Appendix II: Berm Breakwaters

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1. Introduction

Berm breakwaters were introduced in the early 1980's. The design philosophy of berm breakwaters aims at optimising the structure not only with respect to wave load but also possible yield from an armourstone quarry, available equipment for construction and the function of the breakwater. 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 a S-shape profile, which was assumed to be a more stable profile and the structures are sometimes referred to as dynamically stable structures.

A modification of the original berm breakwater has been developed, which is a statically stable berm breakwater, Figure1. This structure is more stable than the original berm breakwater but at the same time less voluminous. This berm breakwater, sometimes referred as "Icelandic type", is build up of several size-graded layers in contrast to the original idea of two stone classes, stones and quarry run. An emphasis is put on maximising the outcome of the armourstone quarry and utilising this to the benefit of the design. The largest stone class is placed on the surface of the

Figure 1. The Sirevåg berm breakwater in Norway built as an "Icelandic type" statically stable non-reshaping structure.

berm to reinforce the structure. The goal in the design of the statically stable berm breakwater is to retain its integrity. Only some minor deformation of the berm is allowed under design conditions, but reshaping into an S-shape profile is prohibited. Still 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. The design approach is not necessarily to merely meet some certain prescribed stability number, $Ho=Hs/\Delta D_{n50}$, but also to correlate the size distribution of stones from the armourstone quarry, the quality of the rock and the wave characteristics at the site. It also takes into account the design 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. In many cases it has been possible to design a berm with high stability of the main armouring layer on top of and at the front of the berm at only marginal or no extra cost. Good interlocking by carefully placed stones in this armour layer is specified in the design.

Berm breakwaters have been designed and constructed in Iceland since 1983. Twenty-nine rubble mound structures of the berm type have been constructed so far, nineteen were new structures, whereas the remaining ten were improvements or repairs of existing breakwaters. Many of the structures have been exposed to design wave conditions with only minor profile changes. A considerable experience has been acquired through this period.

2. Development of berm breakwaters

Various types of rubble mound breakwaters can be termed berm breakwaters. Some of the names that have been used to describe these structures include: naturally armouring breakwaters, dynamically stable breakwaters, reshaping berm breakwaters, S-shape breakwaters, mass armoured breakwaters, statically stable berm breakwaters and multi layer berm breakwaters.

In the late 1970's and early 1980's many researchers and engineers were occupied with the idea of equilibrium slope and the importance of permeability (Bruun and Johannesson, 1976). Lessons were learned from 19th century breakwaters, like the breakwaters in Plymouth, England, and Cherbourg, France, which were built by dumping all quarried material at the breakwater site. The breakwater at Mangalore, India, was also taken as an example. It was built without the benefit of heavy handling equipment using smaller size rock in a rather wide berm. It was stated that when "maturing" these breakwaters might develop an S-shape. Furthermore alternative design were developed as for the Nome terminal in Alaska, where an S-profile was constructed to reduce stone size and crest height of the structure (Bruun, 1985).

In Australia the experience from the damage of the conventional rubble mound breakwater at Rosslyn Bay in 1976 introduced the idea of using commonly available rock sizes with the highest possible permeability. The concept of a mass armoured breakwater was defined as a rubble mound structure designed and built in an initially unstable form, but with sufficient material provided to allow natural forces to modify its shape to a stable profile (Bremner et al. 1987).

In Denmark an alternative design was developed for the Skopun breakwater. It consisted of a berm of narrowly graded stones, 10 to 15 tonnes, with a stability parameter of 2.6 (Jensen and Sorensen, 1987).

In the early 1980's the berm breakwater concept was introduced. For a wave protection of a runway extension in Unalaska, Alaska, Hall et al. (1983) proposed a wide berm of one stone class, where the armour system was designed so that essentially 100% of the quarry was utilised. The stability of the armour layer was to develop during early stages of wave attack. Model tests showed that the greater the thickness of the armour layer, the smaller the stones needed to be. The thickness of the armour layer for a specific breakwater was determined by the gradation of the available armourstones and the incident wave climate. The breakthrough for berm breakwaters was in 1983 when the design of the Helguvik breakwater was accepted for a tanker terminal close to the Keflavik NATO air base in Iceland (Baird and Woodrow, 1987). The design, consisting of two stone classes, a wide layer of 1.7 to 7 tonnes stones and quarry run, was developed for the location using the expected yield of a quarry to be opened close to the site. Baird and Hall (1984) described the basis of the design procedure as the optimisation of the use of locally available quarried material. They proposed lower limits for armourstone size corresponding to stability number between 4.0 and 4.5 and gradation D_{max}/D_{min} close to 1.7. Note that high Ho corresponds to low stability and low Ho to high stability. But at the same time they stated that the concept was intended to make use of the available stones and not necessarily the use of smaller stones than needed for conventional rubble mound breakwaters. Further they concluded that the conventional design with two layers of armourstones could be considered a special case of the more general concept.

Gradually the research and design of berm breakwaters developed more and more towards dynamic or reshaping breakwaters. Van der Meer and Pilarczyk (1986) classified berm breakwaters, or S-shape profiles, as having a stability number, Ho, between 3 and 6. It became the general idea that berm breakwaters were only applicable where large stones were of limited supply. These structures were built up of a homogeneous berm of relatively small stones with a wide size gradation.

A more stable design has been developed in Iceland in close cooperation between all parties involved; designers, geologists, supervisors, contractors and local governments. At the same time the designers have been directly involved in hydraulic model studies and supervision of the construction of the breakwaters. The berm is built up of several stone classes, each with narrow size gradation. Interlocking is specified on top of and at the front of the berm. In contrast to the classifications referred to above, the largest stone class or classes have stability numbers less than 2.7. Instead of looking at the berm as a mass of stones, the design focuses more on each unit as an element of a structure. No construction unit is as far from being standardised as the armourstone in the primary cover layer with regard to form, strength and durability (Viggosson, 1990), Figure 2. This development took place parallel to the development and research on reshaping berm breakwaters as the first berm breakwater in Iceland was constructed at the same time as the first reshaping berm breakwater in 1983 to 1984.

Figure 2. No construction unit is as far from being standardised as the armourstone in the primary cover layer with regard to form, strength and durability. Basalt stones in the Bolungarvik berm breakwater. A large part of the stones has some unopened cracks or geological flaws.

3. PIANC Working Group on Berm Breakwaters or Classification of berm breakwaters (which title is better?)

In 1998 PIANC decided to form a working group under MarCom to study different research results and compile all relevant information into practical guidelines for the design of berm breakwaters (PIANC, 2003). The working group gathered information on constructed berm breakwaters around the world, a process that has been continued by present authors, Table 1. Berm breakwaters may have many forerunners but here only structures built after the introduction of the berm concept (Hall et al. 1983) are listed.

Table 1. A list of constructed berm breakwaters

Country	Number of constructed BB	The year the construction of the first BB finished
Iceland	29	1984
Canada	5	1984
USA	4	1984
Australia	4	1986
Brazil	2	1990
Norway	6	1991
Faeroe Islands	1	1992
Iran	8	1996
Madeira	1	1996
China	1	1999
India	1	2003
Denmark	1	2003
Total number	63	

The Icelandic berm breakwaters constitute nearly half of the constructed berm breakwaters in the world with continuous construction of structures since 1983. The stability parameter for the largest stone classes that are used on top of and at the front of the berm are in the range of 1.5 to 2.8, while the overall stability parameter for the berm is higher. The gradation, D_{max}/D_{min}, for the Icelandic berm breakwater is mainly in the range 1.14 to 1.5 for the larger stone classes and up to 2.0 for the smaller classes.

The berm breakwaters in Canada, USA and Australia were mainly built in the eighties, mostly designed as the original berm structures of two stone classes, stones and quarry run for core. The stone classes for most of these structures have a stability parameter between 3 and 5 and a wide gradation in the range of 2 to 6 and even higher in some cases (Hall, 1987).

It is interesting to note how many berm breakwaters have been constructed in Iran in the few years since they started using these structures (Chegini et al. 2000). The flexibility the berm design offers has been used to construct breakwaters with quarried stones from sand and limestone of low specific gravity and poor quarry yield. The protecting berm on half of the Iranian berm breakwaters is built up stones with stability parameter numbers between 3 and 4, and the other half between 2 and 3. The gradation of the stone classes is rather narrow, mostly between 1.1 and 1.8. In Norway four new berm breakwaters have been constructed during the last several years. This could indicate that when designers and authorities have overcome their scepticism toward berm breakwaters and realised the economy of these structures, they find more and more use for the concept.

While a conventional rubble mound breakwater is required to be almost statically stable for the design wave conditions, berm breakwaters have different stability criteria. Traditionally berm breakwaters have been allowed to reshape, to a reshaped static stable or a reshaped dynamically stable profile, while only few stones are allowed to move on the “Icelandic type” berm breakwater. Berm breakwaters may be divided into three categories (PIANC, 2003):

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

Many of the Icelandic berm breakwaters fall into the category of statically stable non-reshaping structures, with none or only minor profile changes. These are structures like the Bolungarvik berm breakwater in Iceland built in 1993. In 1995 it was exposed to a design storm lasting for two days. Monitoring of the breakwater showed that only few stones had moved from the edge of the berm in two places, Figure 3. The Sirevåg breakwater in Norway falls into the same category, with only a few stones on the entire structure having been moved from their original position after a storm exceeding the design conditions.

Figure 3. The statically stable non-reshaping Bolungarvik berm breakwater after having experienced design storm lasting for at least two days in January 1995.

Some of the early berm structures in Iceland are examples of statically stable reshaped structures. A part of the berm is eroded but the structure functions very well and there is no need to add more material to the profile.

The St. George berm breakwaters in the Pribilof Island Chain of Alaska’s Bering Sea are examples of dynamic stable reshaped structures (Gilman, 2002). They were built during the period 1984-1987. Although these breakwaters are frequently exposed to the design wave conditions they are functioning well. In the first years individual stones did move on the slope but are now more locked. The advanced profile development has been due to gradual settling of the entire mass as the toe has slowly eroded over the years. Still, recent inspections have shown that the profile has reshaped significantly, and the berm on the South Breakwater will need to be restored in the next few years.

The Bakkafjordur breakwater in Iceland is, on the other hand, an example of a breakwater that became dynamic unstable reshaped structure with reshaping reaching the centreline of the breakwater crest (Einarsson et al, 2002). The berm breakwater was built in 1983-1984 partly of 0.5 to 6 tonnes stones and partly of 2 to 6 tonnes stones of rather poor quality quarried at the breakwater site. Deterioration of stones on the berm accelerated the dynamic development of the profile. After a storm in 1992 the breakwater was heavily reshaped and repaired the year after by stones from the same quarry as before. In 1995 the breakwater was exposed to wave conditions in 1995 close to the design wave height of Hs = 4. 8 m with Tp = 12 s. The berm was eroded up the crest and an unstable S-profile had developed, Figure 4. Video recordings show waves breaking in front of and on the breakwater. Inspection of the reshaped profile showed that deterioration of the stones had caused plugging of the voids and the structure did not function as a berm breakwater, resulting in higher forces on the slope and high wave overtopping. The year after the breakwater was repaired with two layers of stones of 4 to 10 tonnes with a slope of 1 to 3 on the top of the reshaped profile and with about 3 stones above 10 tonnes in front of the two layers from elevation –2 to + 2 m. These stones were taken from another quarry further away and were of much better quality than the original berm.

Figure 4. The dynamically unstable Bakkafjordur breakwater after the storm of October 1995 with erosion up to the crest.

The Racine Harbour North breakwater on Lake Michigan, Canada, is also an example of a reshaped berm structure (Montgomery et al, 1987). It was both extended with a berm structure to give additional shelter and the original caisson structure was improved with regard to wave overtopping by the addition of a quarried stone structure on the harbour side. The design wave conditions included 4.5 m high significant wave height from northeast. The breakwater was built in 1986-1987 of quarried limestone of 140 kg to 3.6 tonnes. During the first winter the breakwater was exposed to wave conditions similar to the design wave conditions, but exceeding the design water levels, which resulted in significant reshaping of the berm of the North breakwater extension. Due to higher water levels the berm elevation was raised by approximately 2 feet by adding 1500 tonnes of stone on the front slope of the breakwater. During the first five years the breakwater was frequently exposed to severe wave conditions resulting in excessive reshaping of the berm profile. After a series of model tests in 1991 the stability of the breakwater was improved by placing 6 to 12 tonnes armourstones on the crest of the breakwater. It is interesting that these improvements are very similar to the independent improvements of the Bakkafjordur breakwater.

The Mortavika berm breakwater in Norway protecting a ferryboat harbour is another example of a reshaped berm structure. Constructed in 1991 to 1992 of tonalitic gneiss stones quarried at the construction site. The design wave height was estimated Hs = 5.7 m with Tp = 13.7 s, but raised to 6.5 m for safety reasons . The design is a homogeneous berm structure, two cross sections along the length of the breakwater one built of stones with a mean weight of 8 tonnes and the other of stones with a mean weight of 5.5 tonnes. In the first winter the structure experienced a major storm. The wave height during the storm has been estimated to be in the range 4.6 to 5.8 m. About 100 m of the trunk section of the breakwater reshaped during the storm, with a recession of about 13 m. After the storm the breakwater was prepared and approximately 10,000 m3 of rock were filled into the reshaped structure.

4. Design considerations

The aim of the design of a berm breakwater is to construct a berm with high wave energy absorption, to minimise wave reflection from the trunk and especially from the breakwater head for navigational reasons and to minimise wave overtopping during its lifetime. To fulfil these criteria the berm has to remain stable. Therefore the berm of the Icelandic type berm breakwater is made of narrowly graded stones in several classes with armour cover made of the largest possible stones available from the selected quarries, Figure 5. The void volume of the berm is large with porosity of 35–40%. The wave energy is dissipated in the berm and the bulk flow velocity and wave forces are lower. As the berm is statically stable the abrasion and breaking of the stones due to movement is minimised thus giving the structure a longer service life. This means that the idea of a dynamically stable structure is abandoned in favour of the stable Icelandic-type berm breakwater (Sigurdarson et al. 1998a).

Figure 5. Narrowly graded stones on the Husavik berm breakwater. Class II, 10 to 16 tonnes, in the foreground and class I, 16 to 30 tonnes, further away.

Archetti et al. (2002) have applied the abrasion model of Lamberti and Tomasicchio (1994) on six Icelandic breakwaters to estimate the loss of stone volume due to abrasion. The results from the model show that the volume reduction due to abrasion because of stone movements is only of the order of 1% and therefore not significant. Reduction of stone volume on a reshaping breakwater is on the other hand mainly due to breaking of stones. Tørum et al. (2002) have introduced a method for evaluating the suitability of quarried stone to withstand the impacts they will be subjected to when rolling on a reshaping berm breakwater. The purpose is to estimate the suitability of specific quarries for reshaped statically stable berm breakwaters.

The necessity to minimise wave overtopping in Icelandic fishing harbours is high as the berths are often located just behind the breakwater. Several breakwaters with berth structure on their inner side and suffering large wave overtopping have been protected by statically stable non-reshaping berm structures, which have proven to be very effective in reducing overtopping. Physical model tests have confirmed the advantages of the Icelandic berm breakwater compared to the conventional rubble mound structures (Sigurdarson et al. 1996). Experience with reshaped dynamic structures in Iceland has demonstrated that when stones start to roll up and down the slope and hit each other, breaking and splitting of stones will occur, followed by abrasion. Voids will be filled up with smaller stones and the ability of the structure to dissipate wave energy will decrease. Model tests, which compared wave overtopping and flow through the crest structure before and after reshaping, show an increase with increased reshaping (Jacobsen et at. 2002).

Recently a statically stable non-reshaping berm structure was chosen to protect the Hammerfest LNG plant in northern Norway partly due to low overtopping. The plant is located in sub-polar region where icing from accumulation of frozen sea spray may represent significant difficulties for the plant structures. Wave overtopping was therefore an important factor in the choice of a wave protecting structure.

The berm concept has been proven to successfully increase navigational safety for narrow entrances with heavy breaking waves due to decreased reflection, compared to conventional breakwaters. With higher efficiency in wave damping, the wave reflection from the berm breakwater is lower, but the interaction of incident and reflected waves, can yet cause dangerous waves, especially for small fishing vessels.

One berm breakwater of the Icelandic type has been constructed on a weak foundation, consisting of more than 20 m of soft soil (Sigurdarson et al. 1999). In spite of a total settlement of close to 4 m in some areas, about 2 m more than predicted, it was easy to adapt the berm design to this unstable situation during construction.

The design criteria for conventional rubble mound structures has evolved considerably over the past 30 years. In the seventies a 5% damage was allowed for a 25 to 50 year design return period, to the presently accepted 0 - 2% damage for a 100 year design return period. Structural failure is no longer accepted. The statically stable non-reshaping berm breakwater has been able to satisfy the more strict design criteria required of structures due to increased functional and other performance criteria they now must meet. This is due to our increased knowledge of design wave conditions, the strength and durability of rocks, possible quarry yields and improved construction methods.

The stability number of a conventional rubble mound breakwater is related to damage to the armour layer. Van der Meer (1988) defined the damage level, S, as the erosion area around still water level divided by the nominal diameter of the stones in second power, where S = 2-3 equals start of damage. Generally the actual number of stones eroded in a Dn50 wide strip is equal to 0.7 to 1 times the damage S. This means that start of damage equals erosion of about 2 stones in a given cross-section.

The stability number for the stable Icelandic- type berm breakwater is related to the start of damage or recession of the stones at the edge of the berm. The recession, Re, is the erosion of the stones from the edge or the crest of the berm. It is often used to describe the reshaping of berm breakwaters. The stability criterion for the Icelandic type of berm breakwater is that after the design storm the recession of the berm shall not exceed two stone diameters, Re/Dn50<2. On the other hand stability criterion for dynamic berm breakwaters is often defined so that the recession shall not exceed the total width of the berm, (van der Meer and Koster, 1988) and (Sayao, 1999).

The design criterion for the Icelandic type of berm breakwater has been developing over the past years. Three main parameters are recognised: the stability number of the stone class at the front and on the top of the berm (Ho), the effective width of the berm measured at design water level into the core of the structure (B), and the gradation of armourstone classes (fg). The first two parameters are interdependent, as with higher stability less berm width is needed (Sigurdarson et al. 1998b). The influence of the gradation of armourstones on the berm width has been described by Hall and Kao (1991) where narrow gradation leads to higher porosity and higher stability. Therefore the width of the berm can be reduced if narrowly graded stone classes are used. Jacobsen et al (1999) showed that a berm breakwater reduces the wave energy penetrating around the breakwater head and into the harbour more effectively than a conventional rubble mound breakwater of equal length.

Good interlocking of carefully placed stones at the front and at the edge of the berm is prescribed in the technical specification, which is a part of the design of the Icelandic-type berm breakwater. This is in contrast to the construction methods for dynamic berm breakwaters where armourstones are dumped but not placed. The importance of interlocking is well known from research on conventional breakwaters.

The present authors design many breakwaters each year and for low design wave height, Hs< 2.5 to 3 m, usually choose a conventional design, but for higher wave heights a berm breakwater is chosen. The availability of large rocks is examined with the aim of finding a quarry, which will provide over 15 to 20% of rock with a stability number, Ho, below 2.7.

Prototype experience gained through construction supervision, monitoring and inspection of berm breakwaters has been incorporated in the design. Throughout the lifetime of the structure visual observation and recording is the most efficient and economical monitoring method (Einarsson et al. 2002). To evaluate the functional criteria of the structure, observation during storm conditions is vitally important. Video recordings by local harbour authorities are used to document this observation.

5. Stability tests of Icelandic type or multilayer berm breakwaters

Most of the research on the stability and reshaping of berm breakwaters has been for original berm breakwater constructed of two stone classes, but much less on the more stable Icelandic type of structure or the multilayer berm breakwater.

The difference in reshaping of the original berm breakwater constructed of two stone classes and the more stable Icelandic type berm breakwater with the largest stones used as an armour layer was studied through physical model tests at the Danish Hydraulic Institute (Sigurdarson el al, 1998a) and (Juhl and Sloth, 1998). Comparison between the two types of structure showed a reduction in the erosion volume and recession of the berm for the Icelandic type berm breakwater. An armour layer protecting both the top and the front of the berm was found to be more effective than other alternatives. Another conclusion was that an increase in the berm freeboard was found to reduce the reshaping of the berm.

Tørum (1998) analysed the dimensionless recession Rec/D_{n50} as a function of HoTo for several scale models from different laboratories, DHI and SINTEF. The berms were built up of homogeneous stone class of the original berm type. The data was fitted with a second order polynomial and later a third order polynomial (Tørum et al. 1999). Still Tørum added terms to take into account the gradation of the stones and the water depth (Tørum and Sigurdarson, 2001):

$$\frac{\mathrm{Re}\,c}{D_{n50}} = 0.000027(HoTo)^3 + 0.000009(HoTo)^2 + 0.11(HoTo) - f(f_g) - f(\frac{d}{D_{n50}}) \quad (1)$$

where:

Rec is the recession of the berm due to reshaping, measured from the front edge of the berm towards the crest,

$HoTo = H_s/(((\rho_s/\rho_w)-1)D_{n50})((g/D_{n50})^{0.5})T_z$,

ρ_s = density of stone,

ρ_w = density of water,

g = acceleration of gravity,

T_z = mean period,

$f_g = D_{n85}/D_{n15}$, gradation factor.

The function for the gradation factor is given by:

$$f(f_g) = -9.9f_g^{\ 2} + 23.9f_g - 10.5 \quad (2)$$

and the depth function is given by:

$$f(d/D_{n50}) = -0.16(\frac{d}{D_{n50}}) + 4.0 \quad \text{for } 12.5<d/D_{n50}<25. \quad (3)$$

Tørum et al. (2001) carried out tests on a multilayer berm breakwater, basically on the Sirevåg berm breakwater, see section 8. The results have, however, been analysed in a general context. The stones in the berm were placed by dropping them from a set height. No effort was made to place them in an orderly manner, which is in contrast to the actual design, but it well known that orderly placement and interlocking increase the stability considerably.

The results from the stability tests with multilayer berm breakwater together with equation (1) is plotted in Figure 6 for different gradation, f_g. The left most line without depth correction, the middle line for f_g = 1.8 and the right most line for f_g = 1.1 to 1.2. Test set-up 1 data are the data for Sirevåg breakwater with stone density 2700 kg/m^3 and test set-up 2 is for stones with higher density, 3100 kg/m^3. For the multi layer berm D_{n50} for the largest stone class is used to calculate HoTo as well as Re/D_{n50}. The coefficient of variation, COV, for the dimensionless recession is about 0.35, Tørum (1998).

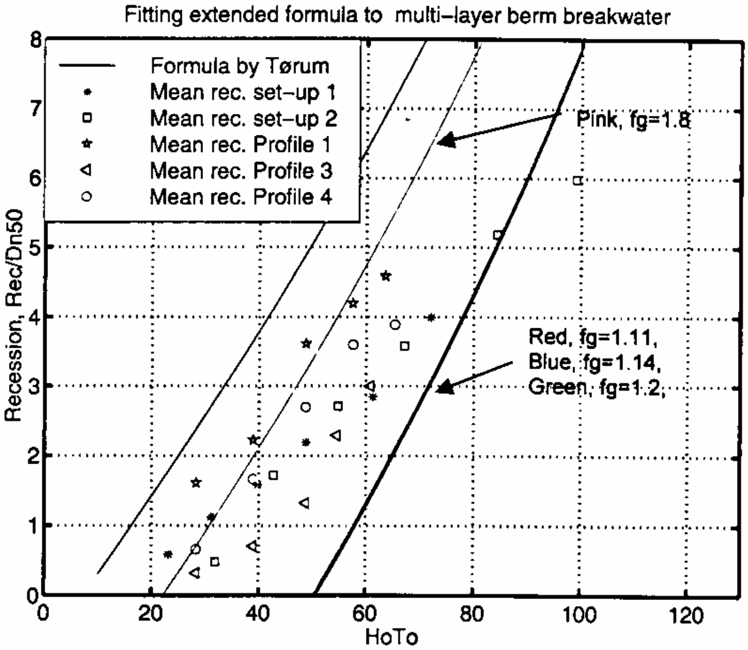

Figure 6. Dimensionless recession vs HoTo for homogenous berm breakwaters. Solid lines for different gradations fg, the leftmost line without gradation correction. Points refer to multilayer berm breakwaters.

6. Quarry yield prediction as a tool in breakwater design

Quarry yield prediction has played an important role in the design phase of harbour breakwater projects in Iceland since the early 1980's, (Smarason et al. 2000). It has proven to be a valuable part of the design process in preparation for successful breakwater projects. Preliminary designs are based on initial size distribution estimates from potential quarries, and the final design is tailored to fit the selected quarry. Quarry selection is a process which aims to provide rocks best suited to the wave conditions of the construction site and at the same time to minimise transport costs and environmental disturbance.

The importance of quarry yield prediction can best be described by a quotation attributed to O.J. Jensen (1984). "In many projects, in which DHI has been involved in recent years, the lack of knowledge of available stone sizes in the quarry has turned out to be decisive for the breakwater profile at a very late stage, namely after initiation of the construction work. In some cases it has been necessary to modify the profile to fit the actual stone classes available." And later "It is for the above reasons extremely important for a breakwater project that information on the specific quarry is available at an early stage."

Often the owner/designer has to rely on the contractor or quarry operator for information on the maximum quarry yield or the size of the largest stones obtainable from the quarry. These estimates are very often biased by the size of equipment the contractor/quarry operator has available.

Dedicated armourstone production is not common and therefore there are not many contractors who have much experience in this field. Guidelines for blasting for armour stones are insufficient and only a few contractors have much experience in drilling and blasting for breakwater construction. The present authors have been trying to change this situation and are gradually training contractors to work the quarries to requested specifications. Many contractors are now familiar with the quarry yield prediction and rely on them in their bids.

It has been demonstrated in many projects that although contractors complained at the beginning of the work that it would not be possible to obtain the predicted quarry yield, the yield prediction was, however, fulfilled in the end. This has often been achieved through small changes in the blasting design (i.e. tilt, burden and spacing of holes) and the amount of explosives used.

Furthermore, increased knowledge through quarry yield prediction and in the production of armourstone from various quarries has allowed the specification of large (10-20 tonnes) and extra large (20-35 tonnes) stones, typically to improve the stability of the berm. The percentage of large stones produced in the quarry can be as low as 2-5 % of the total quarried volume to be used as the largest stone class. Large hydraulic excavators and front loaders (75 to 110 tonnes) that can handle these large to extra large stones have become readily available. These large machines may raise the cost of the projects by 1-2%. Recent projects have utilised large to extra large stones to the advantage of the stability and strength of the berm structures. A relatively low percentage of these largest stone classes can be of great advantage for most breakwaters. This is not only valid for high to moderate wave conditions but also applies to lower wave load conditions where quarries with relatively low yield size distribution are used. For the same design wave condition and stability of the berm, the additional cost of the larger hydraulic excavator is compensated for by smaller berm width. Table 2 shows the results of a few quarry investigations where large and extra large stone have been required, (Smarason et al. 2000). In all cases the actual quarry yield has been pretty close to the prediction. Figure 7 shows the stock pile of 20 to 35 tonnes stones for the Hammerfest LNG terminal in northern Norway. To stabilise the tidal inlet of Hornafjordur three structures have been constructed and two different quarries have been opened, Figure 8. All structures have been designed with the aim of utilising all quarried material.

Table 2. Quarry yield prediction for some recent breakwater projects.

Breakwater site	Rock type	Predicted Quarry Yield >20 t	>10 t	>5 t	Volume (m^3)
Bolungarvik, I	porphyritic basalt	2	5	11	265,000
Blonduos, I	porphyritic basalt	4	9	14	100,000
Hammerfest*, N	gneiss	4,4	10	15	3,000,000
Hornafjördur S-barrier, I	porphyritic basalt	2-5	5-10	15-20	60,000
Hornafjördur E-Barrier, I	gabbro	5-10	10-15	15-20	100,000
Husavik, I	porphyritic basalt	3-4	7-10	12-16	300,000
Sirevåg, N	anorthosite gabbro	15-17	22-25	30-33	640,000
Vopnafjördur, I	porphyritic basalt	10-20	20-30	30-40	40,000

I stands for structures in Iceland and N for structures in Norway

*The figures for the Hammerfest breakwater represent actual quarry yield after 500,000 m^3 production

Figure 7. Stock pile of class I stones, 20 to 35 tonnes, in the Hammerfest LNG terminal, with a quarry yield of about 5%. During the construction period from autumn 2002 to spring 2003 2.3 million m^3 of rock was quarried.

Blast design is the most important factor for a successful breakwater project. It is the deciding factor in securing the desired fragmentation of the rock. It is absolutely vital that the blasting engineer is prepared to adjust his blasting pattern to suit each particular quarry and he may have to adjust his pattern several times within the same quarry to maximise his results. We usually find that a drill pattern with a 3” drill bit close to 3-4 m burden (b) and 2-2.5 m spacing (s) for a bench height of 9-12 m gives the best results in sound porphyritic basalt lavas. The ratio s/b should for best results lie between 0.6 and 0.7.

A new blasthole row should not be drilled until after the clearing of the bench face and quarry floor is completed. Only then can the blasting engineer decide on his drill pattern and tilt of holes. It is important that the holes be drilled parallel with a dip of 70-80°, for best results and minimum damage to the blasted rock. This causes minimum throw of the blasted rock as only the bottom part of the bench is thrown out and the upper part falls into the blasted pile. A low specific charge should be used, generally below 200 g per cubic metre of solid rock, depending on rock soundness and desired block size.

Production of large and extra large armourstone requires a coarser drill pattern than generally used in armourstone production. For optimum results it may be necessary to produce a significant amount of blocks that may be two to three times the largest desired armourstone for the project. These oversized blocks will have to be split afterwards, either by a steel ball, hydraulic hammer or by secondary blasting. It should be emphasised here that the size reduction of the largest block is the area where the contractor can make his biggest earnings on a breakwater project. An unprofessional approach to this part of the work can lead to considerable overproduction in the quarry, which should by no means be rewarded.

Contractors may in the past have been able to claim on quarries where limited preparation was carried out, as the owner had not got the means to prove that excess production could have been caused by mishandling of the quarry. Thorough quarry investigation and quality assurance programme have freed the owners from compensation to the contractors in this area. If, however, the quarry investigation has not been carried out in accordance with the recommendations, unforeseen defects have appeared in some quarries. This has led to overproduction as some of the substandard armourstones have been rejected and unforeseen fracture zones have also been encountered in some quarries.

The quality assurance programme presented by Smarason et al. (2000) aims at finding out the weaknesses of the quarried rock at an early stage. It is important to know the material and its properties, i.e. rock type, discontinuity spacing for quarry yield prediction, density and absorption, strength (point load index), freeze/thaw resistance (in cold climates), and resistance to abrasion in abrasive conditions. No test, however, can replace the personal visual inspection of the experienced engineer or geologist.

Figure 8. The stabilised Hornafjordur tidal inlet with the shore protection on the S-barrier in the foreground, built in 1991, the curved berm jetty in the centre of the photo, built in 1995, and the weir to the skerries to the right and above the jetty, built in 2000.

7. Construction of the breakwater

A berm breakwater can be constructed using readily available methods and less specialized construction equipment and labour compared to the construction of a conventional rubble-mound breakwater.

Usual equipment includes a drilling rig, two or more backhoe excavators, one or more front loaders, and some trucks depending on the hauling distance and size of the project. Backhoe excavators with open buckets or prong are used to place stones. In projects with maximum stone size up to 10 to 15 tonnes it is usual to use backhoe excavators of 40 to 50 tonnes, with maximum stone size up to 20 to 25 tonnes 70 to 80 tonnes backhoe excavators are used and where maximum stone size is 30 to 35 tonnes excavators up to 110 to 120 tonnes are used, Figure 9.

Figure 9. Placing of class I stones, 16 to 30 tonnes, in the head of the Husavik berm breakwater.

Tolerance for the placement of stones on berm breakwater is greater than for conventional breakwater and less strict placing techniques is needed. Usually no careful underwater placement is necessary. The front slope is steep and stones can be dropped or thrown by backhoe excavators or cranes. Placement of stones, up to 5 tonnes, in a slope of 1:1.3 has been achieved down to 8 -10 m water depth with 40 to 50 tonnes backhoes. In the Sirevåg breakwater a 110 tonne backhoe was used to place stones of up to 20 tonnes down to –7.0 m water depth and up to stones of 30 tonnes down to –1.0 m. Good interlocking of carefully placed stones is, on the other hand, specified at the front above low water and up to the edge of the berm, Figure 10.

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

One of the difficult elements in the construction of berm breakwaters is how to build up the thick layers of stones in the berm without filling voids with fines. For some rock type and some stone sizes the excavator can crawl directly out on the stone layers without breaking the stones or causing too much wearing of the belts. In other cases some measures have to be taken to minimise the breaking of stones. Recently contractors have started to use thick steel plates under the belts of the excavators for protection of the belts and stones for this purpose.

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

Experience from Iceland shows that small local contractors can quickly adopt the necessary technique 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 10 contractors. The lowest bid is usually accepted.

Figure 10. Surveying of the class I stones, 16 to 30 tonnes, on the berm at trunk and head sections before building the crest structure of the Husavik berm breakwater.

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

Experience from many breakwater projects has shown that working with several stone classes and careful placement of stones only increases the construction cost insignificantly. It is much more important, both economically and environmentally, to utilize all size fragments of the blasted material. The advantage of sorting the stone mass into several stone classes to strengthen the structure is far greater than the relatively low additional cost.

8. Case history, Sirevåg berm breakwater; Norway

In 1998 the Icelandic Maritime Administration (IMA) and Stapi Ltd. Consulting Geologists were commissioned by the Norwegian Coastal Administration to investigate quarries and design a berm breakwater in Sirevåg, which is located on the west coast of southern Norway (Sigurdarson et al, 2001). The breakwater, Figure _, was to be designed as a statically stable Icelandic-type berm breakwater for a wave height with a 100 years return period. It was also to withstand a wave height with 1000 years return period, which is referred to as the worst case scenario, without total damage.

Sirevåg is exposed to heavy waves from the North Sea. The design wave with 100 years return period for the outer part of the breakwater was established as Hs = 7.0 m with Tp = 14.2 s. Wave measurements were started in the beginning of December 1998 at the location of the breakwater head at 17 m water depth. Measurements are taken every half-hour. Two large storms with waves close to the design storm were recorded during the winter 1998 to 1999, on December 27th with Hs = 7.0 m and Tp = 14 s and on February 4th with Hs = 6.7 m and Tp = 15 s.

To establish a design wave height along the breakwater IMA has performed wave refraction analysis from offshore into the location of the Sirevåg breakwater. The breakwater is partly located on rocky bottom and partly on fine quartz sand. The depth of the rocky bottom is very variable from 3 m to 22 m with steep slopes. Under the outermost 150 m is a flat sand bottom. The breakwater is in all about 500 m long and extends about 400 m into the sea. The equivalent head-on wave height for stability calculations is estimated by the incoming wave height, 50 m or half wave length outside the berm, multiplied by the cosine of the wave obliquity in a power of 0.4 (Lamberti and Tomasicchio, 1997), Table 3.

Table 3. Design Wave Height and the Worst Case Scenario.

Station number along the breakwater (m)	Design wave height, 100 year return period	Worst case scenario, 1000 year return period
	Hs (m)	Hs (m)
0 to 70	4.8	5.3
75 to 125	3.5	3.9
145 to 210	6.2	6.8
215 to 240	6.4	7.3
245 to 275	6.2	6.8
280 to 400	6.7	7.4
Breakwater head	7.0	7.7

In the preliminary design three sets of stone classes were considered. Based on the overall utilisation of all quarried material according to a preliminary quarry yield prediction and fulfilment of stability criteria for all sections of the breakwater, one set was chosen, Table 4.

Table 4. Stone Classes and Quarry Yield.

Stone class	w_{min}-w_{max} (tonnes)	w_{mean} (tonnes)	w_{max}/ w_{min}	d_{max}/ d_{min}	Expected quarry yield
I	20.0 – 30.0	23.3	1.5	1.14	5.6%
II	10.0 – 20.0	13.3	2.0	1.26	9.9%
III	4.0 – 10.0	6.0	2.5	1.36	13.7%
IV	1.0 – 4.0	2.0	4.0	1.59	19.3%

The geological investigation and quarry yield prediction included drilling of 25 cored drill holes and surface scan-lines. Three possible quarries (A, B and C) were assessed for the Sirevåg breakwater. A quarry yield prediction was carried out for the three quarries for a 640,000 m^3 breakwater. The armourstone material is anorthosite gabbro rock of good quality with specific gravity, SSD, of 2.69 and water absorption between 0.19 and 0.26. The point load index exceeds 10. The quarry yield prediction, Figure 11, for a carefully worked quarry is about 50% over 1 tonne, about 30% over 3 tonne and about 15% over 10 tonne. This will result in about 6% in stone class I, 20 to 30 tonne, 10% in stone class II, 10 to 20 tonne, 14% in class III, 4 to 10 tonne, and 19% in class IV, 1 to 4 tonne, Table 4.

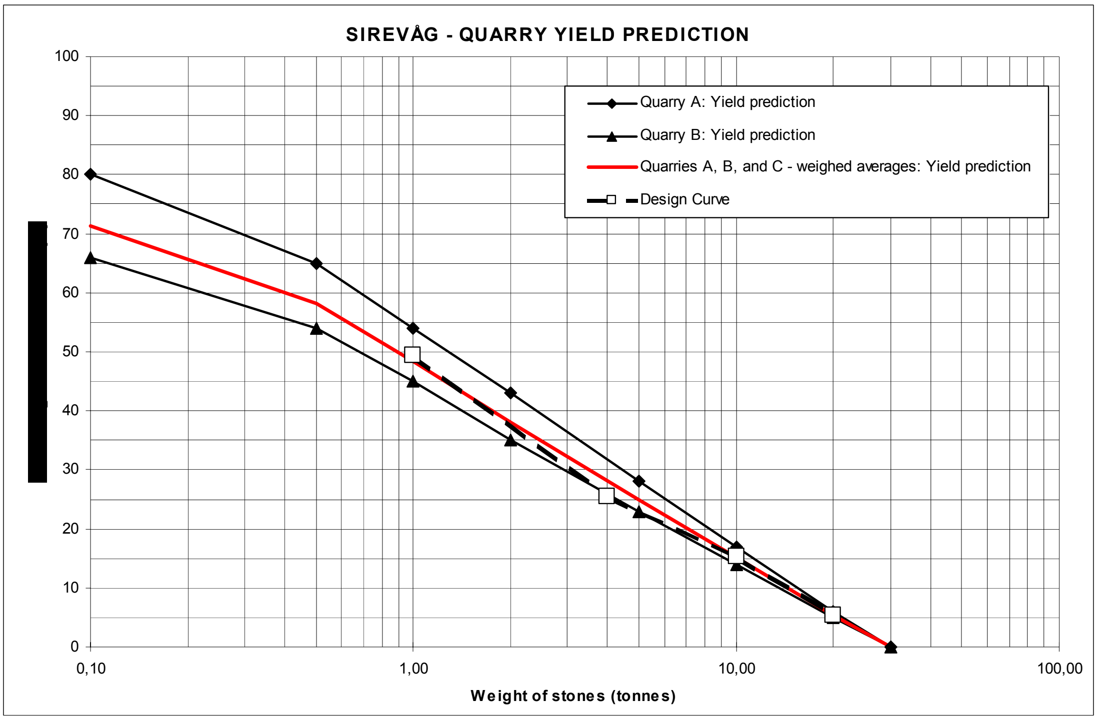

Figure 11. Quarry yield prediction and design curve for the Sirevåg breakwater.

A cross section of the outer part of the breakwater is shown in Figure 12. The design fully utilises all quarried stones over 1 tonne and a 100% utilisation of all quarried material is expected for the project, Table 5.

Table 5. Total volume of the Sirevåg breakwater.

Stone class	w_{min}-w_{max} (tonnes)	w_{mean} (tonnes)	w_{max}/ w_{min}	% of total volume
I	20.0 – 30.0	23.3	33,400	5.2%
II	10.0 – 20.0	13.3	61,400	9.6%
III	4.0 – 10.0	6.0	63,500	9.9%
IV	1.0 – 4.0	2.0	150,500	23.4%
V	0.4 – 1.0	0.6	18,500	2.9%
VI	Quarry run		315,500	49.1%
Total			642,800	100.0%

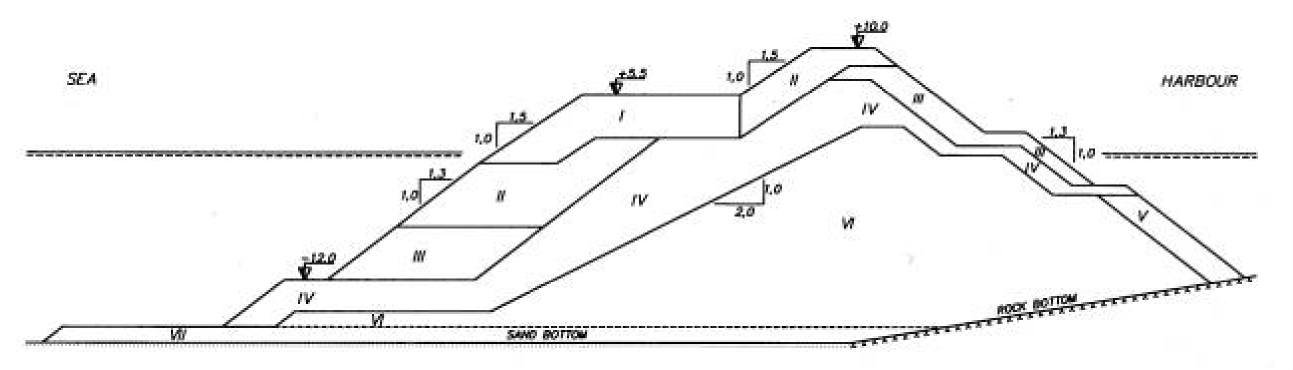

Figure 12 Sirevåg berm breakwater, cross section of the outer part.

Six contractors were pre-qualified to bid on the project. The lowest bidder was E. Pihl & Søn of Denmark. They draw on experience gained by their subsidiary company Istak of Iceland, which has experience in construction of berm breakwaters. The over all construction cost in the lowest bid is about 12 EUR/m^3 (11 USD/m^3). In average the six contractors priced stone classes I and II about 40% higher than classes III and IV, which again were priced about 40% higher than the quarry run. As classes I and II only make up about 15% of the total volume the total price is very little influenced by the handling cost of the largest stones.

To make comparison with other structures more easy the Sirevåg cross section designed for Hs = 7.0 m has been recalculated for a water depth of 20 m. Then the over all construction cost per m length of structure is about 18,000 EUR/m (17,000 USD/m).

The equipment park used by the contractor consists of 4 backhoe excavators (110, 75, 50 and 25 tonnes), 3 front loaders (75 and two 45 tonnes), 3 dumpers, a split barge of 250 m^3 capacity and 3 drilling rigs. In the preparation phase the contractor considered the possibility of using a 200 tonnes crane for placing the largest stones on the breakwater. However, he decided to use a large excavator for both sorting the largest stones and for placing them on the breakwater.

The construction of the Sirevåg berm breakwater started in March 2000 and was finished in August 2001, 3 months ahead of schedule, without any claims from the contractor, Figure 13.

Figure 13. The Sirevåg breakwater was finished in July 2001 three months ahead of schedule.

It has become apparent that in a project of this size larger excavators and wheel loaders are most appropriate for handling the largest stones. It may, however, be equally important to have smaller machines for the sorting and handling of the smaller stone classes, as they are equally critical in the production. The lack of smaller excavators in sorting of smaller stones may lead to the loss of a high percentage of these stones into the quarry run.

The Sirevåg breakwater was hit by a severe storm on January 28, 2002. A Waverider buoy located 200 m off the breakwater head measured wave heights at half an hour interval. Max recording during the storm was Hs = 9.75 m. SINTEF has analysed the measurements and the wave height exceeded Hs = 8.5 m for a duration of 3 hours. The reduction of wave height from buoy to the front part of the breakwater and the breakwater head is 12% according to SINTEF's refraction analysis, which corresponds to Hs = 7.5 m at the front part of the breakwater, well above the design wave of Hs = 7.0 m. The breakwater survived the storm without any reshaping, and only a few stones were moved from their original location, Figure 14.

Figure 14. The Sirevåg breakwater after the storm in January 2002.

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Figure text:

Figure 1. The Sirevåg berm breakwater in Norway built as an "Icelandic type" statically stable non-reshaping structure.

Figure 2. No construction unit is as far from being standardised as the armourstone in the primary cover layer with regard to form, strength and durability. Basalt stones in the Bolungarvik berm breakwater. A large part of the stones has some unopened cracks or geological flaws.

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