defined. They defined a modified mobility index accounting for wave obliquity and for the effects of non-Rayleigh distribution of waves in shallow water, beside wave height and wave period:

$$N_s^{**} = \frac{H_k}{C_k \Delta D_{n50}}\left(\frac{s_{mok}}{s_{mk}}\right)\cdot (\cos\beta)^{2/5} \approx \frac{0.89 H_{kb}}{C_k \Delta D_{n50}} \tag{4.6}$$

where suffix b means in breaking conditions; C_k is introduced so that $H_k/C_k k=H_s$ for a Rayleigh wave height distribution and is set to $C_k=H_k/H_s= 1.55$ for deep water when $H_k=H_{1/50}$. s_{ok} is an average normal wave steepness (0.03 is suggested) so that the numerical values of N_s^{**} and N_s are identical for orthogonal attack of waves of normal steepness in deep water conditions. Therefore the same threshold values suggested for N_s do also apply in this case. The threshold values are also dependent to some extent on stone gradation and water depth, Menze (2000), Tørum and Krogh (2000).

Table 4.1 shows the mobility criterion.

If N_s is used as mobility index, only the effect of wave height is accounted for. If HoTo is used as mobility index, H and T are combined with the same exponent. If N_s^* is used as mobility index H and T are combined with exponents 2 and 1 intermediate between those in Eqs. (4.5) and (4.6).

When the wave intensity exceeds the mobility threshold, stones do move, and if waves are oblique a net long shore transport may occur which may cause erosion and deposition problems.

Vrijling et al. (1991) and van der Meer and Veldman (1992) proposed a formula to evaluate the longshore transport on the trunk, which is useful in a range of high mobility for berm breakwaters ($3.3<N_S<8$).

$$S=5\lozenge 10^{-5} \cdot (H_0T_{0P}-105)^2 : \text{ for } H_0T_{0P} >105 \tag{4.7}$$

Formula (4.7) is based on model tests by Burcharth and Frigaard (1987, 1988)

where suffix *p* refers to the use of the peak period. This formula returns the along-structure transport measured as the expected number of transported stones per wave and is verified in the mobility range H_0T_{0P}=100-400, i.e. in a range of mobility more representative of reshaping breakwaters.

Tab. 4.1 Mobility criterion (the criterion depends on stone gradation)

Regime	Ns≡Ho or N_s^{**}	H_0T_0	Ns*
Little movement	<1.5-2	<20-40	<3.4 – 5.4
Limited movement during reshaping, statically stable	1.5-2.7	40-70	5.4 – 7.8
Relevant movement, dynamically stable	>2.7	>70	>7.8

In the original formula, Vrijling et al. (1991), the threshold value was 100 for HoTo and the multiplying factor $4.8x10^{-5}$. The empirical relationship was reviewed by Frigaard et al. (1996). Based on additional data the reanalysis led to a change in the multiplying factor to $8.6x10^{-5}$. This variability of calibration constant shows the effect of non-represented phenomena. These are mainly wave obliquity and the length of antecedent reshaping, since mobility evidently decreases during reshaping.

The resolution of long shore transport during these tests was around 0.005 stones/wave, which in a storm lasting a few thousand waves corresponds to 10 stones passing a section. This represents a mobility of some technical relevance, and therefore the technical threshold for dynamic stability should be assumed somewhat below the threshold value of this formula.

Eq. (4.8) does not take into account the effect of the obliquity. It overestimates the transport for low and high obliquity but provides an adequate estimate for the obliquity range $15^o < \theta < 35^o$ for which longshore transport is near to the maximum. θ is the angle between the wave direction and the normal direction to the trunk.

Lamberti et al. (1994) and Lamberti and Tomasicchio (1997) present a longshore transport formula calibrated in a wide range of mobility conditions for berm breakwaters at incipient mobility (1994) up to cobble beaches conditions (1997). The authors also provide an empirical correlation between mobility and displacement frequency, and mobility and mean displacement length, established in the range $1.5<N_S^{**}<3.5$ typical of berm breakwaters:

$$N_{od} = N_d \, x \, \frac{D_{n50}}{B} = 2.05 \, x \, N_s^{**} \, x \, (N_s^{**} - 2)^{2.2}$$

if $N_s^{**}>2$, else 0 (4.8)

$$D_s^{**} = l_d \, tan\, gh(kd) \, / \, D_{n50} = 1.4 \, N_s^{**} - 1.3 \qquad (4.9)$$

where N_d the number of units displaced at least once in 1000 waves, B is the length along the breakwater trunk of the observed area and l_d is the average length of displacements. Figure 4.5 shows in a semi-logaritmic presentation that a strict threshold of movement does not exist and that its definition depends essentially on what is assumed to be irrelevant.

The amount of movement can also be defined as surface damage level:

$$S_s = N_d \, D_{n50}^2 / A \qquad (4.10)$$

where A is the observed area. S_s represents the probability that a generic stone in the surface layer of the reshaped profile is displaced at least once during the 1000 waves attack. The empirical relation between N_{od} and S_s is:

$$N_{od} = (13.2 \pm 16\%) \, x \, N_s^{**} \, x \, S_s \qquad (4.11)$$

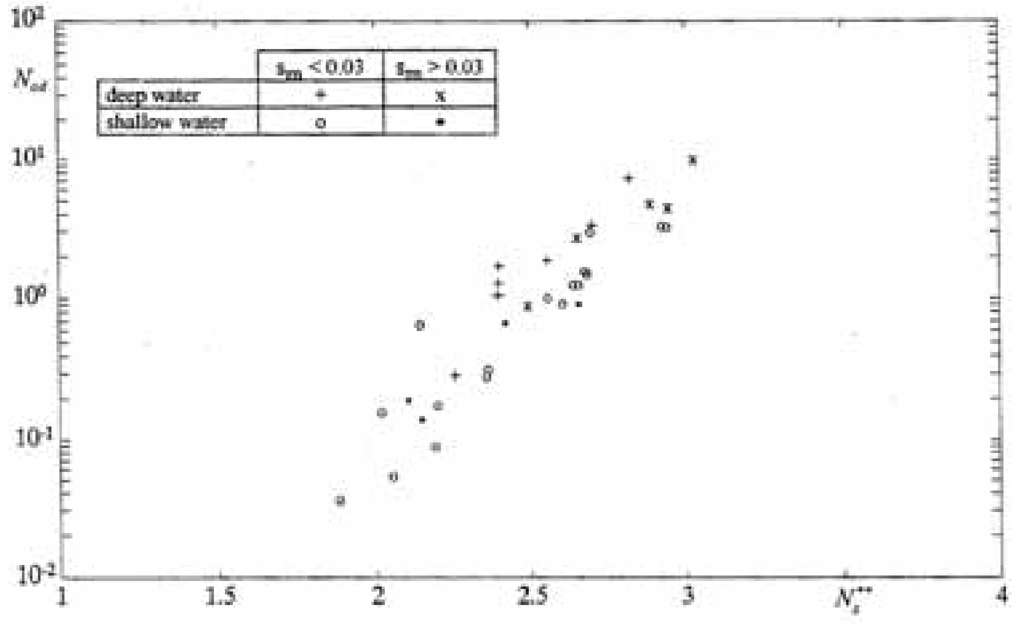


Figure 4.5 Semilog representation of the empirical relation between N_{od} and N_s^{**}. Lamberti & Tomasicchio (1997).

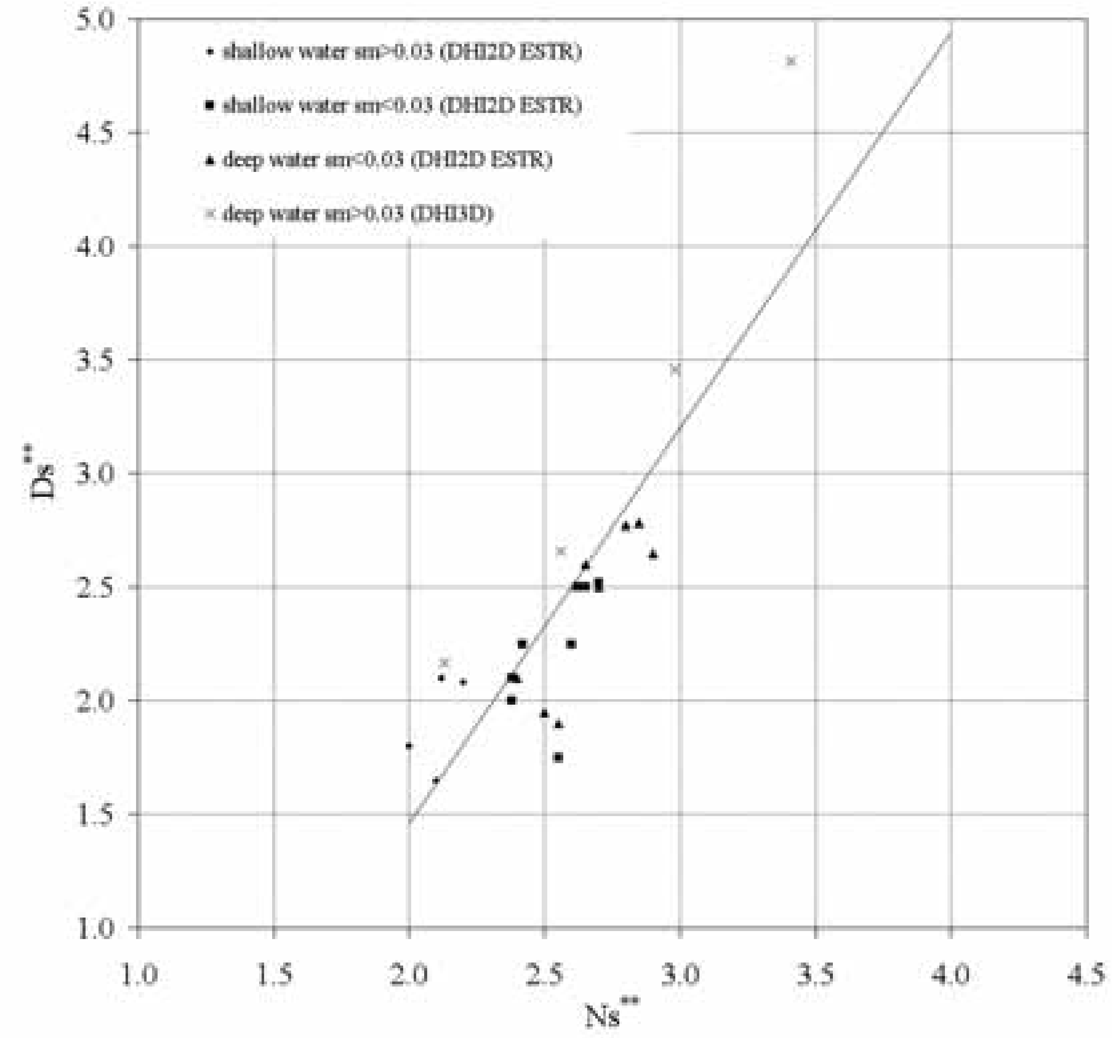


*Figure. 4.6. The empirical relation between number of movements N_{od} and displacement length D_s** and mobility N_s**.*

Figure 4.6 presents a plot of the scaled number of movements N_{od} and mean distance D_s** and the modified stability number; the former is multiplied by the squared ratio at the structure toe (ratio between the vertical and the horizontal dimensions of the particle orbit). This factor includes the effect of both the water depth and the wave period. If the exponent of *tanh* was 1, displacement length would increase as the horizontal dimension of the orbit. A larger exponent is required empirically in order to completely compensate for water depth variation and could be explained as the effect of surf beat, for instance. The standard deviation of displacement length ranged between 0.02 and 0.06; its variation is not significant particularly when it is compared with variation of the mean length.

Assuming that stones do move with the same obliquity as the waves at breaking, the number of stones transported beyond a section, S_N , is:

$$\frac{S_N}{sin\ \beta_{kb}} = \frac{l_d}{D_{n50}} \times \frac{N_{od}}{1000} = f(N_s^{**}) \qquad (4.12)$$

The conceptual model for long shore transport, calibrated with data from wave basin tests, compares favourably with existing long shore transport data in the mobility range considered

Figure 4.7 shows the experimental data from Burcharth and Frigaard (1987, 1988) and of van der Meer and Veldman (1992), who considered developed profiles of reshaping breakwaters attacked by irregular waves. In comparison a low mobility range and a high mobility range can be distinguished. In the first range the correct representation of the effect of wave obliquity can be checked, whereas in the second range the extrapolations of displacement relations can be checked. The continuous line is represented by Eq. (4.9) and the dashed line by Eq. (4.13)

$$N_{od} = exp(4.8N_s^{**} -12) \qquad (4.13)$$

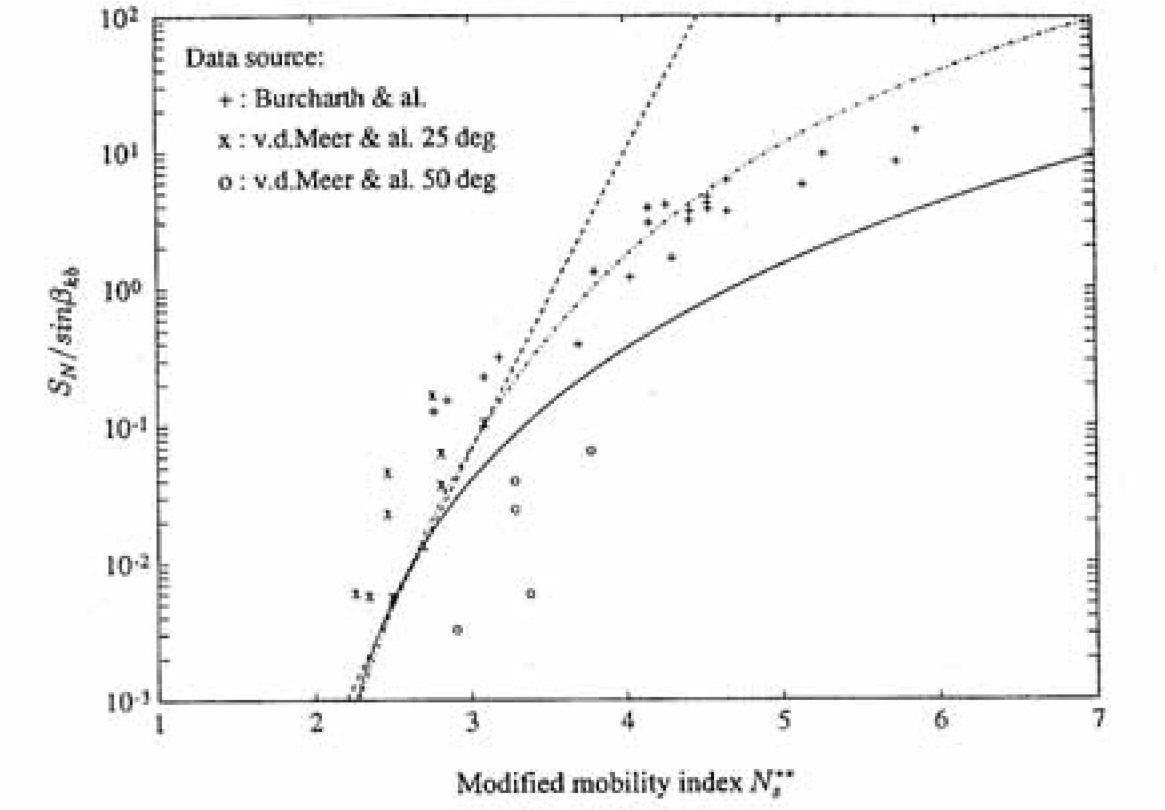


Figure 4.7. Data of Burcharth and Frigaard (1987, 1988) and van der Meer and Veldman (1992) interpreted according to the structure of Eq. (4.9). Onshore wave conditions are used to evaluate mobility and displacement obliquity. Lamberti & Tomasicchio (1997).

Table 4.2. Movement criterion.

Regime	Displacement N_{od}	Displacement S_s
No movement	< 0.10	<4 10^{-3}
Limited movement during reshaping, eventual static stability	0.10 - 2	4 10^{-3} – 2 10^{-2}
Relevant movement, dynamic stability	> 2	> 2 10^{-2}

Data analysis by Lamberti & Tomasicchio (1997) shows that the criteria for movements in terms of S_s and N_{od} in a 1000 waves storm duration are the ones approximately given in Table 4.2.

Alikhani (2000) give the following threshold of stone movement, based on tests at DHI, Alikhani et al (1996) for βo> 45°:

$H_oT_{op} \geq \frac{50}{\sqrt{sin2\beta_o}}$ during the reshaping phase, and (4.14)

$H_oT_{op} \geq \frac{75}{\sqrt{sin2\beta_o}}$ after the reshaping phase (4.15)

Alikhani gives the following longshore transport equation based tests at DHI, Alikhani et al (1996):

$$S=0.8 \cdot 10^{-6} \sqrt{cos\beta_o}\,(H_oT_{op}\sqrt{sin2\beta_o} - 75)^2 \qquad (4.16)$$

where

S = stone per wave
T_{op} signifies that T_0 is based on the peak period.

Frigaard et al (1996a) investigated the influence of stone shape on recession and longshore transport. The length ratio for the stones, defined as the longest side divided by the shortest side, was as shown in Table 4.3.

The average stone mass was W_{50} = 0.0157 kg, the gradation ratio W_{85}/W_{15} was 2.8 (or D_{n85}/D_{n15} = 1.41) and the mass density ρ_s = 2720 kg/m^3 for all stone classes. The tests were run with the waves at an angle of 45° to the breakwater trunk.

Table 4.3. Length ratio for the different stone classes used in the tests by Frigaard et al (1996).

	Length ratio, l/b
Rounded stones	1.0 – 1.5
Normal stones	1.5 – 2.5
Flat stones	2.5 – 3.5
Mixed stones	1.0 – 3.5

The conclusions of Frigaard et al (1996) were the following:

“Any measurable difference in the profiles with different stone types were not observed in the tests.

It was not possible to quantify a difference in the amout of overtopping water. Though it seemed that the berm constructed of the flat stones, i.e. the smoothest slope after reshaping, was producing the most overtopping water. Thus this difference was believed to be insignificant.

A very significant longshore transport were measured for the different stone classes. Longshore transport rates for the flat stones were three to five times higher than the transport rates for the round stones.”

4.4. Stone velocity

During stone movements abrasion and breaking can occur. The tension stresses inside the impacting stones depend on the type of rock and the status of the rock (fissures and cracks) and on kinematics of moved stones. Archetti & Lamberti (1999) give an indication of the “mean” velocity of stones in correlation to the mobility number and consequent stresses during impacts based on observation of stone displacements during tests carried out in the basin of DHI (Juhl et al., (1996)), and based on simple assumptions. The “mean” velocity is also useful for evaluating the travel distance for a moving stone. Tørum and Krogh (2000), on the other hand, give information on the “peak” velocities when a stone is moving on a reshaping berm breakwater. Tørum and Krogh (2000) subsequently used the velocity information to evaluate the probability of the breaking of the stones when they roll on the berm breakwater slope. “Mean” velocity is the mean velocity during an event of motion of a stone, while the “peak” velocity is the peak velocity during an event of motion of a stone.

Lamberti and Archetti (1999) based their model for stone velocity on the concept of Engelund and Fredsoe (1976) for fluvial sediment transport and extended it to waves:

$$V_s = \alpha \, x \, u_* \, (1 - \beta \sqrt{\vartheta_{cr} / \vartheta}) \qquad (4.15)$$

where ϑ is the Shields parameter defined as

$$\vartheta = u_*^2 / (g \Delta D_s) \qquad (4.16)$$

u_* is the friction velocity, proportional to fluid velocity outside the wall boundary layer, ϑ_{cr} is the Shield number at incipient motion (critical) conditions, β is related to the ratio between static and dynamic friction and α is the (constant) ratio of fluid velocity at particle elevation to friction velocity. The equation is based on the assumption that a particle is dragged by the fluid-particle velocity difference at a distance from the wall approximately equal to particle size and is slowed down by dynamic friction on the bed.

An equation similar to (4.15) can be applied to stones moving under waves, Archetti and Lamberti (2000):

$$V_s = \alpha \, x \sqrt{g \, x \, H} \, x \, (1 - \beta \, x \sqrt{N_0^{**} / N^{**}}) \qquad (4.16)$$

where, according to the conceptual derivation, β is the square root of the ratio between static and dynamic friction factors (≅ 0.7 for stones), α is a constant and N_0^{**} stands for the value of the used mobility index at mobility thres-hold.

From the DHI 3D-tests, Alikhani et al(1996), the “mean” stone velocity during each movement was observed. It was noticed that during the same wave attack several stones move, characterised by a wide range of velocities. It is therefore necessary to characterise the velocity distribution, and estimate velocity statistics (i.e. mean, maximum) as function of the wave statistics (generally H_S).

The standard deviation of the “mean” stone velocities increases with the mobility. In fact the maximum “mean” velocity for the highest mobility is 5 times the mean “mean” velocity and for the lowest mobility less than 2 times the mean “mean” velocity. This trend is described by the increasing of δ in Eq.4.17:

$$V_{stones} = \sqrt{H_s \cdot g} \cdot \alpha \left(1 + \left(\frac{N_o^{**}/2}{N_s^{**}}\right)^{\gamma}\right)^{\delta} \qquad (4.17)$$

The best estimates for α, γ and δ are given in Table 4.3:

Table 4 3 – Best parameter estimates for velocity statistics, Eq. (4.17).

Fixed parameters γ, N_0**	Best estimates				Coeff. of determin. % r^2
	N_0^{**}	α	γ	δ	
Vmedian	2.0	0.064	0.5	0	71
Vmean	2.0	0.11	0.5	0.5	73
Vmax	2.0	3.87	0.5	3	97

The velocity standard deviation (V_{sd}) is well fitted with the following equation:

$$\frac{V_{sd}}{\sqrt{gH_s}} = a + b \cdot N_s^{**} \tag{4.18}$$

with a=-0.061, b=0.034, r^2=91%.

In Figure 4.8 the observed and the estimated mean, median and max stone "mean" velocities vs. the predicted ones through Eq. (4.17) are plotted.

While "mean" velocity values are significant for the evaluation of transport, the maximum or "peak" velocity during an event of stone motion is significant for impacts between stones and the evaluation of abrasion and possible breaking of the stones.

Tørum et al (1999) and Tørum and Krogh (2000) measured the maximum or "peak" velocity during a wave cycle during 2D wave flume tests on a berm breakwater. Figure 4.9 shows maximum dimensionless stone velocities found in a wave cycle vs. Ho.

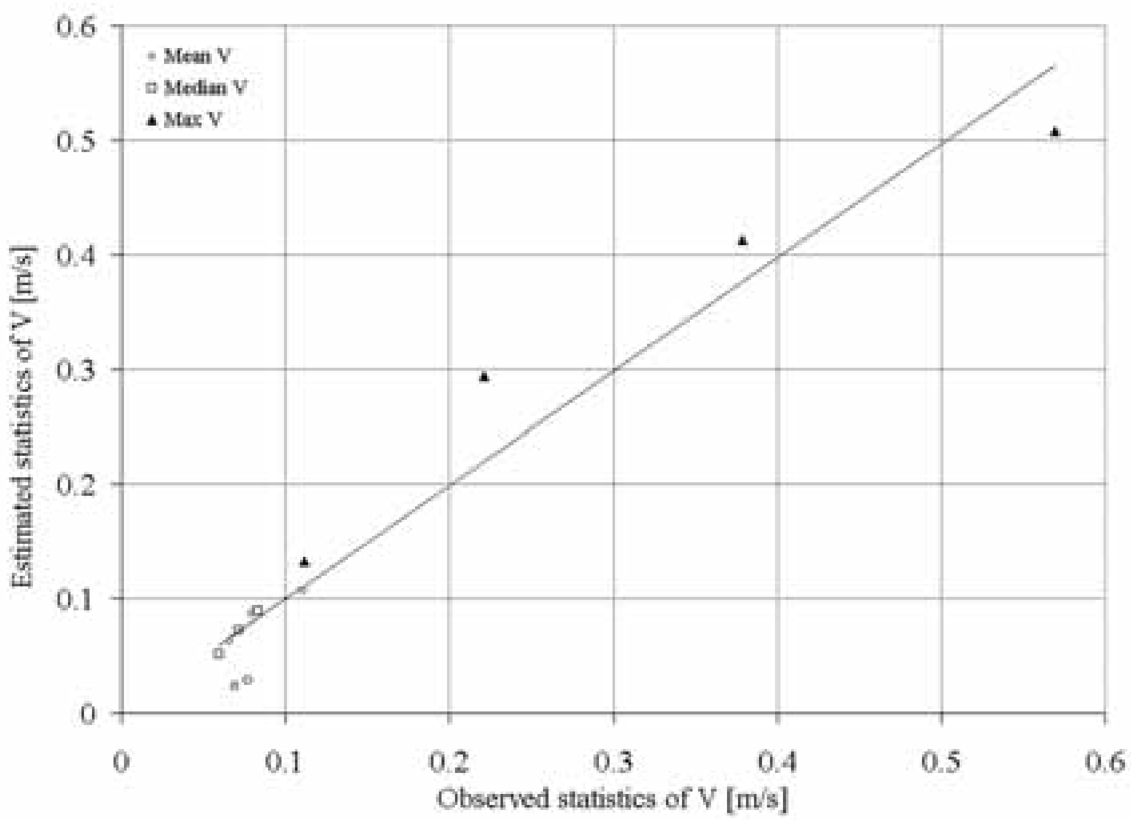


Figure 4.8. Comparison between observed mean velocities during a wave cycle and values estimated with Eq. (4.17) for the different statistics.

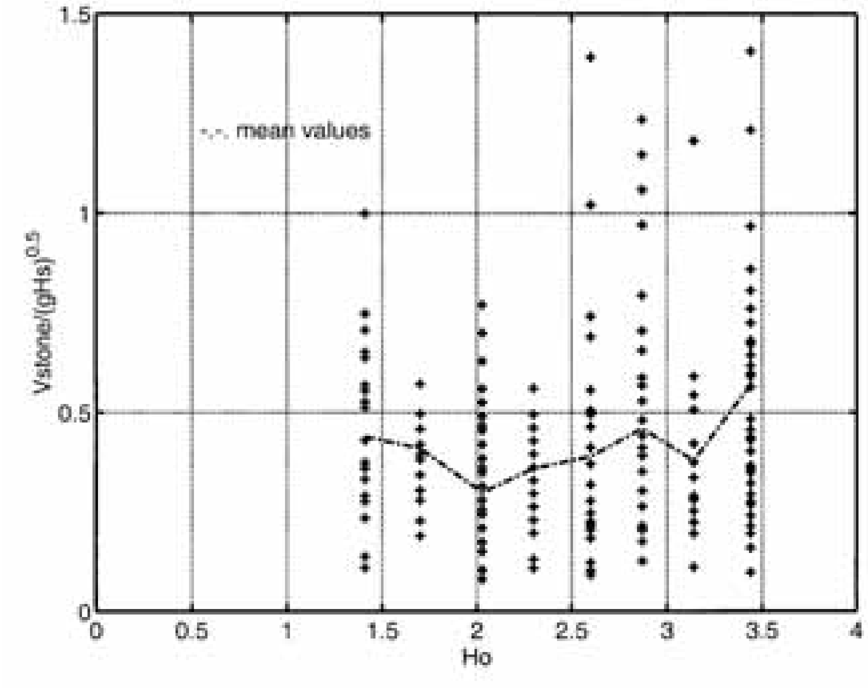


Figure 4.9 Non-dimensional maximum "peak" translational velocity during reshaping vs. Ho. Tørum and Krogh (2000).

Figure 4.9 shows that there is, as expected, a scatter in the results. Since the mean "peak" value of $V_s/\sqrt{gH_s}$ is almost independent of Ho and since the statistical distribution was more or less independent of Ho, all the dimensionless velocity data was merged together. A two parameter and a three parameter Weibull distribution function was then fitted to the data as shown in Figure 4.10.

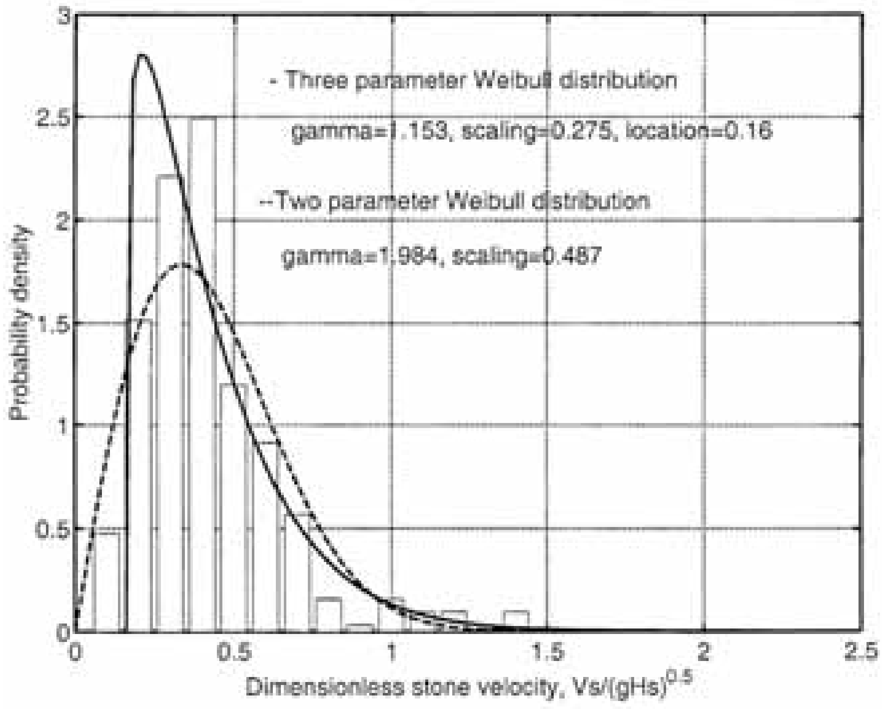


Figure 4.10 Two and three parameter Weibull probability density functions fitted to the "peak" stone velocity data. Tørum and Krogh (2000).

Since the three parameter Weibull distribution does not include values of the dimensional velocity between 0 and 0.16 (location factor = 0.16) it was decided to use the two parameter distribution for further statistical analysis (see chapter 5.2 Stone breaking strength).

The two parameter Weibull probability density function is given by:

$$f(X) = \frac{\gamma}{X_s^{\gamma}} X^{\gamma-1} \exp\left(-\left(\frac{X}{X_s}\right)^{\gamma}\right) \qquad (4.19)$$

The cumulative two parameter Weibull distribution function is given by:

$$F(X) = 1 - \exp\left(-\left(\frac{X}{X_s}\right)^{\gamma}\right) \qquad (4.20)$$

where in this case
$X = V_s/\sqrt{gH_s}$.
γ = shape factor, in this case 1.984
X_s= scaling factor, in this case 0.487.

When comparing the two sets of data and analysis, attention should be paid to the following:

- Data from Archetti and Lamberti (2000) refer to "mean" velocity during a movement event after reshaping took place, wheras Tørum et al (1999, 2000) refer to maximum or "peak" velocity during a movement event during the reshaping phase. This explains to a large extent the difference between the median/mean velocity observed from the different measurements and analysis.
- In the two measurements there is a tendency of growth in the extreme velocities with the mobility index Ho, but this is probably a non-essential feature for actual berm breakwaters since the mobility conditions are not so widespread.

4.5 Redistribution of stone size down the berm slope

It has been observed by Alikhani et al (1996), Tørum (1997) and Tørum and Krogh (2000) that there is a redistribution of the stone size along the slope of the berm of a reshaping berm breakwater. The size of the stones, W_{50}, becomes less than the original value at or slightly below the still water line (SWL), while W_{50} becomes larger at the bottom of the slope. Figure 4.12 shows the variation of the dimensionless $W_{50}/W_{50,homogenous}$ along the slope from the tests of Tørum and Krogh (2000).

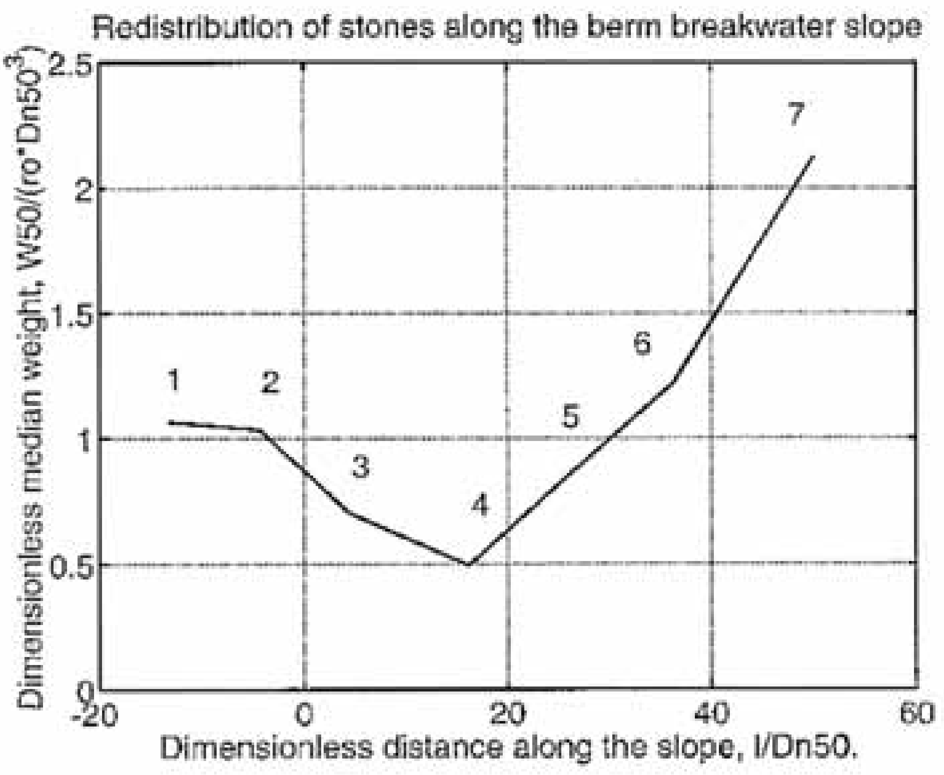


Figure 4.12. Variation of $W_{50}/\rho_s D_{n50,homogenous}$ along the slope of a reshaped homogenous berm breakwater. 0 m-distance is at still water line and $l/D_{n50,homogenous}$ = 55 is at the bottom. $D_{n50,homogenous}$ = 0.0228 m (the nominal "original" diameter. Ho=3.4.

The degree of redistribution, $W_{50swl}/W_{50bottom}$, depends on HoTo and the gradation factor f_g. Alikhani et al (1996) report $W_{50swl}/W_{50bottom} = 0.5$ for f_g=1.8 (homogenous berm) and Ho = 4.0. They also report that the redistribution is less for narrower gradations. Tørum and Krogh (2000) found $W_{50swl}/W_{50bottom} = 0.33$ and $W_{50upper}/W_{50bottom} = 0.25$ for Ho=3.4 and $f_g = 1.7$ (homogenous berm). $W_{50upper}$ is the smallest stone weight found somewhat below the still water line. Similarly Tørum and Krogh (2000) found $W_{50swl}/W_{50bottom} = 0.50$ and $W_{50upper}/W_{50bottom} = 0.42$ for Ho = 2.7.

The observed redistribution is significant, but the observations are based on a limited number of stones in each sample. This poses a question of sample variability. In their reshaped berm breakwater model Tørum and Krogh (2000) marked seven fields, each 20x20 cm, from the top of the breakwater model and down to the bottom of the breakwater. Then the uppermost stone layer was removed and the stones were weighed. The sample size varied from 39 to 79 stones, depending on the size of the stones. The nominal size of the "homogenous undisturbed" stones was in the range D_{n50} = 0.021 – 0.023 m determined from three samples, with 170 stones in each sample. This shows already that there is sample variability.

The subject of sample variability has not been properly addressed in breakwater testing and design. There will even in well mixed laboratory stone masses be inherent scatter between results from finite size samples. In the field there is also the inherent scatter due to the not so well mixed and controlled stone material as in a laboratory. Lefebvre et al (1992) and Belfadhel et al (1996) refer to the measurements on stone sizes in the upper wave protection layer of stones on earth fill dam slopes. The number of stones in each sample was approximately 70 and D_{n50} was in the order of magnitude 0.7 m. The ratio W_{50max}/W_{50min} for each dam (four to eight samples) varied considerably and was in the range 1.4 – 2.7 with a mean value $(W_{50max}/W_{50min})_{mean} = 2.2$. This range is much larger than we will encounter in a model test set up, but there is no information on this variability of "local" values of W_{50} on a breakwater.

Although there might be a sampling "problem", the observed variations of W_{50} along a reshaped berm breakwater slope is considered to be a "real" redistribution. The mechanism for this redistribution is not well understood. A possible mechanism is that the rotational velocities for the stones are the same for the large and small stones. Since most of the stones roll along the slope, the distance the stones will travel in a given fraction of the wave period will be larger for the larger stones than for the smaller stones. Also the larger stones will roll more easily than the smaller stones. Thus, the larger stones will move more easily into the steeper part of the slope, from where they will not be so easy to move by the waves. The smaller stones will be stopped on the flatter part of the slope, where they can be moved more easily up-slope (for the dynamic stable conditions).

The fact that larger stones move more easily than smaller stones on a rough slope like the one which is present on a rubble mound breakwater slope, may also be a contributing factor to the observed redistribution. In natural talus under rock cliffs, the largest stones are located at the foot of the talus.

This redistribution mechanism is interesting from the "stability" point of view of reshaping breakwaters. If the breakwater is designed and built with the reshaped profile as its as built profile, the stone size will be more homogenous along the slope and the stability of the breakwater may be increased. Redistribution leads to a wastage of the larger stones from the area where the stones are subjected to highest wave forces. Van Gent's (1996) numerical results also indicated that when the largest stones are placed in the top layer, these stones disappear from the region with the highest wave forces and that this leads to a wastage of the larger stones once reshaping has occurred. However, as an alternative, construction of a berm breakwater with original S-form would require development of new construction procedures.

4.6. Stability of roundheads

The roundhead is a vulnerable part of a breakwater and special attention has to be taken in design to avoid severe damage to the roundhead. For a reshaping berm breakwater the stones on the roundhead should not move into the area behind the breakwater head in order not to block shipping lanes or otherwise be a hazard to navigation.

Physical model tests have been performed on the stability of roundheads, e.g. Burcharth and Frigaard (1987, 1988), Jensen and Sorensen (1992), van der Meer and Veldman (1992), Juhl et al. (1996), Tørum (1997), Menze (2000).

Burcharth and Frigaard (1987, 1988) carried out their tests on a berm with a high elevation (at the breakwater crest level). They found that Ho should be smaller than 3 if significant continuous erosion is to be prevented. Figure 4.13 shows typical examples from the same study of the erosion of a berm breakwater head exposed to rather mild and severe wave conditions corresponding to Ho = 3.5 and 5,4 respectively.

The tests of van der Meer and Veldman (1992) show that stones from the roundhead to some extent were thrown into the area outside the original toe behind the breakwater roundhead for Ho = 2.7 and HoTo = 96.

Juhl et al. (1996) give general indications on the localisation of the maximum damage for Ho up to Ho = 4.0. The recession of the berm edge and the eroded and accumulated volumes were the damage indexes.

- 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°) up to two times higher than for 45°. 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 wave steepness is of particular importance, and 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°.

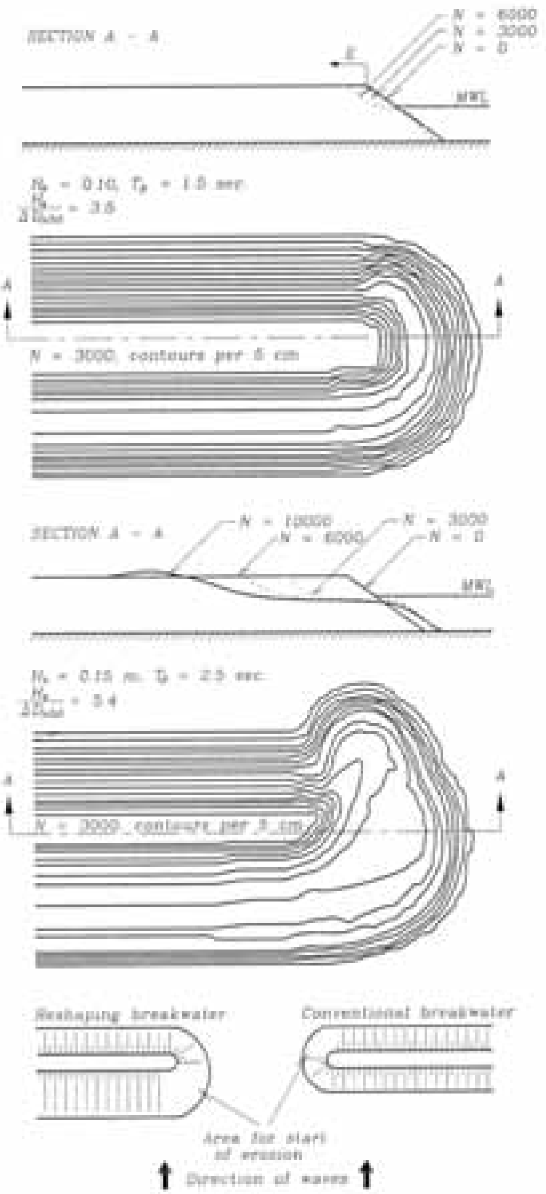


Figure 4.13. Example of erosion of berm breakwater head. Aalborg University Hydraulic Laboratory.

The roundhead did not reshape much during the tests of Menze (2000) for values of HoTo up to 70 (Ho = 2.6).

Gilman (1987) reports on reshaping of the head of the berm breakwater on St. George Island in Alaska's Bering Sea. During the winter of 1986 – 87 storms occurred which approached the design storm in intensity (Ho~2.5-2.8, HoTo~60-70). Even with the half complete nature of the structure, the berm roundhead performed very well and suffered only minor berm profile modification.

It seems that there will be no significant stone movements into the area behind the roundhead, if a berm breakwater is designed as a statically stable reshaped berm breakwater, e.g. Ho < 2.7 and HoTo < 70.

4.7 Stability of the rear side

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

The suggestions for the design are the following:

$R_C/H_S \cdot s_{OP}^{1/3} = 0.25$ start of damage
$R_C/H_S \cdot s_{OP}^{1/3} = 0.21$ moderate damage
$R_C/H_S \cdot s_{OP}^{1/3} = 0.17$ severe damage

A lower value of $R_C/H_S \cdot s_{OP}^{1/3}$ means greater overtopping and therefore more damage; decreasing both terms (crest height and wave steepness) produces an increase in overtopping and in rear side damage.
Andersen et al. (1992) give, based on a specific test programme, the following criterion for the rear side stability. The rear side damage was defined as settlement of the rear side armour layer, which in some cases was followed by an exposure of the core material.

$$R_c > \tan\alpha \; \frac{H_{m0}}{\Delta\sqrt{s_{02}}} \; D_{n50} \cdot \frac{\mu\cos\beta - \sin\beta}{C_D + \mu C_L} \qquad (4.13)$$

with α and β respectively the sea side and the rear side slope (Figure 4.14), C_D and C_L the drag and lift coefficient, s_{02} wave steepness, based on T_{02}, and μ the resistance against rolling and sliding. For the stone material Andersen et al (1992) applied, μ equals 0.9. For this value of μ, expression 4.13 was calibrated to fit experimental data. The best agreement was obtained with $C_D+\mu C_L = 0.08$.

5. REFLECTION, RUN UP, OVERTOPPING AND TRANSMISSION

This chapter presents the hydraulic response of berm breakwaters, with particular attention to the following:

- Wave reflection;
- Run up and run down level;
- Overtopping discharge;
- Wave transmission.

5.1 Governing parameters

Wave conditions for the incident waves at the toe of the structure are principally described by the characteristics wave height $H_{1/3}$ or H_{m0}, the mean or the peak wave period (T_m or T_P) the obliquity β (= equal to the angle between the mean wave direction and the breakwater axis), and the water depth h. Characteristic wave height values that will be used are the 2 % wave height $H_{2\%}$, or other wave height percentiles.

Parameters describing mound characteristics are the permeability P, the crest height, R_c, the stone diameter D_{n50} and the slope angle α. The slope angle α for berm breakwaters is not as precisely defined as for conventional rubble mound breakwaters. For reflection analysis (see chapter 5.2) the slope angle on the flat part of the reshaped berm has been used. For the rear stability another definition has been used (see Chapter 4.7). Van Gent (2001) proposes a method without the need to distinguish a berm in

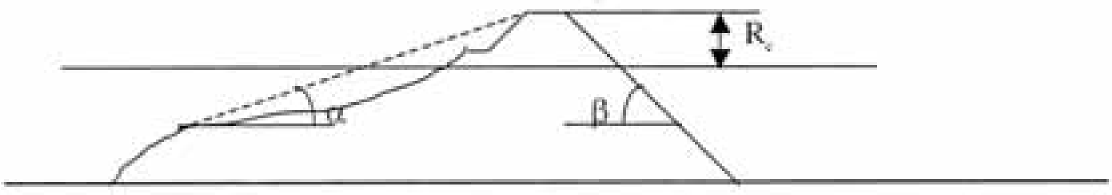


Figure 4.14. Definition of geometric parameters. Andersen et al (1992).

a reshaped profile. However, this method has not been applied to berm breakwaters so far. If the profile is in equilibrium α is dependent on other variables as the waves attack. Normally mobility and overtopping are quite moderate and this almost fixes the values for R_c and D_{n50}.

Therefore, the independent variables are:
Hs, Tp, β and α.

The influence of wave period is often described through the height to wave length ratio, resulting in the wave steepness:

$$s_{om}=2\pi H/gT_z^2$$

The most useful parameter describing wave action on a long slope, and some of its effects, is the surf similarity parameter ξ, also called the Iribarren and Battjes parameter:
$\xi=tan\alpha/s_{om}^{0.5}$ where α is the slope of the mound and s_{om} is a fictitious wave steepness in shallow water.

5.2 Reflection

Berm breakwaters are constructed in harbour and coastal areas to dissipate and reflect wave energy within and from the berm with the aid of natural armour materials.

Porous structures allow a part of the incident energy to penetrate into the protected area and at the same time dissipate and reflect a significant part of the wave energy.

The interaction of berm breakwaters with incoming waves is similar to the interaction of waves with the conventional rubble mound breakwaters. The differences are due to the following points:

- after reshaping, the seaward slope of the berm breakwater is more gentle than that of a traditional breakwater - on average between 1:3.5 and 1:5 on the flatter part of the slope.
- due to the presence of the berm, the length of the reflecting wall is wider than for a traditional rubble mound breakwater.

Reflection data for random waves on conventional breakwaters for smooth and armoured slopes at angles between 1:1.5 to 1:2.5 (smooth) and 1:1.5 to 1:6 (rock) are available (Van der Meer, 1993). The sources are:

- Allsop and Channell (1988);
- Postma (1989)
- Van der Meer (1989).

Reflection can be quantified by the reflection coefficient:

$$K_r = \frac{H_r}{H_i} \qquad (5.1)$$

where H_r is the reflected wave height and H_i is the incident wave height. For irregular waves the significant wave height is frequently used.

Based on experimental tests, the following empirical laws have been developed as best fit of the data:

- Seelig (1983):

$$K_r = a \cdot \xi_p^{\,2}/(b+\xi_p^{\,2}) \qquad (5.2)$$

where $\xi = tan\ \alpha/(2\pi H_s/gT_z^2)$, Iribarren number
and a = 0.6, b = 6.6 for a conservative estimate of rough permeable slopes.

- Postma (1989):

$K_r = 0.14\xi_p^{\,0.73}$ based on his data and (5.3)
$K_r = 0.125\xi_p^{\,0.73}$ based on Allsop and Channel data.(5.4)
$K_r = 0.071\ P^{-0.082}\ cota^{-0.62}\ s_{op}^{-0.46}$ (5.5)

where P is the permeability factor, van der Meer (19987) - ranging between P=0.1 for impermeable core to P = 0.6 for no core and no filter (armour stones in the entire cross section). A value of P = 0.3 – 0.4 is a "normal" value. (van der Meer, 1988).

Figure 5.1 shows measured reflections compared with the proposed equations. In this case the slope angle α is defined as the angle of the flatter part of the slope.

2D test results on a berm breakwater at DHI 96 are well described by the following equation, which also seems to fit well the DHI 92 data:

$$K_r = 0.35\xi_p^{\,0.17} \qquad (5.6)$$

Alikhani (2000) proposes the following empirical equation for reshaped structures (4<cotgα<5) with permeability P = 0.6 based on DHI 1996 data:

$$K_r = 0.044 \cdot s_{op}^{-0.46} \qquad (5.7)$$

Estramed data (Lamberti et al. 1994) for tests in shallow water, show a high reflection coefficient in shallow water at the structure toe, on the order of 0.60, and a significantly smaller value outside the breaker zone.

It is evident, looking at Figure 5.1, how the proposed equations are not able to describe reflection. A more careful analysis of experimental conditions and analysis methods

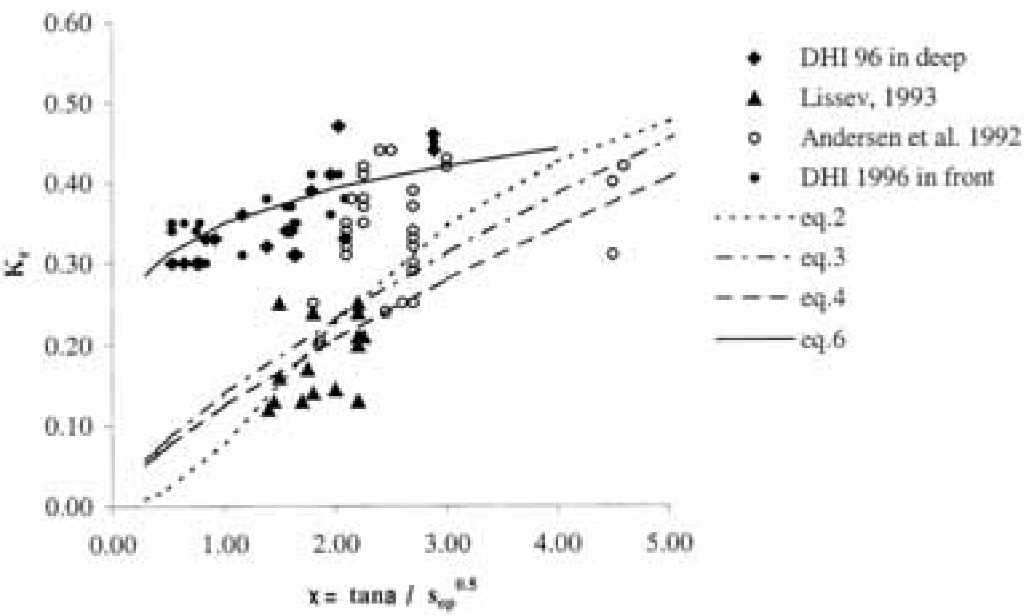


Figure 5.1 Empirical model for reflection (lines) and experimental data on berm breakwaters. Eqs. 2,3,4 and 6 relates to Eqs. (5.2), (5.3), (5.4) and (6.6).

should be performed in order to assure data homogeneity. Moreover, in the comparison between the different data sources, it must be taken into account that reflection may depend on the position of its evaluation. In shallow water when the reflection is evaluated at a certain distance from the structure, the steepest waves might break between the measurement point and the structure. This may result in less reflection of the shortest waves and only the longer waves are reflected, with the result of a lower overall reflection coefficient K_r than for a structure in deeper water when all components are reflected.

The above formulae account for the effects of the structure slope and the wave steepness. These are the main parameters for a straight stable slope where permeability is not too important. On a reshaped berm breakwater the slope varies continuously, making the definition of a single representative slope and of ξ questionable. The greater bulk permeability of the berm, the smaller surface roughness and the different shape all cause some shape effects compared to traditional breakwaters and rubble mound slopes for which formulae 5.1 to 5.5 were initially proposed. In shallow water wave conditions, the relative depth h/H_S is an important parameter affecting the wave breaking process, as well as the profile length to wave length ratio, i.e. phase lag between reflection from different parts of the profile. What figure 5.1 clearly shows is that the Iribarren-Battjes parameter alone cannot explain the variety of conditions that were used during the experiments and that, surprisingly, berm breakwaters do not exhibit reflection coefficient much smaller than conventional rubble mound breakwaters.

5.3 Run-up

Run-up, R_u, and run-down, R_d, depend on a large number of variables including incident wave characteristics, water depth, bottom slope, structure geometry, permeability and roughness. The dependence of water depth and bottom slope may be governed by the effect of the applied wave conditions at the toe. Description of wave run-up and run-down is based on physical model tests. Relative run-up is frequently given by $R_{u2\%}/H_s$, generally expressed as a function of the surf similarity parameter.

A general run-up formula for low Iribarren numbers has the following:

$$R_u = max\ (\xi\ x\ H_s, k) \qquad (5.8)$$

this means that R_u increases with the wave height; as the R_u level exceeds the crest height R_c or the mound slope becomes steeper and waves reflect on the structure slope, the value of R_u is upper bounded by some value k.

Pilarczyk (1990) found that run-up on slope protection decreases with increasing artificial berm width and the reduction rapidly falls off once a certain minimum width is exceeded: for $B>0.25\ L_0$ for non-breaking waves and for $B>4H_S$ for strong breaking waves, i.e. for $H_S/L_0>0.03$. In Table 5.1 he gives an indication of the reduction factor for run-up due to the berm (i.e. $r_b=R_b/R_u$ ratio between run-up on a bermed slope and run-up on a uniform slope) larger than the minimum mentioned above for a water depth at the berm $d>0.5\ H_S$ and $H_S/L_0>0.03$.

Table 5.1 Run up reduction factors. Pilarczyk (1990).	
Slope cotga	r_b
3	0.50 to 0.60
4	0.60 to 0.70
5 to 7	0.75 to 0.80

Ahrens and Ward (1991) express the run-up reduction due to the berm r_b as an exponential function of the berm geometry. They observed that reduction in run-up as a result of the berm is a rather modest 20%. In order to estimate the maximum wave run–up on a revetment fronted by a rubble berm, they suggested the following empirical equation:

$$\frac{R_{max}}{H_{mo}} = \exp\left(0.695 - 11.269\frac{H_{mo}}{L_o} - 0.158B'\right) \quad (5.9)$$

where R_{max} is the elevation of the maximum observed run-up and B' is a dimensionless berm width defined as:

$$B' = \frac{B}{\sqrt{H_o L_o}}\,\frac{h_b}{d_s} \quad (5.10)$$

and B is the berm width, h_b the height of the berm above the toe and d_s the water depth at the toe of the structure.

Several tests were performed by de Waal and van der Meer (1992), van der Meer (1993) and van der Meer and Stam (1992) in order cosider the effects of permeability, obliquity and shape of the structures. Based on the data van der Meer (1998) proposed the following design formula:

$$\frac{R_{u2\%}}{H_s} = \max\,(1.6\,\xi_{op} ?\quad {}_{f}\gamma_{\beta}) \quad (5.11)$$

The particular cases of bermed and/or rough slope under oblique wave attack can be derived from the general equation introducing the reduction factor ∓ $_{b}\gamma_f\gamma_\beta$ where γ_b represents the effect of the presence of a berm, γ_f represents the effect of the rough surface and γ_β represents the effect of wave attack obliquity. Eq. (5.11) is valid in the range $0.5 < \gamma_b\,\xi_{op} < 4$ or 5.

Effect of the berm width B in Eq. (5.11) and in the following equations has been given graphically by Eq (5.12) in van der Meer (1998):

$$\gamma_b = 1 - \frac{B}{L_{berm}}\left(1 - 0.5\frac{d_h}{H_s}\right) \quad (5.12)$$

where d_h is the mean submersion of berm (the distance between the mean of the berm height and the mean water level).

For $H_s < d_h < 2xH_s$ the reduction factor γ_b increases linearly to $\gamma_b = 1$. The suggested reduction factor γ_f for a rough slope due to the rubble layer of the berm is $\gamma_f = 0.60$, van der Meer (1998).

For the evaluation of reduction due to obliquity of wave γ_β, van der Meer (1998) gives the following recommendation:

$\gamma_\beta = 1 - 0.0022 \times \beta$ (β in degrees). (5.13)

More details on reduction factors are available in van der Meer (1998).

De Rouck et al (1998) and De Rouck et al (2001) measured run-up in full scale of the Zeebrugge rubble mound breakwater with 25 ton grooved cube concrete armour blocks. Run-up was also measured in three model test setups of the same structure in three different laboratories. The results indicate that the run-up on scale models on rubble mound structures, from which overtopping data mainly comes, is smaller than in full scale. But the results also showed that there was a significant difference between the run-up results of the three different laboratories. However, the reasons for the differences are not fully resolved.

5.4 Overtopping

When run-up level exceeds the crest height waves overtop the structure. The evaluation of the overtopping discharge is normally related to the evaluation of run-up.

In the definition of overtopping discharges, it is common to use the mean discharge q per meter run [m^3/s per m]. Critical values of q for various situations are suggested in CIRIA/CUR (1991) when vehicles, pedestrian and buildings are in danger. If pedestrian, vehicles or buildings are not in danger, more overtopping may be allowed. But the stability of the rear side has to be ensured, Chapter 4.7.

Van der Meer and Jansen (1995) proposed the following formulas for wave overtopping on slopes:

Breaking waves on foreshore:

Average: $Q_b = 0.06\exp(-5.2R_b)$ (5.14)

Recommended: $Q_b = 0.06\exp(-4.7R_b)$ (5.15)

where

$$Q_b = \frac{q}{\sqrt{gH_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}}$$

$$R_b = \frac{R_c}{H_s}\,\frac{\sqrt{s_{op}}}{\tan\alpha}\,\frac{1}{\gamma_b\gamma_d\gamma_f\gamma_\beta}$$

Non-breaking waves on foreshore:

Average: $Q_n = 0.2\exp(-2.6R_n)$ (5.16)
Recommended: $Q_n = 0.2\exp(-2.3R_n)$ (5.17)

Where

$$Q_n = \frac{q}{\sqrt{gH_s^3}}$$

$$R_n = \frac{R_c}{H_s}\,\frac{1}{\gamma_b\gamma_d\gamma_f\gamma_\beta}$$

γ_b = reduction factor taking into account a stepped slope
γ_d = depth reduction factor = $1-0.03(4-d/H_s)^2$ for $d/H_s < 4$
= 1 for $d/H_s > 4$
γ_f = friction reduction factor
γ_β = reduction factor for oblique wave attack
= $1 - 0.0033\beta$, β in degrees

Van der Meer (1998) proposed a new formula for wave overtopping on dikes in accordance with wave run on a slope:

$$Q = \frac{q}{\sqrt{gH_s^3}} = \frac{0.06}{\sqrt{\tan\alpha}}\gamma_b\xi_{op}\exp\left(-4.7\frac{R_c}{H_s}\frac{1}{\xi_{op}\gamma_b\gamma_f\gamma_\beta\gamma_\upsilon}\right) \quad (5.18)$$

but not greater than:

$$\frac{q}{\sqrt{gH_s^3}} = 0.2\exp\left(-2.3\frac{R_c}{H_s\gamma_f\gamma_\beta}\right) \quad (5.19)$$

where γ_v is a reduction factor due to a vertical wall (if any) on the slope.

Overtopping measurements on reshaped berm breakwaters have been made by Lissev (1993), Lissev and Tørum (1996) and Kuhnen (2000). Lissev (1993) and Kuhnen (2000) carried out the overtopping tests by collecting the water that topped over the rear edge of the breakwater as shown in Figure 5.3. Since a berm breakwater is generally rather "transparent" in the crest region, the total overtopping discharge is larger than the discharge over the crest.

Irregular wave trains with wave height varying between H_S = 0.146 m to H_S= 0.295 m and wave period between T_P = 2.0 to 2.8 were used during the Lissev (1993) tests. The resulting coefficients in Eq. (5.15) are the following: Q_0 = 4600, c = -21.
The coefficient Q_0 is clearly dimensional. Alikhani introduced a scaling factor for mean overtopping $R_c^2T^{-1}$ and rewrote Eq. (5.16) as:

$$Q = \frac{C_0}{R_c^2T^{-1}}\exp(C_1F') \quad (5.20)$$

Lissev (1993) data corresponds to C_0 = 1288000 and C_1 = -21 in Eq. (4.20).

Lissev (1993) data also gives as average for head on non-breaking waves:

$$Q = \frac{q}{\sqrt{gH_s^3}}\,1.5\exp(-2.1\frac{R_c}{H_s}) \quad (5.21)$$

No analysis of overtopping over berm breakwaters has been made where the slope angle has been considered.

Figure 5.4 shows the dimensionless overtopping over a reshaped berm breakwater vs. dimensionless crest height for the Lissev (1993) and Kuhnen (2000) data. A comparison has been made with van der Meer and Jansen (1995) data averaged (av), Eq. (5.16), and recommended (rec), Eq. (5.17) for non-breaking waves on the foreshore. For the van der Meer and Jansen (1995) relation the γ-factors have been given the following values: γ_f = 0.50, γ_d = 1.0, γ_b = 1.0 and γ_b = 1.0. The Kuhnen (2000) data compare well with Lissev (1993) data, while the van der Meer and Jansen (1995) relation for a conventional rubble mound breakwater shows, as expected, larger overtopping values than the reshaped berm breakwater data. The results of Sigurdarson and Viggoson (1994) indicate, as expected, that the overtopping discharges for non-reshaped berm breakwaters are less than for reshaped berm breakwaters. It should be mentioned that generally there is a tremendous spread in experimental data on wave overtopping.

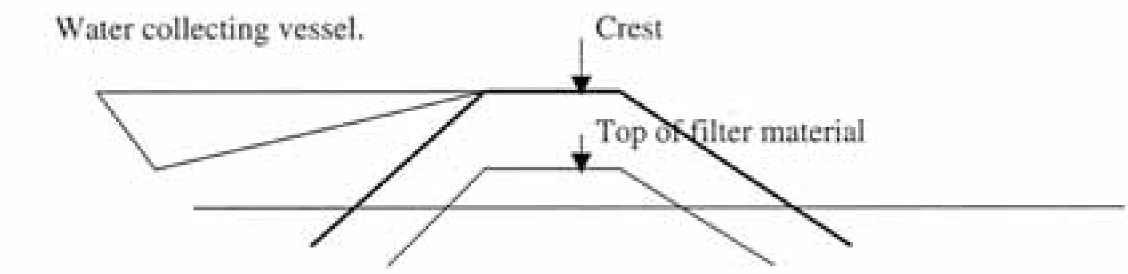


Figure 5.3 Water collection for the Lissev (1993) and the Kuhnen (2000) overtopping tests.

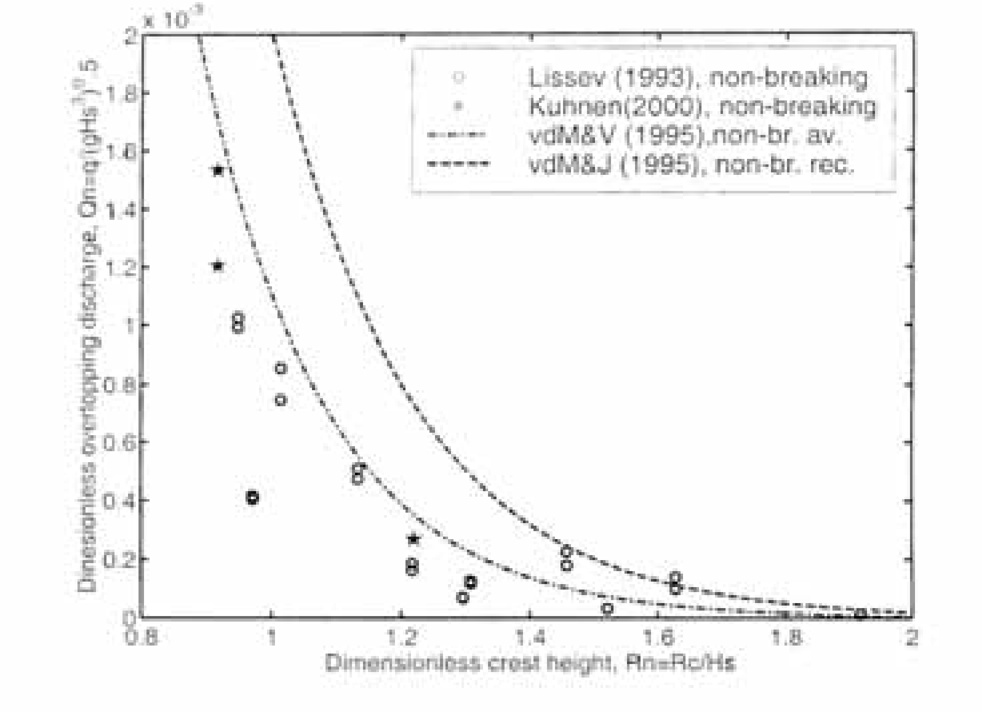


Figure 5.4. Dimensionless overtopping discharges. Non-breaking waves. Comparison between overtopping on conventional (vdM&J) and berm breakwaters (Lissev, Kuhnen).

5.5 Transmission

Berm breakwaters may be constructed with a lower crest in respect to conventional breakwaters, so quantification of transmission is important in their design. The severity of transmission is described by the transmission coefficient

$$K_t = \frac{H_t}{H_i} \qquad (5.22)$$

where H_t is transmitted wave height and H_i is incident wave height respectively.

The transmission over berm breakwaters is dependent on the geometry of the structure, in particular the freeboard height and the berm width, how much the berm has reshaped and on the coarseness of the core material.
Indications of transmission for conventional breakwaters in relation to the relative crest height are given by van der Meer (1990), based on data from experiments performed by Seeling (1980), Powell and Allsop (1985), Daemrich and Kahle (1985), Ahrens (1987) and van der Meer (1988). For relative crest height $1.2<R_C/H_S<2.0$ the transmission coefficient is $K_t = 0.10$.

Other tests and analysis have been performed by van der Meer and D'Agremond (1991), van der Meer and Daemen (1994), van Gent (1995) and Juhl and Sloth (1998).

The empirical equation reads:

$$K_t = max\ \{a \cdot R_c/D_{n50} + b,\ 0.075\} \qquad (5.23)$$

with $a = -0.24+0.031 \cdot H_i/D_{n50}$ and $b = 0.51-0.52s_{op} +0.0323 \cdot H_i/D_{n50} -0.0017 \cdot (B/D_{n50})^{1.84}$, where B is the width of the breakwater.

Goda (1969) proposed the following empirical equation:

$$K_t = 0.5\left(1 - sin\ \frac{\pi}{2\alpha}\left(\beta + \frac{R_c}{H_i}\right)\right) \qquad (5.24)$$

where α and β are functions of the breakwater shape.

Results measurements of wave transmission through berm breakwaters are given by Lissev (1993), van Gent (1995) for $R_c/H_s>1.2$ and $K_t < 0.10$. The transmission in the case of Lissev (1993) and van Gent (1995) was partly due to overtopping and partly due to wave transmission through the core, which in both cases was relatively coarse and may not be representative for "real" core material.

Jacobsen et al (1999) investigated the wave propagation around berm breakwater heads. They found that wave heights along the inner side of a berm breakwater were less than for a conventional rubble mound breakwater. In the "far" field areas the difference was less.

6. MATERIAL SPECIFICATION

6.1 Rock type, rock quality and quarry yield

Smarason et al (2000) gives an account how rock quality is considered in Icelandic berm breakwater design. Their account may also be useful as a guide for others for their quarry investigations. The following is an excerpt of the paper of Smarason et al (2000).

"Commercial rock quarries are relatively few in Iceland. New quarries are therefore often needed for new harbour projects, although existing ones can sometimes be used. Selection of suitable quarries begins with inspection of geological maps and aerial photographs of the area adjacent to the planned breakwaters and the area is widened until successful. The search for suitable armour stone quarries is initially directed at any prominent thick lava flows, which may be accessible on low ground or in accessible benches or hillocks. Promising sites are visited and inspected visually for geological features such as rock type, weathering forms, pores, pore fillings (amygdales), alteration and joint density. Further investigation is performed through pneumatic drilling and core sampling at promising sites before bidding. Quarries may be located right by the breakwater structure or up to 40 km away. However, the transport distance is commonly 5 - 15 km. Core material is sometimes produced more economically in poorer quarries closer to the structure or dredged from sediments on the sea floor. Possible quarry yield and rock quality is weighed against transportation distance in each case, to optimise cost effectiveness and strength and durability of the structure. Environmental impact assessment is carried out for quarries exceeding 150.000 m^3 in total volume in situ of rock, or 220.000 m^3 of blasted rock, as the bulking factor is usually about 1.45 to 1.50."

"Scan-lines at horizontal and vertical rock exposures are used to measure the fracture density of the rock. Two scan-lines, at right angles to each other, are measured with a tape measure on the upper surface and also in vertical sections if possible. Pneumatic drilling is carried out to give an idea of the thickness of the rock, possible size of quarry and an idea of the soundness and alteration grade of the rock. Core drilling is usually carried out to give further information regarding spacing of discontinuities (joints and fractures), rock quality, specific gravity, absorption, point load index, freeze/thaw resistance and optical inspection in thin section. Measurements of discontinuity spacing in scan-lines and cores are used to establish an idea of the possible size distribution of blasted material from the rock. It is assumed, in the interpretation, that the shape of the stones is on average cubic."

"The conventional rubble mound breakwaters are usually built as rubble mound structures consisting of an inner core of fine material of quarry run, covered with an armour layer of stones. The armour layer of a conventional rubble mound structure usually consists of two layers of armour stones, or concrete armour units if armour stones are not readily available, and a filter layer or an under layer to prevent the finer material from being washed out. The armour layer extends about 1.0 to 1.5 times the design wave height above the design water level and to the same distance below the lowest water level. The size of the armour stone needed to resist the wave energy is proportional to the wave height in third power. This means that for conventional rubble mound breakwaters very large stones are often needed in large quantities. This design method can be characterised as a "demand-based design" (Leeuwestein et al. 1995)."

"The design philosophy of berm breakwaters aims at optimising the structure not only with respect to wave load but also possible yield from an armour stone quarry, which can be characterised as "supply-based design". The initial idea of berm breakwaters was that they should be wide voluminous structures, built of two stone classes with a wide size gradation, allowing a considerable reduction of armourstone size. These structures were allowed to reshape, with stones moving up and down the slope, into an S-shape profile, which was assumed to be a more stable profile and the structures are sometimes referred to as reshaped static or reshaped dynamically stable structures. Experience with a dynamically stable structure in Iceland has, however, demonstrated that when stones start to roll up and down the slope and hit each other, high abrasion and splitting of stones will occur. Voids will be filled up with smaller stones and the ability of the structure to dissipate wave energy will decrease. The forces acting on each rock unit on the slope will increase, which accelerates the dynamic movement of the stones and increases breaking and splitting even further."

"An "Icelandic type" of berm breakwaters has been developed, where the structure is less voluminous and more stable than the original berm breakwater concept anticipated.

An emphasis is put on maximising the outcome of the armour stone quarry and utilising this to the benefit of the design. The goal of the design of the Icelandic type berm breakwater is that it shall be non-reshaped statically stable. Only some minor deformation of the berm is allowed under design conditions, but reshaping into an S-shape profile is prohibited. It is recognised that the reshaping will increase during the lifetime of the structure, where stone quality is insufficient and because of repeated wave action. It is not necessarily the approach of the design to fulfil certain prescribed stability parameters, $Hs/\Delta D_{n50}$, but to look at the correlation between the armour stone quarry, size distribution and quality. It also takes into account the design wave, wave height, wave period and direction; water depth; function of the breakwater, for what purpose is it built, and whether wave overtopping is a problem or not. We have in many cases been able to design a berm with high stability of the armouring layer at no extra cost. Good interlocking by carefully placed stones is advantageous, especially on the front and at the edge of the berm."

"Close collaboration between the designer and geologist in the preparation of berm breakwater projects has proven very effective over the years. This has resulted in better designs and better use of the quarried material. This collaboration gives the designer the chance to fully utilise material from the quarry down to the smallest possible stone size and has often resulted in 100 % utilisation of the quarries. Close co-operation between the geologist and the project supervisor with the contractor is often necessary to achieve maximum results in the quarry. Blasting and sorting of armour stone is by no means an easy task and slight alteration of spacing and tilt of drill holes may at times help to improve blasting results. It has to be realised that the contractor and the buyer should work as a team aiming for the same goal. Experienced contractors rely on the predicted yield curves in their bidding."

"Recent developments in berm breakwater design have aimed at using large to extra large stones (10-20 tonnes and 20-30 tonnes) in the more exposed parts of the structures, as many of the better quarries are found to produce 10 to 20 % of armourstones exceeding 10 tonnes in size. As large backhoe excavators that can handle extra large stones have become readily available, we have started prescribing these large and extra stones to the advantage of the berm breakwater structures in some recent projects. A relatively low percentage (1-3 %) of the largest stone class can be an advantage for most breakwaters. This is not only true for extreme wave conditions where these extra large stone classes are most needed but also applies to more moderate wave load conditions and where quarries with lower size distribution are used."

"The Icelandic armour stone quality assurance programme, shown in Table 6.1, has been adopted and modified from CIRIA/CUR (1991) and Hardarson (1979). Important properties are rock type, density and absorption, strength (point load index), freeze/thaw resistance in cold climates, alteration and inner binding of minerals (in thin section under the microscope), and resistance to abrasion in abrasive conditions. Discontinuity spacing, for quarry yield prediction, is also an important factor. Formation thickness, overburden and transportation distance are also considered."

"Table 6.2 gives an overview of some important geological data on quarries for selected breakwaters in Iceland and Sirevåg in southwest Norway. It provides information on rock type, absorption, density, point load index (I_{S50}), freeze/thaw resistance (Swedish standard SS 13 72 44) as well as the predicted maximum quarry yield of armour stone over 1 tonne and large and extra large armour stone over 10 and 20 tonne for the individual quarries."

Although it is advantageous to design a berm breakwater as a non-reshaping berm breakwater - especially if the costs involved are not excessive - it may not always be feasible to do so with the quarry yield that can be provided. A method described in Chapter 6.2 may be used to assess if the stones from a specific quarry are suitable for reshaped berm breakwaters. Experience shows also that reshaping berm breakwaters are behaving satisfactory, e.g. the St.George berm breakwater in Alaska. The St. George berm breakwater is reshaped and experiences the design wave conditions frequently (on at least an annual basis). This probably causes rolling of the stones on the berm slope. There are apparently some broken stones on the St. George berm breakwater, indicating that one should be careful when designing for dynamic stable reshaping berm breakwaters.

The "beauty" of berm breakwaters is that they are "tough" and may easily be designed to withstand waves far above even the 100-year design wave height. This is to some extent dealt with in the Chapter 12 on "Probability Analysis of Hydraulic Stability".

Table 6.1.Guidelines for quality control of armour stone of igneous rocks.

Test	Excellent (A)	Good (B)	Marginal (C)	Poor (D)	Comments
Rock type	Gabbro, Porhyritic basalt, Dolerite	Granite, Anorthosite Ol.-tholeiite, Alkali basalt	Tholeiite basalt, Andesite	Rhyolite, Dacite, Hyaloclatite,	Guidelines for rock types without correlation to rock density.
Specific gravity (SSD) (tonn/m^3)	>2,9	2,65-2,9	2,5-2,65	<2,5	Density of rock is a good indicator of hydraulic stability in a breakwater.
Water absorption (%)	<0,5	0,50-1,0	1,0-2,0	>2,0	Important indicator of alteration and resistance to degradation, especially in cold climate.
Freeze/thaw test. Flaking in kg/m^2	<0,05	0,06-0,10	0,11-0,20	0,21-0,50	Swedish standard SS 137244 in a 3% NaCl solution for concrete.
Point Load Index $I_{S(50)}$ (MPa)	>8,0	5,0-8,0	3,0-5,0	<3,0	Correlates with rock dentity and indicates resistance to breakage of blocks.
Alteration of minerals	No alteration	Little alteration	Considerable alteration	Heavy alteration	Alteration inspected in thin sections.
Inner binding of minerals	Excellent	Good	Fairly good	Cleavage visible	Inspection in thin section.

The guidelines in Table 6.1 indicate general criteria on stone quality. A method to investigate more specifically the stone resistance against breaking when rolling on a reshaping berm breakwater is given in Chapter 6.2.

Table 6.2. Geological data on some quarries in Iceland and Sirevåg, Norway.

Locality	Quarry	Rock type	Absorption %	Density bulk dry	Density bulk ssd	Point Load index I_{S50}	Freeze/thaw Weight loss kg/m²	Predicted Max Quarry Yield >1 tonne	>10 tonne	>20 tonne
Akureyri	Krossanes	Tholeiite basalt	0,5	-	2,90	8,8	0,12	5	0	0
Bakkafjördur	Bakkafjördur	Tholeiite basalt	0,9	-	2,87	-	3,61	13	0	0
Blonduos	Uppsalir	Porph. basalt	0,51	2,87	2,88	9,1	0,15	32	9	4
Bolungarvik	Skalavikur heidi	Porph. basalt	-	-	2,86	-	0,05	34	5	2
Borgarfjördur - eystra	Os	Dolerite	0,55	2,90	2,92	6,3	-	21	4	2
Dalvik	Halshöföi	Porph. basalt	0,99	2,88	2,91	10,0	-	24	2	0
Djupivogur	Hamar	Porph. basalt	0,54	-	2,90		-	23	5	2
Gilsfjördur	Deild	Porph. basalt	0,67	2,87	2,89	10,0	-	51	20	10
Grundarfjördur	Mjosund	Alkali basalt	1,34	2,82	2,85	8,0	-	25	6	3
Keflavik	Helguvík	Olivine-tholeiite	1,38	2,79	2,82	-	0,00	31	5	3
Hornafjördur	Halsendi	Gabbro	0,32		3,00	10,8	0,04	47	21	15
Hornafjördur	Kambhorn	Gabbro	-	-	-	6,8	-	35	10	6
Hornafjördur	Smyrlabjörg	Porph. basalt	-	-	-	-	-	48	13	4
Husavik	Katlar	Olivine-tholeiite	1,28	2,88	2,92	6,4	0,05	10	0	0
Husavik	Hlidarhorn	Olivine-tholeiite	0,87	2,9	2,93	-	-	32	<10	5
Olafsfjördur	Gardur	Porph. basalt	0,60	2,9	2,91	-	0,04	33	6	3
Siglufjördur	Selskal	Porph. basalt	0,61	-	2,90	9,3	0,04	34	14	9
Sirevåg, SW-Norway	Quarry A	Anorthosite	0,26	2,68	2,69	10,8	-	54	25	17
Sirevåg, SW-Norway	Quarry B	Anorthosite	0,19	2,69	2,69	10,2	-	47	22	15
Sirevåg, SW-Norway	Quarry C	Anorthosite	0,38	2,66	2,67	9,8	-	51	23	17
Skagastrond	Asinn	Olivine-tholeiite	0,86	2,86	2,89	-	0,00	40	15	10
Thorshofn	Thorshofn	Olivine-tholeiite	1,35	2,81	2,85	7,7	-	25	4	2
Vopnafjördur	Grenisklettar	Porph. basalt	0,5	-	2,85	-	0,11	65	30	20
Vopnafjördur	V - Hraungardur	Porph. basalt	-	-	-	-	-	60	20	10

6.2 Stone breaking due to impacts while rolling

6.2.1 Breaking and abrasion strength of stones

Unlike other types of breakwaters, the berm breakwater is frequently designed to reshape. The allowed degree of reshaping may depend on various considerations. One of the items to be considered is the breaking strength of the stones to withstand the impacts they will encounter when they roll on the reshaping berm. Abrasion of the berm stones may also pose a problem, especially for dynamically stable berm breakwaters.

Although there have been some attempts to evaluate the stresses in rolling stones on a berm breakwater (Frigaard et al, 1996, Archetti and Lamberti, 1999), there has been no good method to evaluate the probability for breakage of stones rolling on a berm breakwater slope. Tørum and Krogh (2000) and Tørum et al (2000, 2002) developed a method to evaluate the probability of the stones being broken to help answer the question if berm breakwaters should be allowed to reshape or not. Their method requires 3 – 5 days of drop testing of fairly large stones in the selected quarry. Because the method is relatively new and specifically designed for berm breakwaters, the background and the procedures for the method are briefly described here.

There are three fundamentally different ways of reducing the size of rock pieces or stones on a berm breakwater by mechanical action:

1) impact breaking, 2) compression and 3) abrasion.

The first one could be illustrated by a hammer hitting a stone until it falls to pieces. The second one can be likened to squeezing the stone between plates until it cracks, and the third one would be like rolling the stone back and forth resulting in chipping off of small fragments. Impact breaking and abrasion are the most dominant mechanisms that break down armouring stones mechanically.

The breaking of moving stones when they hit other stones will depend primarily on the impact energy and the ability of the stones to resist this impact energy.

The impact energy is equal to $E= 0.5K_i m V_s^2$, where m is mass of stone, V_s is the velocity of the stone and K_i is an impact factor varying between 0 and 1.3 (see below). Statistical information on the velocities of the stones is given in Chapter 4.4 from which the probability density function, $p_S(E)$, for the impact energies can be obtained.
The ability of the stone to resist breaking is dependent on the stresses induced in the stone during impact, the mechanical strength of the solid stone and the number of fissures in the stone. The fissures may be "natural" fissures or fissures imposed during blasting and handling of the stone.

Tørum and Krogh (2000) considered use of Hertz's method and/or the Finite Element Method to calculate the stresses in a stone. A primary reason for not using such methods was that stones with internal fissures would experience a different stress distribution than homogenous stones, which is normally assumed for stress calculations. It was decided to make drop test experiments on stones in two quarries at Hasvik and Årviksand, Norway, following a method developed by Krogh (1980a, 1980b, 1999). It was thus possible to obtain statistical information on the breaking energies for stones of approximately the same size and for varying sizes. The probability density function for the breaking strength of the stones with respect to impacts, $p_R(P)$, was then obtained. Finally one can calculate the failure probability, P_f, according to the general concept of failure probability when considering two variables, in this case the impact energy, E, and the breaking strength, P, of stones. The probability of failure is then given by, e.g. Melchers (1987):

$$P_f = \int_0^\infty F_R(x)\, f_s(x)dx \qquad (6.1)$$

where

F_R = cumulative distribution function of the resistance (necessary energy to break the stones)
f_s = probability density function of the impact energy available to break the stones
x = parameter, in this case energy.

In addition to the drop test results, information was obtained on abrasion, the Youngs modulus, uni-axial compressive strength, brittleness and flakiness of stones from the quarries where the stone investigation was carried out. The results from the latter investigations are difficult to link directly to the conditions on a reshaping berm breakwater due to scaling problems.

6.2.2 Drop tests on quarried stones

The drop tests were carried out in two quarries, at Hasvik and Årviksand, where armouring stone for the construction of local breakwaters was produced. The rock type at Hasvik is a gabbro with an average density of 3050 kg/m^3, while the rock at Årviksand is a garnet rich gneiss with a density of 2980 kg/m^3.

There are two basic alternatives for the testing: 1) to drop the stones onto the ground and 2) to drop steel weights onto the stones lying on the ground. For alternative 1 the drop height will be the distance from the lower side of the stone to the ground, and for alternative 2 from the underside of the weight to the top edge of the stone.

Both alternatives are acceptable. However, if a constant drop energy - which is the product of the stone weight and the drop height and the acceleration of gravity - is required, the drop height has to be set according to the weight of the stone in alternative 1, which will differ within certain limits. Alternative 2 gives constant drop energy providing the drop height is constant. Since constant drop energy is preferable, alternative 2 is the easiest to perform. In addition, it is always difficult to hold and release unevenly shaped stones compared to steel weights easy to hook up. For these reasons alternative 2 was chosen for the drop tests. Figure 6.1 shows a sketch of the test set-up.

Because the drop weight has a diameter close to the size of the stone, the weight will always hit the highest point on the stone. Similarly, when two stones hit each other they will usually touch at one point with a force directed according to the movement of the stone.

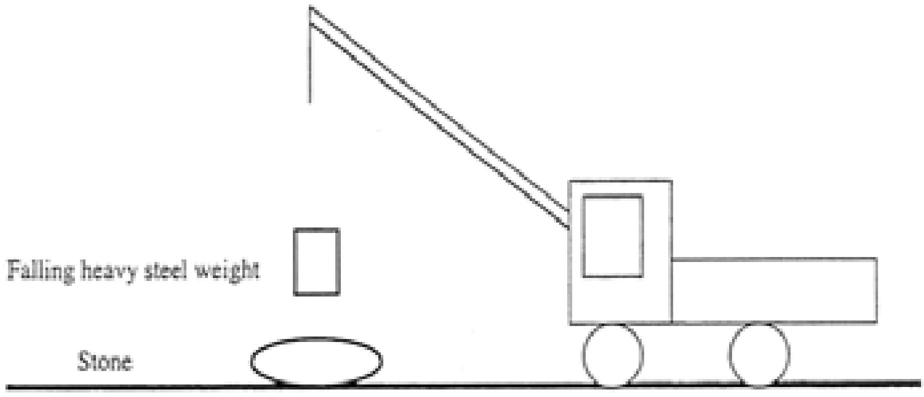


Figure 6.1. Sketch of drop test set-up.

To manipulate the drop energy for the different stone sizes, three steel weights were used: 51.5, 303.5 and 1709 kg, respectively. Throughout the tests, these weights made it possible to limit the drop height from 0.2 to 5.0 meters. Prior work has shown that the speed of the weight at the moment of touching the stone does not influence on the result of the drop tests as long as the drop energy is kept constant. The weights were hooked up in a release mechanism fastened to a strap on the beam of the excavator. The release mechanism was activated by pulling a rope fastened to it.

A site with solid rock was selected for the drop tests. The surface should be as even as possible where the stones are to be tested. Such a surface will give by far the most reproducible conditions when moving from one site to another. Piling up big stones as a base for the testing, as required for the CIRIA-CUR tests, CIRIA/CUR (1991), cannot be easily reproduced when going from one quarry to another.

The drop test is a statistical procedure, which means that a number of stones have to be tested before having a reliable average value. Three stone classes were tested. The approximate weights of the stones in each class were 20, 200 and 1400 kg. The range of the stone weights in each class was the mean weight ± 20 %. In this procedure, 12 to 25 stones were tested for each set of variables: stone size and drop energy. It could be objected that this was too low a number, but since each stone size was tested at different energy levels, the total number of stones within each size evened out the results statistically. A higher number of stones would have lengthened the test period in the quarry.

During the tests each stone was subjected to only one impact, even though it is recognised that a stone, not breaking after one impact, may break after a repeated number of impacts. Because of this, the test conditions resemble the conditions a stone on a reshaped static stable berm breakwater will experience. The stones move down the slope only once and will frequently experience only one major impact when it hits another stone during maximum velocity. On a reshaped dynamic stable berm breakwater a stone may be subjected to repeated number of impacts as it moves up and down the breakwater slope. Hence the results obtained with the drop tests method discussed here may be less valid for a reshaped dynamic stable berm breakwater than for a reshaped static stable berm breakwater.

The drop test gives values for the applied drop energy (P [Joule]) and the corresponding breaking frequency or probability of being broken, F, for each stone size. These values are plotted in Figure 6.2 for both the Hasvik and Årviksand quarries. Each curve represents the results of the tests on the three different stone classes from each quarry. Each point in the diagram represents the percentage of broken blocks for a particular drop energy. As mentioned, each point represents 12 to 25 stones. For example, 16 out of 23 stones or 69.5 % of the 200 kg stones at Årviksand quarry were broken for a drop energy of 10.120 Joule, while only 4 out of 23 stones, or 17.3 %, were broken for a drop energy of 2.980 Joule.

Krogh (1980a) found the following relation between the breaking energy, P , the energy to break 50 % of the stones, P_{50}, and the probability F of being broken:

$$P/P_{50} = \exp((F-0.5)/\beta) \qquad (6.2)$$

where
β = coefficient obtained from the experiments. The following relations were found for the drop tests at the two quarries in Hasvik and Årviksand

Hasvik: $\beta = 0.44$, $P/P_{50} = \exp((F-0.5)/0.44)$

Årviksand: $\beta = 0.50$, $P/P_{50} = \exp((F-0.5)/0.50)$

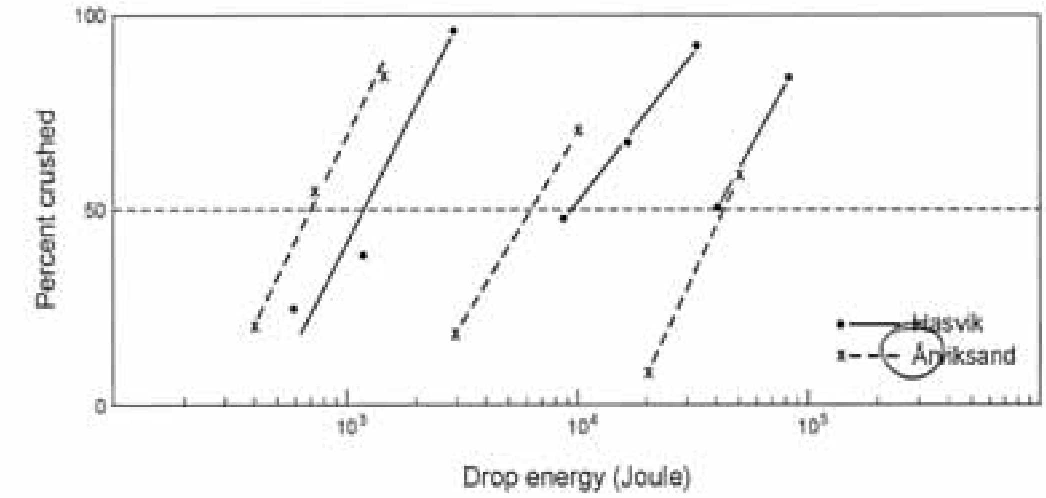


Figure 6.2. The probability distributions of strength for different stone sizes at the Hasvik and Årviksand quarries.

The P_{50}-values calculated from the curves in Figure 6.2 have been plotted in Figure 6.3 as a function of the stone volume. The volumes are derived from dividing the weights by the stone density. In the log-log diagram the plots define straight lines that can be expressed by the formula:

$$P_{50} = k \cdot V^{\alpha} \tag{6.3}$$

where
k = coefficient
V = the volume of the block
α = coefficient

From the drop test results the following coefficient values were obtained:

Hasvik: $\alpha = 0.842$, $k = 81070$, $P_{50} = 81070 \cdot V^{0.842}$

(V in m^3, P_{50} in Joule)

Årviksand: $\alpha = 0.970$, $k = 98310$, $P_{50} = 98310 \cdot V^{0.970}$

The values α, β and k were determined for the armour stones at Hasvik and Årviksand from the drop tests performed. These values give the basic breaking properties of the rock in the two quarries. They define the spread in strength for each stone size and the absolute strength as a function of stone volume.

By extrapolating from measured values, the energy necessary to break 100 % of the stones of a certain size can be determined. The maximum energy that will not break any of the stones of the same size can be further determined. The value P_{50} could be extrapolated to larger stone volumes than those tested. However, all extrapolations should be done with caution.

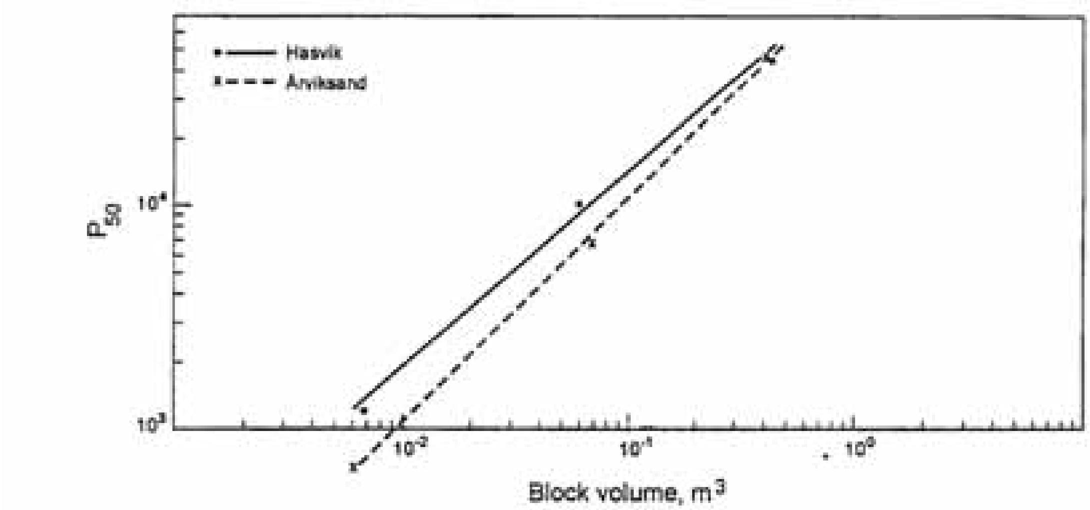


Figure 6.3. The breaking strength P_{50} (Joule) vs. stone volume (m^3) for the Årviksand and Hasvik quarries.

All such extrapolations are risky, and one must always question the validity of extrapolation. If the testing had stopped at 200 kg stones, one might have objected to the extrapolation beyond that weight. However, stones of 1400 kg were also tested and it was found that one could have accurately extrapolated from the 200 kg and less weights to the 1400 kg values. Therefore it is believed that one can extrapolate significantly beyond the breaking strength for the stone weight of 1400 kg.

The tests performed give the basic strength properties of armouring stone. However, the test procedure requires three to five days of work depending on the type of equipment available and the number of stones to be tested. Therefore, it cannot be considered to be a normal procedure for repeated analysis. However, it might be developed into a standardised method to certify armour stone from different quarries.

6.2.3 Combining results of stone velocity results and drop test results

When either the stone hits the "ground" or the heavy drop steel weight hits the stone during the drop tests, it will "stop" almost immediately. The impact force is not known, but it may be assumed it is related to the kinetic energy of the stone just before it hits the ground (or the heavy steel weight just before it hits the stone). This energy is again equal to the potential energy of stone or the heavy steel weight before they are released.

When the stone rolls on a berm breakwater it will not necessarily lose its kinetic energy completely when it hits another stone. The stone may continue to roll with a new angular and transitional velocity. The impact force between the two stones is not known, but it may be assumed that it is related to the difference of the kinetic energy of the stone before and after the impact.

When a stone hits another stone the situation is schematically and simplified as shown in Figure 6.4.

With reference to Timoshenko and Young (1951) the velocity v just after impact for a sphere with radius r can be calculated. Hence the difference in the kinetic energy of the rolling sphere before and after the inelastic impact can be calculated as follows:

$$E_{dif} = \frac{7}{5}m\frac{1}{2}\left(v_c^{\,2} - v^2\right) = \frac{7}{10}m\left(v_c^{\,2} - \left(\frac{2}{7}\right)^2 v^2_c\left(1+\frac{5}{2}(1-\frac{h}{r})\right)^2\right) =$$

$$\frac{1}{2}mv_c^{\,2}\,\frac{7}{5}\left(1-\left(\frac{2}{7}\right)^2\left(1+\frac{5}{2}(1-\frac{h}{r})\right)^2\right) = \frac{1}{2}mv_c^{\,2}K_i$$

where
h = step height
r = radius of the sphere
K_i = impact factor

$$K_i = \frac{7}{5}\left(1-\left(\frac{2}{7}\right)^2\left(1+\frac{5}{2}(1-\frac{h}{r})\right)^2\right)$$

This difference in kinetic energy can now be considered as available for breaking of the stone. If it exceeds the energy needed to break the stone, the stone will break.

In the following, the probability of failure of the stones is evaluated by considering only the two variables 1) the impact energy and 2) the required breaking energy. Although both are dependent on the mass of the stone, the velocity and the strength of the stones are considered to be independent. Hence Eq. (6.1) is used to calculate the probability of failure when only two independent variables are considered.

The impact energy is given by:

$$E = \frac{1}{2}mK_iV_s^{\,2} = \frac{1}{2}mK_igH_s\left(\frac{V_s^{\,2}}{gH_s}\right) \tag{6.5}$$

The probability function of E for a given significant wave height H_s and a given impact factor K_i is then:

$$f(E) = \frac{f\left(\frac{V_s^{\,2}}{gH_s}\right)}{\frac{1}{2}mK_igH_s} \tag{6.6}$$

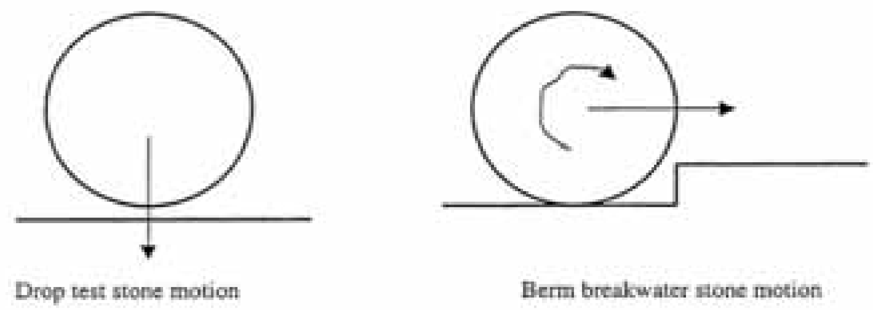


Figure 6.4 A rolling sphere hits a step.

The probability density function $f(V_s^2/gH_s)$ is given previously by a Weibull distribution function with a shape factor $\gamma = 0.877$ and a scaling factor $X_s = 0.237$ (see section 4.4).

We also need the cumulative probability distribution function for the breaking strength of the stone and, for curiosity, the probability density function $dF/d(P/P_{50})$ for the breaking energy. These are obtained as follows.

From Eq. (6.2) the cumulative distribution function is obtained:

$$F = \beta \ln\left(\frac{P}{P_{50}}\right) + 0.5 \tag{6.7}$$

and the probability density function:

$$f\left(\frac{P}{P_{50}}\right) = \frac{dF}{d\left(\frac{P}{P_{50}}\right)} = \frac{\beta}{\left(\frac{P}{P_{50}}\right)} \tag{6.8}$$

Figure 6.5 shows the cumulative probability distribution and the probability density function for the Årviksand and the Hasvik stone breaking data. In reality the probability distribution curves have S-shapes. But since no data for the upper (close to F = 1) and lower (close to F = 0) regions are available, no speculation on the real form of the curves in these regions is made. The "true" form of the curves in these regions will not significantly change the probability of failure results.

In further analysis the Årviksand stone breaking data, β=0.504, α = 0.966 and k = 98306, are used and extrapolated to larger stone sizes than applied during the field tests.

Figure 6.6 shows an example of the probability density functions for impact energy and strength "energy" of a stone for Hs =7.0 m, Ho =3.0, W_{50} = 8.000 kg, ρ_s = 2700 kg/m^3 and K_i = 1.4. F_R*f_s is also shown in the same diagram.

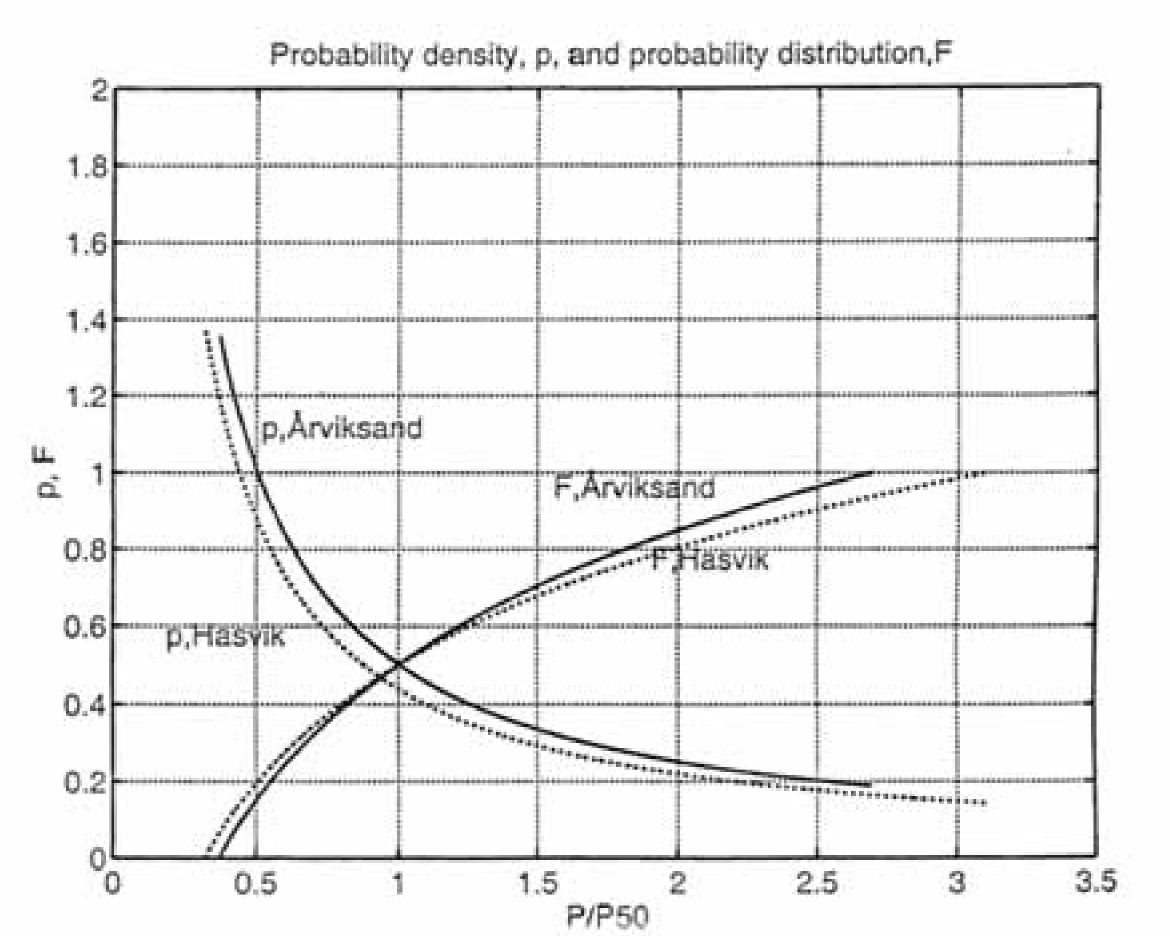


Figure 6.5. Cumulative probability distribution F and probability density function f for the Hasvik and Årviksand stone breaking data.

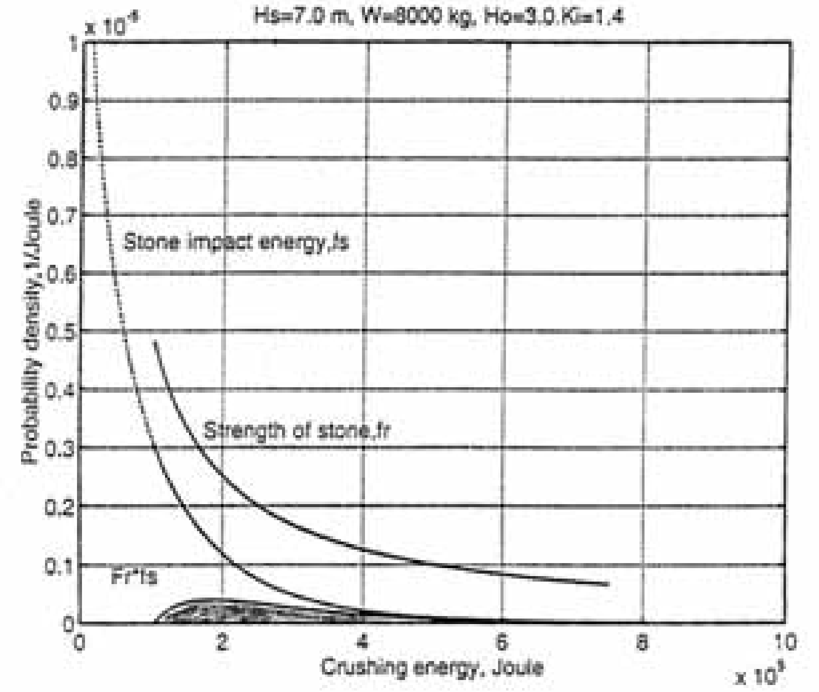


*Figure 6.6 Probability density function for f_s and breaking strength f_r of stone as well as F_r*f_s for W = 8000 kg, H_s = 7.0 m. H_o = 3.0 and K_I = 1.4.*

The probability of failure, P_f, for different conditions has been numerically calculated. Figure 6.7 shows as an example the probability of being broken for different stone weights in a stone class for W_{50} = 2.600 kg, H_s = 4.0 m, Ho = 2.5 and K_i = 1.4. The results show that the probability of being broken is virtually the same for all stone sizes in the segment.

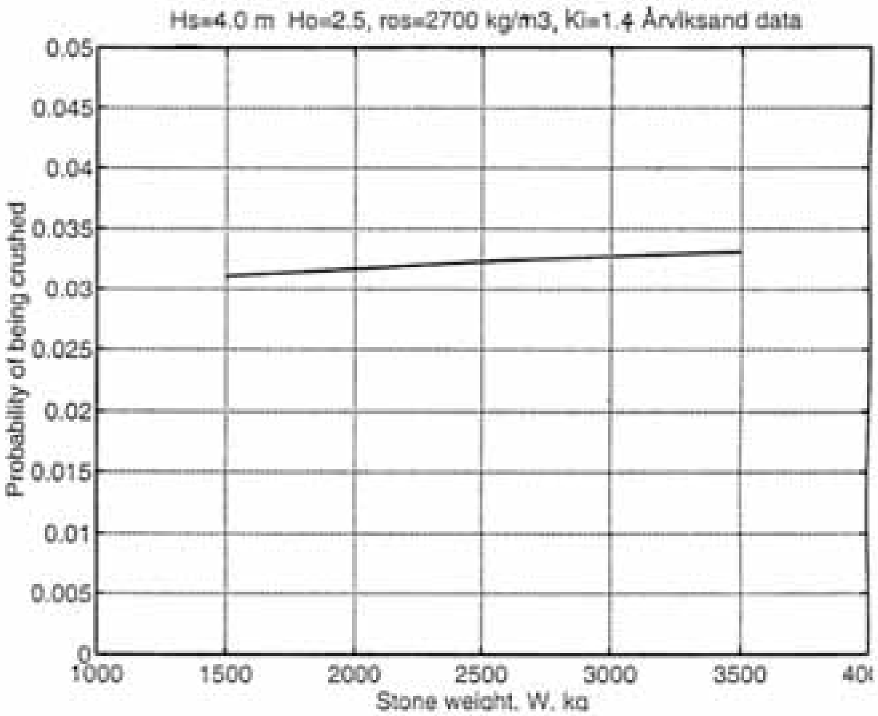


Figure 6.7. Probability of being broken for different stone size in a stone class. W_{50} = 2600 kg.

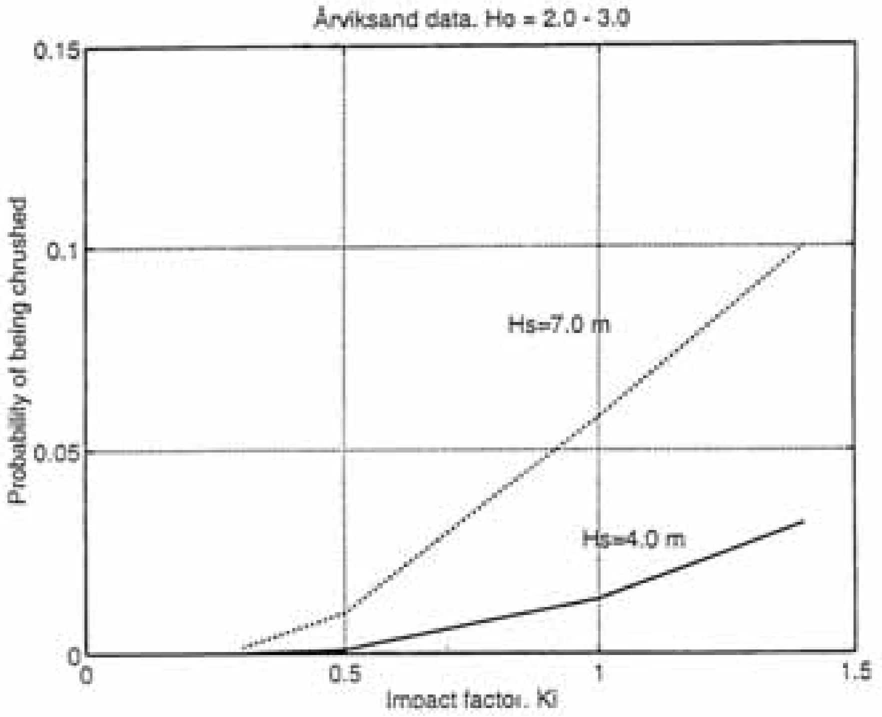


Figure 6.8. Probability of failure of stones (being broken) for different wave heights and different impact factors. Range of H_o = 2.0 – 3.0.

It can be concluded from these calculations that the probability of failure is the same for all stones for a given wave height H_s and a given K_i.

The probability of failure for significant wave heights H_s= 4.0 and 7.0 m and for different impact factors K_i have also been calculated. These probabilities are shown in Figure 6.8. It is seen that both the significant wave height and the impact factor have an influence on the probability of failure. The impact factor has been considered a fixed value, while in reality it is a statistical parameter with a mean value of approximately 0.7. So far the statistical variation of K_i has not been included.

The calculated probabilities of failure are based on statistical distributions of stone velocities obtained from samples from laboratory tests and field drop tests on samples of stones, coupled with an evaluation of impact energies "available" in inelastic impacts when a rolling sphere hits a step. How then should the calculated probabilities of failures be interpreted and what should be considered as an acceptable probability of failure?

As a first approach to interpreting the results, the probability of failure is considered as that fraction of stones that will be broken on a reshaped static reshaped berm breakwater where Ho<~2.7. If an average impact factor K_i = 0.7 is considered, Figure 5.7 indicates that approximately 3 % of the stones will be broken for H_s = 7.0 m and approximately 0.5 % will be broken for H_i = 4.0 m.

In general, failure criteria should be supported by the experience of the behaviour of a structure. No such experience on the failure of stones on a berm breakwater where the breakwater was systematically monitored, is available yet.

What are the consequences if small percentage of the stones on a berm breakwater will be broken?

Tørum and Krogh (2000) observed that a berm breakwater will reshape into a static stable berm breakwater when Ho< ~2.7. The stones will then move primarily downward on the berm slope into a resting position and will seldom move upward again. When Ho> ~2.7 the berm breakwater becomes reshaped dynamically stable, e.g. stones will roll up and down the slope while the profile shape remains stable. In the reshaped static stable conditions the stones will be subjected to only a few impacts, while in the reshaped dynamic stable condition a stone may be subjected to many repeated impacts. Abrasion, in addition to impact breaking, may also then play a more important role in the deterioration of the stones. Tørum and Krogh (2000) found, in considering the abrasion tests in a tumbling mill, that there is a scaling problem in relation to converting the tumbling mill abrasion results to full scale use for a reshaped dynamically stable berm breakwater. This scaling problem was also encountered by Mikos and Jaeggi (1995).

During the hydraulic tests it was also observed that there is a redistribution of stone sizes (without any breaking) with increasing wave heights, Tørum et al (1999). The larger stones travel down the slope and the smaller stones remain on the upper part of the slope. Hence it seems that one do not need to be afraid if some stones will be broken, even on the upper part of the slope, especially for the reshaped static stable berm breakwater. In fact it seems to be an anachronism to build a reshaping berm breakwater by building a berm that is allowed to reshape. During the reshaping process the larger stones will end at the footing of the berm, while they would have given better protection if they had been on the upper part of the berm. A more stable berm breakwater may have been obtained if the stones with the original mixture had been placed in a profile corresponding to the reshaped profile. This holds probably especially for a reshaped static stable profile. Economical construction methods to build the berm with a profile according to the reshaped profile should be developed.

It has to be born in mind that the breaking tests were carried out for stones with weights up to approximately 1400 kg. Although it is believed that the results can be extrapolated to larger stone sizes, it is recommended that drop tests be followed up with additional breaking tests, including smaller and larger size stones than used in this study. Similarly it will be useful if a berm breakwater could be monitored after construction to gain experience with regard to breaking and possible abrasion of the stones.

6.2.4 Conclusions of the breaking strength evaluation method

The main conclusions from the study on the breaking strength of quarried stone for berm breakwaters (Tørum and Krogh, 2000) are:

- The statistical distribution of the dimensionless velocity $V_s/(gH_s)^{0.5}$ is independent of Ho or HoTo.
- Previous methods developed for the statistical distribution of the breaking strength of smaller size sand/stone grains can be applied for breakwater stones.
- The statistical distribution of the dimensionless stone velocities and the statistical distribution of the breaking strength of stones can be combined into a probability of failure (breaking) analysis method.
- The probability of failure analysis method shows that stones of the quality found in most Norwegian quarries, e.g. the quarries in Hasvik and Årviksand, can be used for reshaped static stable berm breakwaters (Ho<~2.7) without an excessive number (<~5%) of stones being broken.

7. LIMIT STATE DESIGN ON HYDRAULIC STABILITY

Limit state design approach has not been customary in breakwater design. Tørum et al (1999) considered limit state design for berm breakwaters.

A general requirement for a statically stable berm without any reshaping appears rather rigorous, as a limited number of large storms can hardly develop a dangerous degradation of stones of sound rock during the lifetime of the structure. The limit state design is an attempt to establish a rational bridge between the three design philosophies - non-reshaped statically stable, reshaping statically stable or dynamically stable - by considering the actual block degradation resistance.

The Serviceability Limit States (SLS) specify general functional requirements for the breakwater with the profile as built and after reshaping. In addition, it prohibits stone motions under any but the most severe sea states.

Depending on the rock quality, limited breakwater reshaping is accepted for extreme sea states. The acceptability of the reshaping should be checked in the Ultimate Limit States (ULS).

The breakwater has to withstand accumulated reshaping and armour stone degradation effects for all large storms during its lifetime. This should be checked in the Fatigue Limit States (FLS).

Breakwaters are frequently designed without formal safety factors. To ensure the necessary safety and toughness margins, the structural integrity is controlled for worst credible sea state. This is done in the Accidental Limit States (ALS).

The proposed different limit states are as follows (Tørum et al, 1999):

Serviceability Limit States SLS.
To be checked for sea states occurring 50 times during the design lifetime.
Requirement: No significant motions of the stones due to the waves.
Requirement: No wave transformation through the breakwater.
Requirement: Overtopping depends on required conditions behind the breakwater.
The serviceability limit state must be controlled in the as built condition and after reshaping for the Ultimate Limit States condition (see below).

Ultimate Limit States ULS.
To be checked for a sea state with a 100 year recurrence period.
Requirement: It is acceptable for the berm to reshape. However, the residual berm width should not be less than $4xD_{n50}$. After reshaping the distance from the reshaped profile to the lower layer with smaller stones, possibly a filter layer, should be larger than 1.5 D_{n50} or at least 2 m. The armour stones should be able to withstand the reshaping without splitting, which reduces D_{n50} due to the motion of the stones.

Fatigue Limit States FLS.
Check for repeated sea states with a 10-year recurrence period after reshaping in ULS.
Requirement: No significant further reshaping.
No additional splitting and abrasion of the stones, which reduces D_{n50}.

Accidental Limit State.
Check for sea states with a 10.000 years recurrence period.
Requirement: The breakwater should remain intact.

These limit state design criteria have been applied for the design of a berm breakwater at Sirevåg, Norway.

8. SOIL STABILITY

Soil stability has to be considered for berm breakwaters as for any other type of breakwater, but soil mechanics is not dealt with in detail in this report. In addition to general consideration of soil mechanics, one may want to review De Rouck (1992), who deals specifically with the soil stability of breakwaters, including the soil stability of the breakwater itself. Generally speaking the bearing capacity of the soil is adequate without any special precaution taken for a "normal" firm sea bottom. However, test borings should be performed to make sure that the soil conditions are adequate or if some special measures have to be taken.

A berm breakwater has been built in Iceland on very weak soil (Sigurdarson et al 1999), which required special consideration for earthquakes and settlement. The foundation consisted of more than 20 m of thick soft organic silty soil. The breakwater was built using dredged basalt sand and shell fine sand and blasted material from land. Stability analysis showed that the breakwater had to be constructed in stages so the foundation could accommodate increased shear stresses through consolidation.

The influence of earthquakes on stability and settlement was evaluated on the basis of a selection of records of events with an average return period of about 50 years. These criteria yielded an earthquake of magnitude 6.0 – 6.3 on the Richter scale occurring at distances of 17 – 20 km. Seismic calculations were performed with the computer program SHAKE 91. The potential for liquefaction was assessed in accordance with Seed and DeAlba (1986) and with a similar method introduced by Ishira (1993). Based on these evaluations it was concluded that liquefaction of the foundation soil under the centre of the breakwater was unlikely. However, liquefaction is likely in an existing soil deposit near the front of the toe of the embankment. In final design the breakwater was strengthened by 5 meter extra berms on both sides of the breakwater to reduce damage in case of an earthquake.

Settlement was estimated, based on oedometer test results. A settlement of 2.0 m was estimated at the centre of the breakwater six months after the construction had finished with an additional 0.4 m settlement 20 years after construction had finished. Actual measurements indicated that the settlement was 1.3 m six months after the construction was finished.

9. FILTERS

9.1 Available filter criteria for coastal structures

The filter criteria, which are generally available for coastal structures, are summarized in Table 9.1 (Oumeraci, 1996). In addition, Rankilor (1981) recommended the empirical design curves shown in Fig. 9.1 from which the required grain size ratio D_{15}/d_{15} may be determined as a function of the uniformity coefficient of the base soil.

Most of the filter criteria presently used for designing coastal structures are essentially similar to those for steady flow; i.e. they are based on Terzaghi filter rules. However, it has been clearly shown (Sherard 1984) that these criteria are rather conservative when applied to steady flow situations. The grain size criterion $D_{15}/d_{15} < 4$ to 5 has been shown to be too conservative, resulting in a factor of safety of 2 according to extensive test results. However, a question does arise whether the safety factor of even 2 is acceptable as standard criteria for coastal structures. At present, this question is still open, despite extensive experimental work conducted in the Netherlands (De Graauw et al 1983 and Molen-Kamp et al 1979) and in the former USSR (Belyashevskii et al 1972) on filters subject to cyclic, turbulent flow conditions.

In view of the very complicated flow conditions and transport processes involved in the coastal environment, rational design rules for filter construction in coastal structures are only possible if based on investigation of the actual flow regime and transport mechanisms.

Another problem, which has not yet been seriously addressed in coastal structures, is filter thickness. Generally speaking, design criteria for filter thickness are not solely governed by hydro-geotechnical aspects but also by considerations of economy and practica-bility in the construction process. For instance, an overly thin filter layer may be very difficult or impossible to construct under water. On the other hand, sufficient thickness may reduce risks, which would result from possible segregation and/or settlement. Moreover, where suitable filter material is not available, thicker filter layers may be provided to relax grain size criteria. In addition, a thick filter layer also has a beneficial effect on the stability of the overlying armour units, because downrush velocities - which are responsible for most damages- decrease; i.e. more energy dissipation takes place within the thick filter layer.

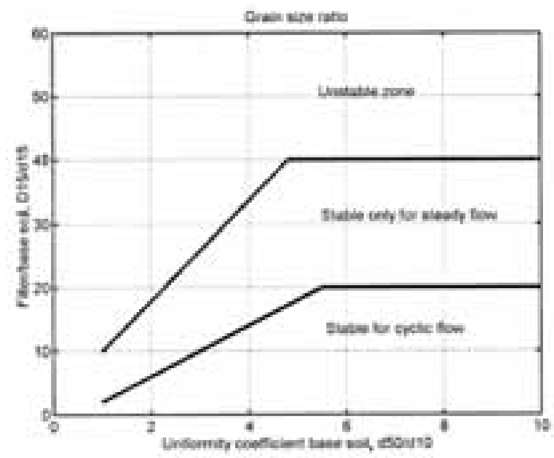


Figure 9.1. Filter design curves recommended by Rankilor (1981).

Table 9.1: Available filter criteria for coastal structures. (Oumeraci, 1996)

	Investigators	Filter criteria	Observations
1	BELYASHESV-SKII ET AL. (1972)	$D_{60}/D_{10} < 0.2\ D_{50}/d_{50}$	for graded rock filters
2	AHRENS (1975	$D_{15}/d_{15} < 4$	for rip-rap revetment underlayers
3	THOMPSON and SHUTTLER (1976)	$D_{15}/d_{85} < 4$ $D_{50}/d_{50} < 7$ $D_{15}/d_{15} < 7$	for rip-rap revetment funderlayers
4	DE GRAAUW et al. (1983)	$D_{50}/d_{50} < 2$ to 3	for granular filters under strong cyclic (reversing) flow
5	VAN ORSCHOT (1983)	$D_{50}/d_{50} < 3$ or $W^{*)}_{50\ (armour)}/W^{*)}_{50}$ (filter) < 25 to 30	for underlayers of rubble mound breakwaters*) W_{50} = Average Stone Weight
		$D_{(armour)}/d_{(under)} < 2.2$	for uniform armour units (breakwaters)
6	SPM (1984)	$D_{15}/d_{85} < 5.0$	for graded stone armour, filter blankets/bedding layers
7	ENGINEER MANUAL (1986)	$D_{15}/d_{85} < 4$ to $5 < D_{15}/d_{15}$	for graded rock filters soil
		$D_{15}/d_{15} < 4$	for underlayer of stone armour

From a hydro-geotechnical point of view, the filter thickness should satisfy two criteria: (1) accommodate the time dependent washing out of base soil particles into the filter matrix and (2) provide enough cross sectional area to allow the free outflow of water without excessive pressure build up. Thus, the attempts which have been made to investigate filter thickness, have been directed along two distinct parts:

(a) seepage analysis: the main objective of these investigations is to provide sufficient cross sectional area to allow the free outflow of water without excessive pressure. Given a filter material with a permeability k, the required thickness t_f is calculated according to the allowable discharge q and the allowable hydraulic gradi-ent i ($t_f = q/k \quad i$). This procedure, however, completely ignores the clogging of the filter by the migration of soil particles into the filter and its clogging (Cedergen, 1962).

(b) probability analysis: the random migration of washed out particles is analy-sed and the required filter thickness to arrest the washing out of base soil is obtained. Silveira (1965) has treated the problem of particles washing out as a stochastic process and has also developed an equation to compute the mean distance travelled by a soil particle before clogging, based on the absorbing state of Markov chain. Thanikachalam et al. (1975) have used queuing theory to describe the random motion of washed out particles and the clogging of filter pores and to formulate a theory for calculating the mean and standard devi-ation of the washed out particles. The filter thickness is then determined from the computed mean and standard deviation, the number of particles retained per pore opening and the average grain size of the filter. The results of Thanikachalam et al. (1975) appear to agree well with those of Silveira (1965). Moreover, they clearly show that increasing the filter thickness beyond a certain limit does not substantially increase the percentage of trapped particles.

The selection of a filter thickness satisfying both functionality (hindering washing out base soil particles and build up of excessive pore pressure) and constructionability (minimising segregation and settlement; precision of construction, etc.) still remains one of the most unresolved problems in the design of coastal structures. To date, no rational design criteria and or general empirical rules exist for the selection of filter thicknesses. Very often, the minimum filter thickness required is determined more by constructionability criteria than by any other criterion. In breakwaters the filter layer thickness should be 1.5 $D_{n50, armour}$ or at least 1.0 - 1.5 m. $D_{n50, armour}$ is the dimension of the armour layer stones or the stones in a subsequent layer in multilayer filters.

9.2 Concluding remarks on filters

The following summarizes the state of the art with regard to filters for berm breakwaters:

- Current filter criteria for steady flow are rather conservative, due to the lack of information on the flow and transport mechanisms involved;
- Conventional filter criteria for steady flow cannot be readily applied to cyclic flow conditions;
- Results erived from investigations on uniform filters normally conservative when applied to broadly graded filters. For instance, a well graded filter with $C_u = D_{60} / D_{10} = 20$ may catch soil particles of about half the size com-pared to a uniform filter with the same D_{15};
- Filter criteria expressed by grain size ratio related to the finer fractions of the filter material like D_{10} and D_{15} and the coarser fractions of the base material (d_{85}) are more reliable:
- Broadly graded filter are expected to constitute a more feasible alternative to conventional uniform filters;
- Substantial cost savings may be achieved by accounting for superimposed loads and for the transport mechanisms beyond the initiation of motion.

Development of reliable design rules can only be achieved by investigating the (1) mechanics of the initiation of motion and the (2) transport mechanisms after the initiation of motion.

Synthetic filter cloth has not been used for filter construction for berm breakwaters and will probably not be used in foreseeable future, except maybe as scour protection.

During construction of the St. George berm breakwater, Alaska, filter fabric was used to try to strengthen an experimental work pad section on an armour stone stockpile. The stone, however, was so large that the fabric could not support the weight of the pad material (shot rock) on top of it and was thoroughly torn up in the process.

10. SCOUR AND SCOUR PROTECTION

10.1 Scour

Scour can pose a significant threat to marine structures. Scour or erosion of fine materials from the base of the structure, can cause toe failure and a slumping of the structure's armour layer. With a berm breakwater, scour may occur along the seaward side of the trunk or around the head.

An EU reseach project on scour at coastal structures, SCARCOST, has been completed recently (Sumer et al 2000). During this project scour at piles and breakwaters was investigated along with the effect of wave induced pore pressures on scour.

Sour on conventional rubble mound breakwaters has been extensively investigated in small scale models by Fredsøe and Sumer (1997) and Sumer and Fredsøe (2000).
For berm breakwaters, scour and scour protection has not been investigated very extensively. Van der Meer and Veldman (1992) carried out two dimensional tests on a berm breakwater trunk at two different scales. Kuhnen (2000) investigated scour and scour protection around a berm breakwater head.

There is a problem with extrapolating results from small scale model scour tests due to scale effects. This is because the transport mode in the small models is most likely the bed load transport, while the transport mode in the coastal environment is frequently the suspension mode. The van der Meer and Veldman tests (1992) were apparently in the suspension mode, while the tests conducted by Fredsøe and Sumer (1997), Sumer and Fredsøe (2000) and Kuhnen (2000) were in the bed load mode. Figure 10.1 shows results of tests by Sumer and Fredsøe (2000), Kuhnen (2000) and van der Meer and Veldman (1992). S is the scour depth and d is the water depth.

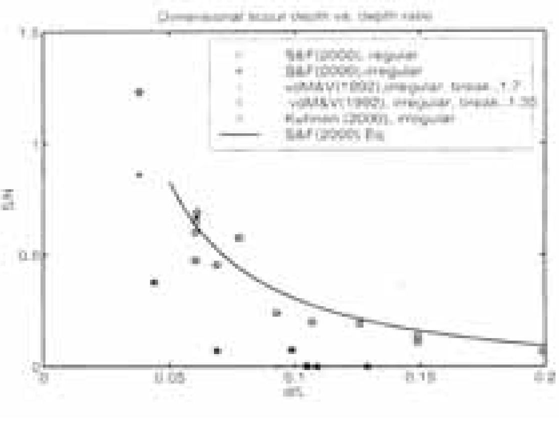


Figure 10.1 Scour at berm breakwaters compared with conventional rubble mound breakwaters.

10.2 Scour protection

Scour protection should most probably be provided at berm breakwaters if they are built on sand. Such a protective stone layer is illustrated in Figure 10.2.

There has been no systematic investigation into the requirements for the stone size, width, B_p, or thickness, t_p, for a scour protection for berm breakwaters. Some information is available on the stability of toe berms for conventional rubble mound breakwaters that might be useful for berm breakwaters also (Aminti and Lamberti 1996) and (Meulen et al 1996).

Aminti and Lamberti (1996) give the following relation:

$$\frac{H_s}{\Delta D_{n50}} = (1.1 + 33 s_m + 0.15 \frac{h_t}{D_{n50}}) N_{od}^{\ 0.2}$$

where
s_m = wave steepness, based on H_s and T_z.
h_t = toe depth, Figure 10.1
N_{od} = number of stones removed from the structure in a strip with a width of one D_{n50}

Van Meulen et al (1996) set the following criteria for N_{od} for a breakwater toe structure:

N_{od} = 0.5: Start of damage
N_{od} = 1.0: Acceptable damage
N_{od} = 4.0: Unacceptable damage

Kuhnen (2000) tested a two layer blanket as scour protection for a berm breakwater in 17.5 m water depth for H_s= 7.0 m and T_z = 10.6 s. The protection layer was 20 m wide with stones in the sieve diameter range d = 0.28–0.56 m (d_{mean} = 0.40 m). In breakwater design D_n = $(Q/r)^{0.333}$ is frequently used. Tvinnereim (1981) showed that there is a relation such that:

$D_n = c^{0.333} d$

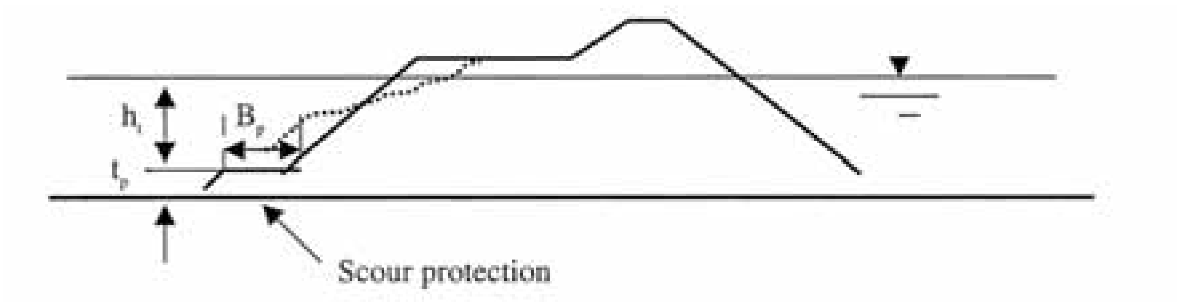


Figure 10.2. Scour protection.

For quarried rock of fairly uniform size Tvinnereim found c = 0.58, which gives D_n=0.83d. For the scour protection tested by Kuhnen (2000) D_{n50} is then approximately 0.34 m.

The scour protection tested by Kuhnen (2000) was acceptably stable. If we apply Aminti and Lamberti's relation (1996) the necessary D_{n50} = 0.34 m for N_{od} ≈2.3. This indicates that the Aminti and Lamberti relation can be used as a first approximation to obtain the necessary stone size for the scour protection layer. However it should again be stressed that there have been no systematic investigations into the required stone sizes for a scour protection layer for berm breakwaters.

There might be a need for a filter layer under the scour protection layer. Standard filter design criteria should be relevant for the design of filters for scour protection also. The width of the scour protection layer should be such that any scour outside the scour protection layer should not be harmful to the structure. A berm breakwater that has been designed to reshape is not as sensitive to scour as a conventional rubble mound breakwater. The tests of van der Meer and Veldman (1992) indicate that the berm breakwater section tested did not fail, even when there was a considerable scour hole in front of it.

11. CONSTRUCTION METHODS

One of the primary benefits of berm breakwaters when compared with conventional rubble mound breakwaters, is their greater acceptable tolerances. These tolerances relate both to the stone gradation and to placement accuracy. As a consequence, construction methods are generally simpler for berm breakwaters, offering substantial savings over the more rigorous approach normally adopted for conventional rubble mound breakwaters.

Because the success of the berm breakwater depends so much on the porosity of the structure, it is imperative to try to eliminate material smaller than the minimum required to meet gradation. This is difficult to achieve and some smaller material will almost inevitably enter the berm in the following ways (Gilman 2001):

- Breaking of armour stones when dropped
- The dispersal of small material when picked up along with larger armour stones for placement in the berm - through the use of tongs, orange peel, grapple or filter skip bucket
- Running heavy tracked equipment on the berm
- Deliberately pushing small material onto the berm in order to build pads for the equipment to work on the berm

The latter two are the most serious ways in which the berm is "contaminated" since they involve larger amounts of material than the first two ways listed. It is very difficult to extract or remove the work pad material. The small material falls into the voids of the berm up to two layers deep into the armour stone, from which elevation it is impossible to remove without removing the overlaying armour stones - in other words, without dismantling the breakwater. Further, once the breakwater has been subjected to a few storms with overtopping waves, the pad material is sufficiently spread about and jumbled as to render it almost impossible to remove.

The design must take this potential "contamination" into account and field inspections must stress the minimisation of "contamination" through proper education of construction personnel and monitoring of the work in progress.
A berm breakwater can be constructed using readily available and less specialised construction equipment and labour compared to the construction of a conventional rubble-mound breakwater.

The usual equipment consists of a drilling rig, two or more backhoe excavators, one or more front end loaders, and several trucks depending on the haul distance and size of the project. In addition, stones may be dumped from barges. In most cases split barges are used.

Backhoe excavators with open buckets or prongs, up to 110 tonnes, are used to place stones. In projects with maximum stone size up to 12 to 15 tonnes it is common to use backhoe excavators of 40 to 50 tonnes. At the Sirevåg breakwater, Norway, constructed during the years 2000 and 2001, a 110 ton backhoe was used to place stones up 20 tons down to –7.0 m water depth and up to 30 tons stones down to –1.0 m.

Large cranes have been used in some projects (Gilman 2001) but they are usually considered more expensive than backhoe excavators. The placing rate with cranes is much lower than with backhoes and the machine cost per hour is higher (Sigurdarson 1999). Cranes need a much finer and more stable work pad than a backhoe, which can crawl on uneven stone layer (Figure 11.1).

When the first berm breakwaters were built, bulldozers were used to push stones to the berm. This resulted in breakage of stones and dispersal of too many fines that plugged the voids.

The tolerances for placement of stones is greater for a berm breakwater than those for a conventional breakwater design and less strict placing techniques are needed. Usually no careful underwater placement is necessary. The front slope is steep and stones can be placed by backhoe excavators or cranes. Care has to be taken when placing the stones in such a way that they do not break due to

Figure 11.1. Construction of the Sirevåg berm breakwater, Norway, 2000 – 2001. The 110 ton excavator is placing Class II stones (10 – 20 tons). The vertical opening of the "claw" is 2.5 m and the weight of the stone in the "claw" is approximately 10 tons. Note that the excavator crawls on the uneven stone layer without any specially prepared work pad.

impacts. To what extent they should be dropped or thrown by the backhoe depends on the stone quality. Placement of stones, up to 5 tons, on a slope of 1:1.3 has been achieved down to 8 -10 m water depth with 40 to 50 tons backhoes. Experience from Iceland shows that small local contractors can quickly learn the necessary techniques to construct berm breakwaters successfully (Sigurdarson et al 1997). Each breakwater project is tendered out and even in a small market like Iceland there is competitive bidding for the works from up to ten contractors. The lowest bid is usually accepted.

The risk during construction is also much lower and repairs are also much easier for the berm breakwaters than for the conventional breakwaters.

Good interlocking of carefully placed stones may be advantageous for ensuring a long design life at the front and the edges of the berm.

Experience from many berm breakwater projects has shown that working with several stone classes and placement of stones only increases the construction cost insignificantly while leading to a better utilisation of the quarry material, thus lowering the total costs.

During construction each layer placement is controlled by soundings. Hence the construction of an apparently complicated multilayer berm breakwater such as the Sirevåg berm breakwater (Figure 4.2) can be well controlled.

The construction period for larger projects often extends over two years and experience has shown that partially completed berm breakwaters function well through winter storms. Repairs are much easier than for the conventional breakwaters.

Construction cost has been cut considerably in some recent projects by using dredged material, usually coarse sand and gravel, as a part of the inner core of the structure. Although the typical Icelandic berm breakwater is constructed of several layers, the advantage of using simple construction methods is still achieved. The advantages of sorting the stone mass into several stone classes to strengthen the structure are far greater than the disadvantages of the relatively low additional cost.

Continuous monitoring of the structure during its construction phase is a necessity. Sections have to be measured at the completion of each stone layer; stones have to be weighed, and visual control of the form and structure of the stone matrix has to be carried out. This is necessary to achieve the specified design and to make "as built" drawings as a reference point for further monitoring. It is also sometimes necessary to monitor the surrounding area during construction to ensure that every aspect is behaving as expected, such as siltation, scour etc.

12. PROBABILITY ANALYSIS ON HYDRAULIC STABILITY

Reliability analysis for berm breakwaters has not been carried out to the same extent as it has been for conventional rubble mound breakwaters, e.g. PIANC WG 12 (1992). For berm breakwaters a crucial factor is the recession of the berm. Tørum et al (1999) made an attempt to estimate the probability of not exceeding a certain recession based on the uncertainty of spread in model test results. Tørum (1998) analysed the dimensionless recession Rec/D_{n50} vs. the parameter HoTo for model test results at DHI and SINTEF. There were obvious differences in the results obtained in different test series the same laboratory and in different laboratories. However, Tørum (1998) was not able to discover why differences occurred. He therefore concluded that the results were of the same population with an inherent unknown scatter mechanism. By fitting a 3rd polynomial fit to the data he arrived at Eq. (6.27) for $d/D_{n50} = 25$ and fg =1.8. The coefficient of variation was $COV = \sigma_{HoTo}/Rec_{HoTo} = 0.337$, where Rec_{HoTo} is the mean recession for a specific HoTo value and σ_{HoTo} is the standard deviation of the data. Figure 12.1 shows the data points for the dimensionless recession Rec/D_{n50} vs. HoTo.

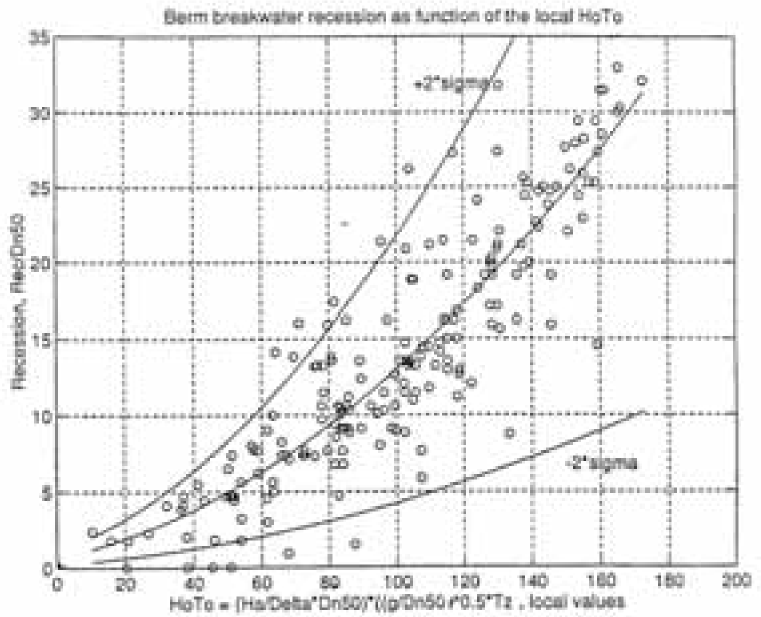


Figure 12.1. Dimensionless recession, Rec/D_{n50} vs HoTo. Tørum et al (1999).

The scatter of the data is defined by:

$$\frac{f - f_K}{f_K} = f\ (HoTo)$$

where
f = datapoint
f_k = mean value for a given HoTo-value
f(HoTo) 0 function of HoTo.

The scatter of the data is shown in Figure 12.2, while Figure 12.3 shows the standardised test data distribution compared to a standardised normal distribution.

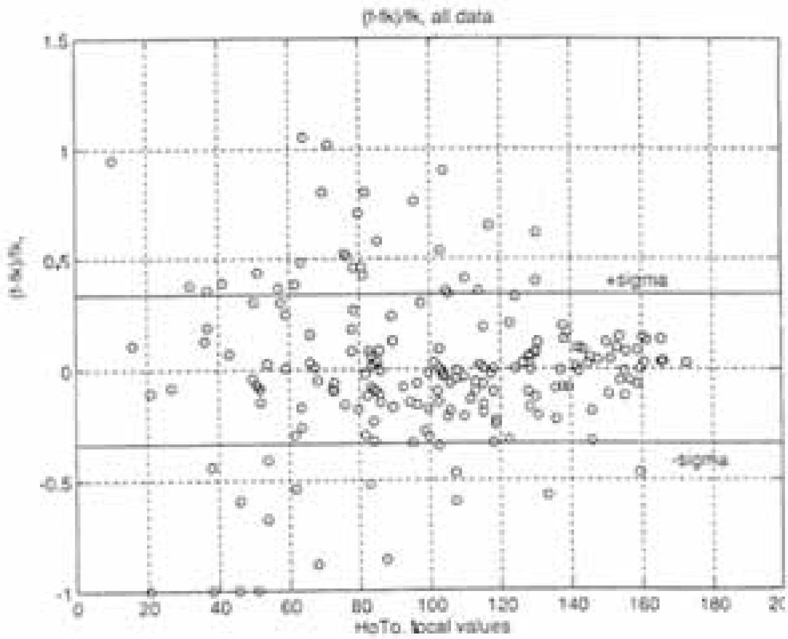


Figure 12.2. $(f\text{-}f_k)/f_k$ as a function of HoTo at the breakwater. Tørum (1998)

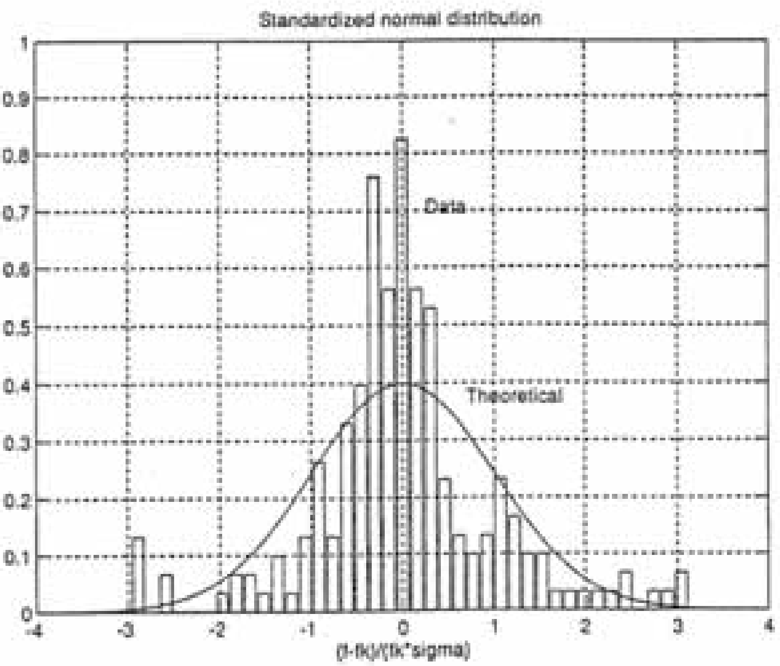


Figure 12.3. Standardised test data distribution compared to a standardised normal distribution.Tørum (1998).

There are apparently some "outliers" in the test data, but there is no information to justify throwing away the apparent "outliers". Although the data may not be normally distributed it is assumed for now that the data are following a normal distribution. The probability density function for the recession of the berm is then:

$$p(\mathrm{Rec}\,|\,HoTo) = \frac{1}{\sigma\sqrt{2}}\exp\left(-\frac{(\mathrm{Rec}-\mathrm{Rec}_{HoTo})^2}{2\sigma^2}\right), \tag{12.1}$$

As an empirical relation, based on analysis of many wave records, the relation between T_z and H_s is:

$T_z = 5.4\, H_s^{1/3}$

(T_z in seconds when H_s in meters)
and the following relation is obtained:

$$HoTo = \frac{5.4 H_s^{4/3}}{\Delta D_{n50}}\sqrt{\frac{g}{D_{n50}}}$$

The spread in the relation between H_s and T_z is neglected and the following relation is obtained:

$$p(\mathrm{Rec}\,|\,HoTo) = p(\mathrm{Rec}\,|\,H_s) = \frac{1}{\sigma\sqrt{2}}\exp\left(-\frac{(\mathrm{Rec}-\mathrm{Rec}_{H_s})^2}{2\sigma^2}\right)$$

Quite frequently the significant wave height probability density function follows a Weibull probability density function:

$$p(H_s) = \gamma\left(\frac{1}{H_c}\right)^{\gamma}(H_s - H_o)^{\gamma-1}\exp\left(-\left(\frac{H_s - H_0}{H_c}\right)^{\gamma}\right) \tag{12.3}$$

where
H_o = location factor
H_c = scale factor
γ = shape factor

The probability of not exceeding a certain recession R is then:

$$F(\mathrm{Rec} \le R) = \int_{H_s=0}^{H_s}\int_{0}^{R} p(H_s)\,p(\mathrm{Rec}\,|\,H_s)\,dH_s\,d\,\mathrm{Rec}$$

)

The uncertainty of the wave data is on the order of magnitude $\sigma_{Hs} = 0.08H_s$. This uncertainty is considered small compared to the uncertainty in the test results. Hence it seems justified as a first approach to neglect the spread in the significant wave height when calculating the probability of recession.

The preceeding analysis has been used in the early preliminary design of the Sirevåg berm breakwater, Norway. The final design is shown in Figure 4.3. At the time of the evaluation the wave parameters shown in Table 12.1 applied at the outer end of the breakwater. The Weibul parameters for the waves at Sirevåg were estimated to be: $H_o = 0$, $H_c = 1.5$ and $\gamma = 1.50$.

Table 12.1. Design waves at the outer end of the berm breakwater in Sirevåg. October 1998.

Return period Years	H_s m	T_p s	T_z s
1	4.35	12.1	9.0
10	5.60	13.1	9.7
100	6.60	13.9	10.4
1.000	7.70	14.5	10.8
10.000	8.80	15.5	11.6

The recession of the berm in the preliminary design of the Sirevåg berm breakwater was calculated with an equation similar to the Eq. (4.2), however without the gradation and depth factors. It should also be mentioned that the mean recession for the 10.000 year wave height is the mean recession for the 100-year wave height + 2σ, $Rec_{mean,10.000} = Rec_{mean,100} + 2\sigma_{Rec,mean,100}$.

Figures 12.4 shows the probability of exceeding the significant wave height and the probability of exceeding the mean recession. What Figure 12.4 primarily indicates is that the probability of exceeding the mean recession for a given significant wave height is much larger than the probability of exceeding the significant wave height itself. For example Figure 12.4 indicates that the mean recession for $H_{s,,10.000}$, $Rec_{mean,\ Hs=8.8} = 15.4$ m. The probability of exceeding $H_s = 8.8$ m is approximately $5x10^{-7}$. The probability of exceeding $Rec_{mean,\ Hs=8.8\ m} = 15.4$ m is approximately 10^{-5} or about 20 times higher than the probability of exceeding $H_{s,\ 10.000} = 8.8$ m. It can also be said that if the berm is made 15.4 m wide there will be the same probability of recession exceeding this width as the probability of exceeding the 1.000 year wave height $H_{s,1.000} = 7.6$ m.

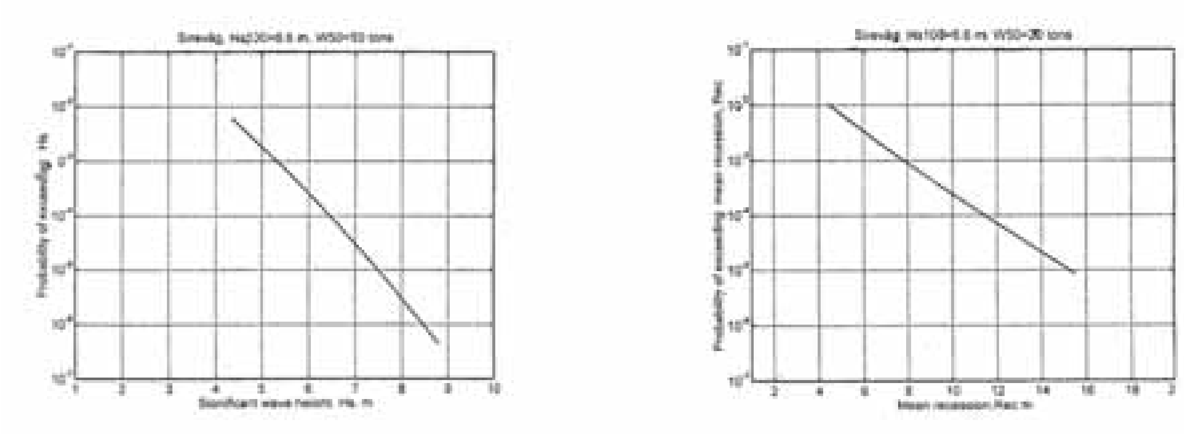
13. COSTS

Breakwater costs depend on local factors such as labour, availability of quarried stone, equipment and experience of contractors, transportation costs, environmental restrictions etc. It is thus difficult to state anything universal about the costs of berm breakwaters. The only "universal" statement that can be made is that life cycle cost comparisons should be made to help determine the choice between different breakwater concepts.

Sirevåg berm breakwater, Norway.

The cross section of the Sirevåg berm breakwater is shown in Figure 4.2. The breakwater is founded on dense sand and rock and there were no special soil stability issues.

The total volume of the Sirevåg berm breakwater is 642.800 m^3 (Sigurdarson et al 2001). The overall construction cost is 11 USD/m^3 or 12 EUR/m^3. On average the six contractors who bid the project priced stone Classes I and II (Figure 4.3) about 40 % higher than stone



Probability of exceeding of H_s Probability of exceeding of mean recession

Figure 12.4. Probability of exceeding of significant wave height and recession for the Sirevåg berm breakwater, Norway.

Classes III and IV, which again were priced 40 % higher than the quarry run material. As Classes I and II make up about 15 % of the total volume, the total price is not influenced greatly by the handling of the largest stones.

No detailed cost comparison with other structures was made for the Sirevåg berm breakwater. To make comparisons with other structures easier, the cost of the Sirevåg cross section designed for $H_s = 7.0$ m has been recalculated for a water depth of 20 m. At this depth the overall construction cost per unit length of the structure is about 17.000 USD/m or 18.000 EUR/m.

Lamma breakwater, Western Harbour, Hong Kong, China.

Ligteringen et al (1992) compared costs for different types of breakwaters for the Lamma breakwater, Western Harbour, Hong Kong. The 100-year wave height $H_s \approx 6.0$ m. Five types of breakwaters were considered: 1) Non overtopping conventional rubble mound breakwater with concrete units, 2) Berm breakwater, 3) Piled breakwater, 4) Caisson on low mound and 5) Skirted breakwater. The water depth was approximately 16 m. The soil conditions were poor.

The cost comparison showed that a berm breakwater with stone weight 0.5 – 4 tons rock was by far the cheapest solution. The value of Ho in this case is Ho = 3.7. Hence this berm breakwater will reshape into a reshaped dynamically stable berm breakwater for the 100-year design wave.

Extension of the berm breakwater at Hafnarfjördur, Iceland.

Sigurdarson et al (1999) give a relative cost comparison between a conventional and a berm breakwater for the extension of the breakwater at Hafnafjördur, Iceland. The breakwater is founded on weak soil and is designed for a 100-year wave of $H_s = 2.8$ m and $T_p = 12$ s. The cost comparison showed that the cost of the conventional rubble mound breakwater was 1.3 – 1.5 times the cost of a berm breakwater.

St. George berm breakwater, Alaska, USA.

The berm breakwater at St. George, Alaska is located in approximately 8 m maximum water depth. The design waves are depth limited. The cost of this breakwater has been estimated to approximately US$ 16.000 per linear m.

NOTATIONS

a = coefficient, used in different relations
b = coefficient, used in different relations
A = observed damaged area
B = length along breakwater trunk
B = berm width
B' = dimensionless berm width
B_p = width of scour protection layer
c = coefficient
C_D = drag coefficient
C_L = lift coefficient
C_k = characteristic coefficient
C_o, C_1 = coefficients in overtopping discharge formula
d = water depth
d_s = ater depth at toe of structure
D_n = $W/\rho_s)^{1/3}$
D_{n15} = 15 % of the stones have a smaller diameter than D_{n15}
D_{n50} = median diameter
D_{n85} = 85 % of the stones have a smaller diameter than D_{n85}
f_d = depth factor
f_g = gradation factor
$f_s(x)$ = probability density function of the impact energy
$f_r(x)$ = probability density function of resistance energy to break the stone
$F_R(x)$ = cumulative distribution function of the resistance energy to break the stone
g = acceleration of gravity
h = step height
h_f = depth of intersection point between original berm and reshaped berm
h_t = depth to scour protection layer
H = wave height
H_I = incident wave height
H_t = transmitted wave height
H_r = reflected wave height
H_s = ignificant wave height
H_{mo} = "significant" wave height based on spectral analysis
H_k = characteristic wave height
$H_{x\%}$ = x % of the waves are larger than $H_{x\%}$
Ho = stability number
$HoTo$ = wave period stability number
k = wave number
k = coefficient
K_I = impact factor
K_r = reflection coefficient
K_t = transmission coefficient
l_d = average length of displacement of a stone
L = wave length
L_o = deep water wave length
m = mass of stone
N_s ≡ Ho = stability number
$N_s{}^*$ = mobility index
$N_s{}^{**}$ = modified stability number
Nod = number of units displaced at least once in 1000 waves
P = impact energy to break a stone
P_{50} = impact energy to break 50 % of the stones
P_f = probability of failure
P_r = percentage of rounded stones in the berm
q = overtopping discharge
Q = dimensionless overtopping discharge
r = correlation coefficient
r = radius of a sphere
R_b = vertical run up on a slope with a berm
R_u = vertical run up height on a uniform slope
R_c = breakwater crest height
Rec = recession of berm
s_{mo} = wave steepness based on mean wave period
s_{mk} = haracteristic wave steepness
s_{op} = wave steepness based on peak period
S = longshore transport (number of stones pr. wave)
S = scour depth
S_s = damage level
t_p = thickness of scour protection layer
To = wave period parameter
T_z = mean zero up-crossing wave period
T_p = peak period
u^* = friction velocity
vc = translatory velocity of a rolling sphere before hittin a step
v = translatory velocity of a rolling sphere after hitting a step
V = volume
V_s = stone velocity
V_s = standard deviation of stone velocity
W_{50} = median stone weight
X = parameter in Weibull probability distribution
X_s = scaling factor

α = coefficient
β = slope angles
β = related to the ratio between static and dynamic friction
β_o = angle between wave direction and the normal to the longitudinal axis of the breakwater trunk
β_{kb} = characteristic angle between wave direction and the normal to the axis of the breakwater trunk at wave breaking
γ = shape factor in Weibull probability distribution
γ = coefficients, with index, in run-up and overtopping formulas
μ = friction factor
ρ_s = density of stone
ρ_w = density of water
υ = Shields parameter
ξ = Iribarren number
Δ = (ρ_s/ρ_w)-1, relative density

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