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Stable berm breakwaters

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ABSTRACT: Berm breakwaters have basically developed in two directions. On one hand are the dynamic structures built of few stone classes and are allowed to reshape. On the other hand are the more stable structures built of several stone classes, where only few stones are allowed to move. These structures are sometimes referred to as Icelandic-type berm breakwaters. The general method for designing an Icelandic-type berm breakwater is to tailor-make the structure around the design wave load, possible quarry yield, available equipment, transport routes and required functions. Quarry yield prediction is presented as a tool in breakwater design. The Sirevåg berm breakwater is designed for 100 years design wave conditions of Hs=7.0 m as a non-reshaping stable berm breakwater with expected mean recession less than 2 stone diameters. It is built up of five stone classes, the largest stone class being 20 to 30 tonnes, fully utilising all quarried stones over 1 tonne and a 100% utilisation of all quarried material is expected for the project. Construction cost for a cross section on 20 m water depth is presented which proves that the structure is very economical.

1 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. Basically berm breakwaters have developed in two directions. On one hand are the dynamic structures built of few stone classes, usually only two, core material and berm stones. On the other hand are the more stable structures, sometimes referred to as Icelandic-type 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. These breakwaters were built by dumping all quarried material at the breakwater site. It was stated that when “maturing” the breakwaters might develop an S-shape.

In the early 1980’s the berm breakwater was introduced. For the protection of a runway extension in Unalaska, Alaska, Hall et al. (1983) proposed a wide berm of one rock 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.

Gradually the 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 parameter, $Hs/\Delta Dn50$, 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 partners involved, designers, geologists, supervisors, contractors and local governments. The Icelandic type berm breakwater is build up of several size-graded layers in contrast to the original idea of one rock class

2 PIANC WORKING GROUP ON BERM BREAKWATERS

In 1998 a PIANC Working Group was established to formulate guidelines for the design of berm breakwaters. The group should study different research results and compile all relevant information into practical guidelines for the design of berm breakwaters. A part of this work has been to gather information on constructed berm breakwaters around the world, 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. The Icelandic berm breakwaters constitute nearly half of the constructed berm breakwaters in the world.

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). In Norway three new berm breakwaters have been constructed during the last three years. This could indicate that when designers/authorities have overcome the scepticism of berm breakwaters and realised the economy of these structures, they find more and more use for the concept.

Table 1. A list of constructed berm breakwaters

Country	Number of constructed BB	The year the construction of the first BB finished
Iceland	27	1984
Canada	5	1984
USA	4	1984
Australia	4	1986
Brazil	2	1990
Norway	5	1991
Faeroe Islands	1	1992
Iran	8	1996
Madeira	1	1996
China	1	1999
Total number	58	

3 DESIGN PHILOSOPHY

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 life time. To fulfil these criteria the berm has to be 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. 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). Tørum et al (1999) also introduced limit state designs for berm breakwaters, a method which also is applicable for performance evaluation.

On the other hand Tørum et al. (1999), Tørum et al (2001b) have introduced a new test method for evaluating the breaking strength of the stones in relation to the impact energies a rolling stone on a dynamic berm may encounter. The purpose is to develop methods to estimate the suitability of specific quarries for dynamic berm breakwaters.

The Icelandic fishing harbours are small but they have to withstand some of the most severe wave conditions in the world. As berths are often located just behind the breakwater it is necessary to minimise the wave penetration into the harbours and the wave overtopping. The berm concept has proven to be a successful solution for navigational safety for small entrances with heavy breaking waves due to little reflection and low overtopping compared with wave conditions associated with conventional breakwaters. One berm breakwater of the Icelandic type has been constructed on a weak soil 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.

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. 1999). 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.

4 DAMAGE CRITERION, DESIGN PARAMETERS

The design criteria for rubble mound structures has developed considerably over the past 30 years, from being 5% damage for 25 to 50 year design return period, to the present 0 - 2% damage for 100 year design return period. Structural failure is no longer accepted. The increased demands on functional and technical criteria of the structures has led to much stricter criteria for the design of the berm thanks to the increased knowledge on design wave conditions, the strength and durability of rocks, possible quarry yields and the construction methods.

The stability number of a conventional rubble mound breakwater is related to damage on 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 D_{n50} 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 (usually the 100-year storm) the recession of the berm shall not exceed two stone diameters, $Re/D_{n50}<2$. 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), (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 parameter of the edge of the berm (Ho), the width of the berm measured on design water level into the core of the structure (B), and the gradation of armourstone classes (f_g). 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) and also by Tørum et al (2001a)

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 of dynamic berm breakwaters where armourstones are dumped but not placed. The importance of interlocking is well known from conventional breakwaters.

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

5 STABILITY TESTS

Although multilayer berm breakwaters have been built for many years in Iceland, few well documented stability test existed, until Tørum et al. (2001) carried out tests, basically on the Sirevåg berm breakwater (see later). 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 orderly, which is in contrast to the actual design. As has been shown for single layer rubble mound breakwaters, Hald et al (1998), an orderly placement

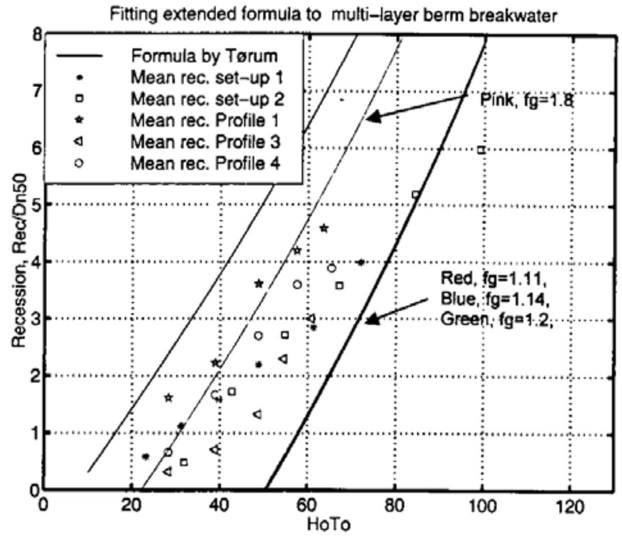

Figure 1. Dimensionless recession vs HoTo for homogenous berm breakwaters (solid lines for different gradations f_g) and multilayer berm breakwaters (points).

of the stones with a backhoe will increase the stability, provided that the orderly placed stone layer goes far down the slope. However, satisfactory orderly placement can only be achieved down to approximately 1 m below low water.

Tørum et al. (2001a) also analysed several test series with homogenous berms, but with different gradation and water depth. They arrived at an equation for the dimensionless mean recession of the berm, Re/D_{n50}. The mean recession depends on $HoTo = H_s/(((\rho_s/\rho_w)-1)D_{n50})((g/D_{n50})^{0.5})T_z$, where ρ_s= density of stone, ρ_w = density of water, g = acceleration of gravity, T_z = mean period. The mean recession is also dependent on the stone gradation, $f_g = D_{n85}/D_{n15}$, and dimensionless water depth d/D_{n50}. This equation is plotted in Figure 1 for different gradation, f_g, together with data for multilayer berm breakwaters. Test set-up 1 data are the data for Sirevåg breakwater. For the multilayer 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).

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 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 for information on the maximum quarry yield. But dedicated armourstone production is not common and therefore there are not many contractors that 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 yields prediction and rely on the 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 (20-30 tonnes) stones, typically to improve the stability of the edge of the berm. By increasing the size of the stones at the edge of the berm by a factor of two, the design wave height may increase by 25%. The percentage of large stones produced in the quarry can be as low as 2-5 % of the total quarried volume to allow for this 25% increase in design wave condition. 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 rela-tively 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 condi-tions but also applies to lower wave load conditions where quarries with relatively low yield size distri-bution 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).

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 suite each particular quarry and he may have to adjust his pattern several times within the same quarry to maximise his re-sults. We usually find that a drill pattern with a 3" drill bit close to 3-4 m burden (b) and 2-2.5 m spac-ing (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.

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

Breakwater site	Predicted Quarry Yield			Volume
	>20 t	>10 t	>5 t	(m^3)
Bolungarvik	2	5	11	265,000
Blonduos	4	9	14	100,000
Hornafjördur, S-Barrier	2-5	5-10	15-20	60,000
Hornafjördur, E-Barrier	5-10	10-15	15-20	100,000
Husavik	3-4	7-10	12-16	300,000
Sirevåg, Norway	15-17	22-25	30-33	640,000
Vopnafjördur	10-20	20-30	30-40	40,000

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 200 g (+/-50 g) per cubic metre of solid rock, depending on rock soundness and desired block size. Contrary to CIRIA/CUR (1991) we maintain that explosives with a high shock energy and lower gas content give better results. We also prefer explosives with higher detonation velocities, close to the sound velocities of the rock mass. Otherwise the sound wave may be reflected from the quarry face back to the blasted wall before the explosive have opened up the blast line, causing unnecessary additional damage to the blasted armourstone.

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 block will have to be split afterwards using a single 2” to 3” hole or a row of narrower hand drilled holes for more accurate splitting into two pieces. A single hole cannot be recommended unless the quarry yield is somewhat better than the design requires. A steel ball of 6-7 tonnes is sometimes used to split blocks but it can only be recommended in quarries exceeding the demand of the design. 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 (Smarason et al. 2000). 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 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.

7 SIREVÅG BERM BREAKWATER

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. The breakwater, Figure 2, was to be designed as a 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 year 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 estab-lished by SINTEF as Hs = 7.0 m with Tp = 14.2 s (SINTEF 1999). Wave measurements were started in the beginning of December 1998 at the location of the breakwater head at 17 m water depth. Meas-urements 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.

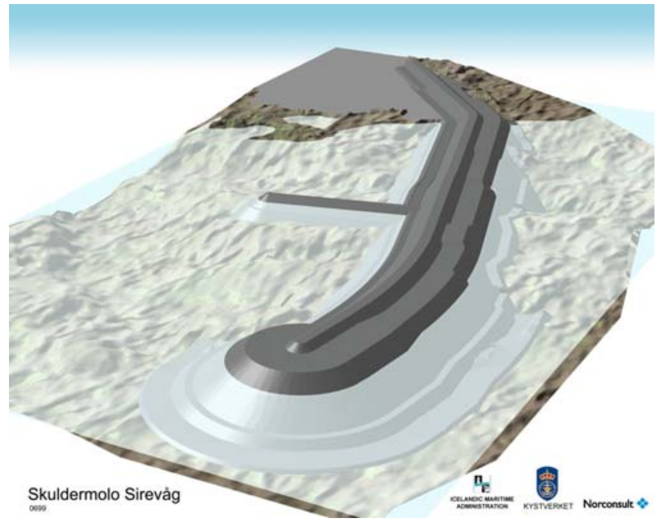

Figure 2. The Sirevåg berm breakwater

To establish a design wave height along the breakwater IMA has performed wave refraction analysis from offshore into the location of the Sirevåg breakwater (IMA, 1999). The HISWA wave model was used for this purpose. The breakwater will partly be 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 ex-tends 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

During the preparation phase of the Sirevåg project various model tests were performed at SINTEF. An interesting study was made to compare wave damping for different configurations of berm breakwaters with a conventional rubble mound breakwater (Jacobsen et al. 1999). The analysis shows that berm breakwaters reduce the wave energy penetrating around the breakwater head and into the harbour more efficiently than a conventional rubble mound breakwater of equal length.

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%

With reference to Figure 1 the mean recession for the 100 year design wave conditions, $HoTo_{100} = 48$, is found to be $Re/D_{n50,100} = 2.0$ or Re = 4.2 m. For the 1,000 year condition, $HoTo_{1000} = 60$, the mean recession is $Re/D_{n50,1,000} = 3.0$ or Re = 6.3 m, and for the 10,000 year conditions, $HoTo_{10,000} = 72$, $Re/D_{n50,10,000} = 3.5$ or Re = 7.3 m.

As long as HoTo is below approximately 70 (Ho below 2.7) the breakwater will reshape into a reshaped static stable breakwater, e.g. the stones will move down the breakwater once and not up and down the breakwater slope. When HoTo is above 70 (Ho above 2,7) a berm breakwater will reshape into a reshaped dynamic stable berm breakwater, e.g. the stones will move repeatedly up and down the slope.

Tørum et al (1999) proposed some limit state criteria for berm breakwaters. The accidental limit state criteria was that for the 10,000 year wave conditions the width of the berm should at least be equal to the mean recession $W_B = Re_{10,000,mean}$. The Sirevåg berm breakwater meets these criteria.

There has not been any investigation of the capability of the stones from the Sirevåg quarries to withstand impacts. The quarries are classified to give high quality stones and in view of the results of Tørum et al. (2001b) it is believed that there is only a very small probability that the stones will be crushed during the reshaping process, even for the 10,000 year waves. The Sirevåg berm breakwater thus performs very well for waves exceeding by far the 100-year design wave conditions.

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 (Stapi

Figure 3. Sirevåg berm breakwater, stock pile of stone classes I and II

Consulting Geologists 1999). The armourstone material is anorthosite gabbro rock of good quality, Figure 3 with specific gravity, SSD, of 2.69 and a water absorption between 0.19 and 0.26. The point load index exceeds 10. The quarry yield prediction, Figure 5, 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.

A cross section of the outer part of the breakwater is shown in Figure 6. 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%

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

Figure 4. Construction of the Sirevåg breakwater, the 110 tonnes excavator placing class II stone

subsidiary company Istak of Iceland, which has experience in construction of berm breakwaters. The over all construction cost in the lowest bid is about 1300 JPY/m^3 (11 USD/m^3 or 12 EUR/ 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=7,0m has been recalculated for a water depth of 20 m. Then the over all construction cost per m length of structure is about 2 million JPY/m (17,000 USD/m or 18,000 EUR/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. He, however, decided to use large excavator both in sorting the largest stones and placing them on the breakwater.

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 too high percentages of these stones into the quarry run.

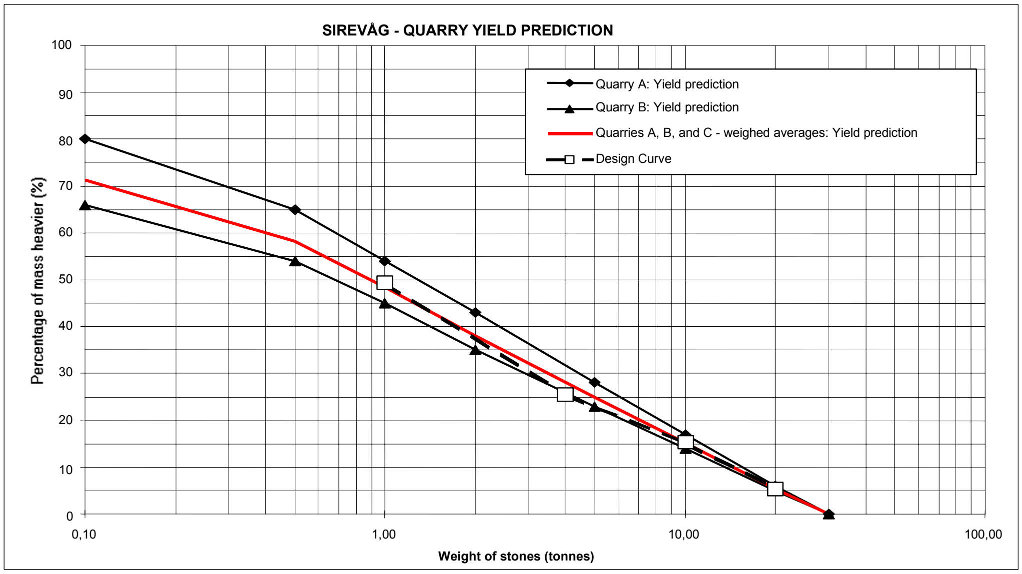

Figure 5. Quarry yield prediction for and design curve for the Sirevåg breakwater.

The number of personnel at the construction site has been between 30 and 35 during the main construction period

The contractor started production in quarry B in March 2000, but from July 2000 he has been quarrying both in quarry A and B. Quarry B is expected to give about 400,000 m^3 and quarry A about 220,000 m^3. By the end of December 2000 about 580,000 m^3 have been quarried in quarries A and B, of which 290,000 m^3 have been placed by the split barge, 140,000 m^3 have been filled from land and 130,000 m^3 are on stock pile. The yield from the quarry has been more or less as predicted, classes I and II have been slightly over prediction, where as classes III and IV have been slightly under. The blasting technique the contractor has chosen coupled with the lack of small excavators may have contributed to the lack of stones in the lighter classes.

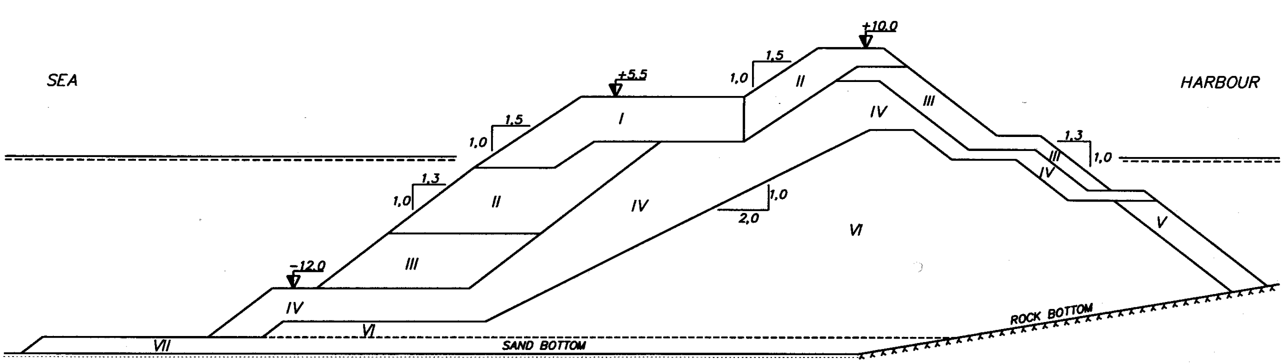

Figure 6. Sirevåg berm breakwater, cross section for the outer part.

Figure 7. Inspection of the front slope of the berm of the Sirevåg breakwater, a section with class I stones.

8 CONCLUSIONS

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 reason and to minimise wave overtopping. The Icelandic type berm breakwater has proved to be a successful solution for navigational safety in harbour entrances with heavy breaking waves.

The design and construction of rubble mound breakwaters is full of variables. Definite criteria to be fulfilled in the design of berm breakwaters will not be set out. The structures will, however, be designed to be as stable as possible. It is the design methods that should be dynamic not the structures.

The Sirevåg berm breakwater performs very well even for the 10.000-year wave conditions.

9 ACKNOWLEDGEMENTS

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