## BERM BREAKWATERS AND QUARRY INVESTIGATIONS IN ICELAND

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ABSTRACT

Rubble mound berm breakwaters are an attractive alternative to the conventional breakwaters. Armour stones can be much smaller, utilization of quarry is better, reshaping occurs instead of failure and performance against overtopping is better.

In Iceland, where breakwaters and shore protection are mostly constructed from locally quarried rock, the berm-type design concept has been widely used. The Icelandic Harbour Authority has attained over 12 year experience in the design and construction of rubble mound berm structures. Seventeen such structures have been constructed since 1983. Further four are under construction this year. All berm breakwaters in Iceland have been constructed by local contractors who have developed considerable experience in this type of work.

Basalt lavas, dolerite or gabbro are the rock types used for armour stone of most Icelandic breakwaters. Rock quality is variable depending on rock type, jointing, density and degree of alteration. Gabbro and porphyritic basalt have excellent to good quality, whereas other basalts are generally good to poor. Fracture measurements are used to estimate joint intensity and forecast quarry yield for planned rubble mound structures. There is a good correlation between fracture frequency and actual quarry yield and forecasts based on simple joint measurements have in several cases been very close to the results of blasting.

The paper describes the design philosophy for rubble-mound berm breakwaters, The estimated yield from an armour stone quarry is used as an integrated part of the design in an attempt to optimize the utilization of the quarry. All size grades are used from the quarry. The largest stones are used in the most critical areas to strengthen the structure. Experience shows that this does not increase the construction cost. The aim is to minimize stone movement and make a statically stable structure, if a suitable quarry can be found in the vicinity. The design approach is to tailor-make the rubble mound structure with respect to wave load and possible quarry yield. Where this design approach has been followed breakwaters that have experienced wave conditions close to the design wave conditions do not show any deformation of the construction profile.

## 1. INTRODUCTION

A berm breakwater can often be built at a considerable cost savings compared with a conventional breakwater with two layers of armour stone, especially at large water depth and where design waves

are high. The main armour can be constructed using relatively small stones. At exposed areas large armour stones for a conventional breakwater design are usually not available in Iceland, thereby requiring prefabricated concrete units, which significantly increase the construction cost.

Up to 100 % utilization of the quarry yield can be achieved. On the other hand for conventional breakwater an overproduction of core material will be required to produce the required volume of armour stones. The good utilization of the quarried rock is very important in Iceland where shortage of sufficiently large armour stones can be a problem.

The structure can be constructed using readily available land based methods and less specialised construction equipment compared to conventional breakwaters. Usual equipment are a drilling rig, two or three backhoe excavators, sometimes a front loader, and some trucks, the number of which depends on the transport distance. The construction procedures are quickly adopted and local contractors with limited experience in coastal works can be used as the tolerance for the placement of stones are greater than for conventional breakwater designs. Shortage of funds often make it necessary to extend the construction over a two year period. Experience has shown that partially completed berm breakwaters function well through winter storms, and repairs are much easier than for conventional breakwaters.

Failure of a conventional two layer breakwater tends to be abrupt, should the design waves be exceeded, potentially leading to extensive failure. On the other hand, an abrupt failure does not occur for a berm type breakwater if the design conditions are exceeded, and it is quite possible that insignificant damage or deformation would occur under such condition.

The berm, which is a horizontal platform built above the design water level, creates a large area where the waves can propagate amongst the armour stone mass, which is highly porous. The wave energy is dissipated in the armour stone mass and the hydrodynamic forces on the stones are greatly reduced allowing much smaller stones to be used for the armour compared to conventional breakwaters. One positive side effect of the energy dissipation is a minimum wave reflection, but reflecting waves can cause problems for vessels navigating in front of the structure.

In Iceland, seventeen rubble mound structures of the berm type have been constructed since 1983. Ten of those were new structures whereas the others where improvements or repairs of existing breakwaters. Four new rubble mound breakwaters of the berm type are currently under construction. The significant design waves for these breakwaters vary from Hs = 2 m to over 6 m with wave peak periods Tp ranging from 10 to 20 seconds. Some of the breakwaters that have been constructed extend into deep water (25 m) and some are exposed to breaking waves.

Various examples of breakwater projects are discussed, and their location can be seen in figure 1. The Bakkafjordur breakwater built in 1983 and 1984 is the first berm type breakwater to be built in Iceland and probably in the world. The berm concept was introduced in Iceland through the design phase of the the Helguvik breakwater in the early eighties by its originators, (Baird and Hall, 1984). The Bolungarvik breakwater which was completed in 1993 and the Blonduos breakwater which was completed in 1994, are of the berm type. At Akranes and Husavik, berm type structures were built on the ocean side of existing piers as protection but not least to reduce wave overtopping. The Keilisnes project is the design of a harbour for a proposed aluminium smelter, where a large berm type breakwater is proposed with a quay on the leeside.
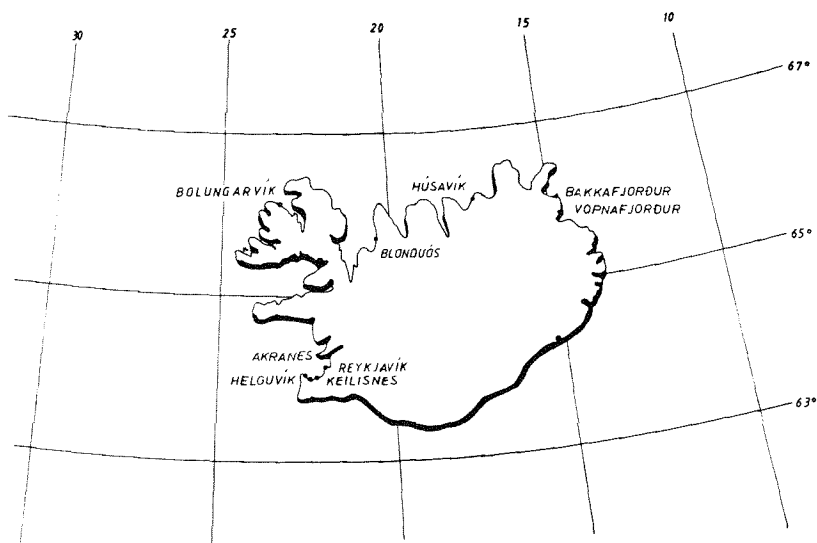
Figure 1. Iceland. Location of harbours referred to in the text.


## 2 SELECTION OF QUARRY

Icelandic breakwaters and shore protection are built almost entirely from blasted blocks from basaltic lavas and occasionally gabbro or dolerite intrusions (Sigurdarson et al, 1994, and Smarason, 1994). Porphyritic basalt is generally preferred since plutionic rocks are rare. Massive porphyritic basalt occurs at a few hundred metres intervals in the Tertiary formation in the eastern, northern and western parts of Iceland and often form prominent benches in the landscape that can be traced by eye across fjords and valleys.

Potential armour stone sites are investigated in the vicinity of proposed breakwaters to locate a suitable quarry with dense durable rock and a coarse fracture pattern to obtain largest possible blocks. Quarries are usually available within a reasonable distance from proposed construction sites. The area closest to the site is searched first and the circle gradually increased until search is successful. Sometimes we are able to locate a quarry adjacent to the construction site although the distance is usually 5-15 km and the furthest distance is about 35 km. Transportation of armour stones can be a large part of the construction cost and as a rule of thumb it could be about 50 % of the cost of a breakwater if the transportation distance is 10 km. Rocks of poorer quality close to the construction site are therefore sometimes accepted for the core and smaller armour layers and armour stones of more durable rocks are transported further for the berm itself.

Land in Iceland is privately owned, state owned, or owned by the local community. The land owner has to agree to the quarry before work van go ahead or the case may go to arbitration.

The Nature Conservation Council must agree to the location of a new quarry and the larger quarries exceeding 200.000 m³ must undergo environmental assessment. The Nature Conservation Council has the power to stop or halt construction. Conflict with their interest is therefore avoided.

## 3  ARMOUR STONE QUALITY AND DURABILITY

Breakwater rocks must generally be dense (specific gravity over 2.8) and durable with respect to freeze/thaw action and abrasion to be accepted for rubble mound structures. Armour stones are expected to stay within the design criteria for at least 50 years to keep the maintenance cost down.

The in-service durability of rock depends largely on its density, and resistance to weathering, abrasion and wave action. The bulk specific gravity (SSD) of a solid vesicle free basalt is around 3. It decreases with increased porosity (vesicles). The range of specific gravity in a sound basalt lava is usually 2.7-2.9. Lower specific gravity usually means that water is absorbed more easily and in our climate this is likely to accelerate the deterioration of the rock through freeze/thaw action and makes the rock more vulnerable to movement and abrasion. This is, however, generally not a severe problem in a relatively fresh (unaltered) basalt with specific gravity over 2.8 if the armour stones size meets the design criteria. Alteration of primary minerals accelerates the deterioration of basalt through freeze/thaw action, wave action and abrasion. The presence of calcite, laumontite, swelling clays and epidote can be fatal. Use of basalt containing these alteration minerals should be avoided if possible and it is essential that the rock be thoroughly tested if such rocks are to be used for a rubble mound structure.

The degradation rate of some armour stones of porphyritic basalt in the breakwater at Vopnafjordur, north-east Iceland, has been found to be up to about 10-20 cm from each side of armour blocks over a period 20 years. The porphyritic basalt in the armour there is fairly altered with calcite and laumontite present as alteration minerals. Similar alteration is present in the rock quarried by the side of the breakwater at Bakkafjordur. The rock type there is tholeiite basalt. This basalt type is much more vulnerable with respect to alteration than the porphyritic basalt, resulting in much faster deterioration of the rock in the breakwater. The difference shows up clearly in freeze/thaw test and a sample of the rock from Bakkafjordur lost 3.6 kg/m² in 35 freeze/thaw cycles, whereas the porphyritic basalt from Vopnafjordur only lost 0.11 kg/m² (Smarason, 1994).

Blasted basalt blocks are usually of suitable size and shape for breakwaters unless the rock has suffered intense tectonic fracturing in one direction. This may result in closely space parallel joints that can seriously affect the aspect ration of armour stones produced through blasting. The rock in the armour stone quarry at Blonduos in northern Iceland is affected by this (Smarason, 1994).

Considerable variation in yield can be expected within the same quarry, due to variation in the rock formation. This makes a comprehensive quality assurance programme necessary during the construction phase to ensure that the armour stones meet the requirements of quality and durability. The quality assurance programme must include the following requirements:
- Grading, weight requirements, specific gravity and water absorption compatible with the design criteria.
- Visible inspection for defects, joints, aspect ratio and colour index.

## 4  JOINTS, FRACTURES AND SIZE ESTIMATES

Spacing of discontinuities (joints and fractures) in a rock mass control the optimum size of armour stones that can be obtained. Smarason (1994) has developed a simple method to forecast the yield of a quarry through blasting. By measuring two scanlines at right angles at the surface and/or vertically and horizontally in a natural or blasted cliff face, Core samples are also measured if available. A frequency plot of the measurements is made for each scanline and an average worked out mathematically. This is then used as a basis to forecast the yield of the quarry in question as well as

the size distribution of the blasted rock. This method seems to work in most cases for Icelandic basalts where the yield over 1 tonne is approximately 10-50%.

Three types of fractures and joints occur in basalts, i.e. cooling joint, flow cleavage and tectonic fractures, The cooling joints and flow cleavage are primary and form as the rock mass solidifies, whereas the tectonic fractures are secondary and form when the rock breaks up because of external tension or movement released in earthquakes. Tectonic fractures are often filled with alteration minerals and dykes of magma that intruded the fractures. Cores tend to have a higher fracture frequency than horizontal exposures in the same basalt rock mass, which is compensated for in the forecast of quarry yield, if cores are available.

A forecast done for a quarry at Bolungarvik in NW-Iceland, where a 200,000 $m^3$ breakwater was completed in 1993, proved to be reasonably accurate. The forecasted yield over 1 tonne was 34% whereas the actual quarry yield over 1 tonne turned out to be about 38% (Smarason, 1994).

Table 1 shows the armour stone classes that were selected for the proposed Keilisnes breakwater at the northern shore of the Reykjanes Peninsula, Fig. 1. The estimated quarry yield for each class is based on a carefully selected and blasted quarry. The last column in the table is an estimated design yield where the percentages above 20 tonnes have been distributed to the lower weight classes. For the design of the breakwater cross sections, the estimated design yield is used in an attempt to maximize the utilisation of the quarry. The bulk specific gravity (SSD) at this potential quarry site has been measured 2.80 - 2.84.

**Table 1**  Armour stone classes for the proposed Keilisnes breakwater

| Classes | Weight (tonnes) | Estimated quarry yield | Estimated design yield |
|---------|-----------------|------------------------|------------------------|
|         | > 20            | 5%                     | 0%                     |
| I       | 14-20           | 4%                     | 4%                     |
| II      | 8-14            | 6%                     | 8%                     |
| III     | 3-8             | 19%                    | 20%                    |
| IV      | 1-3             | 17%                    | 19%                    |

The estimated design yield of armour stones over 1 tonne is 51%. This is fully utilized in the design necessitating a production of about $600 * 10^3$ $m^3$ of blasted rock.

## 5. DESIGN GUIDANCE FOR ARMOUR STONE SIZE

Coastal structures can be classified by the dimension less stability parameter $H_S/\Delta D_{n50}$, where $H_S$ = the significant wave height, $\Delta$ = relative stone density and $D_{n50}$ = characteristic diameter of armour units (rock or concrete) = $(W_{50}/\rho_S)^{1/3}$ where $W_{50}$ is the median stone weight and $\rho_S$ is the stone density. Practical cases from Juhl and Jensen, 1993, show that the parameter $H_S/\Delta D_{n50}$ is in the range from 2.5 to 4.1 which is in the lower end of the classification made by Van der Meer (1988) which defines S-shaped and berm breakwaters as having the $H_S/\Delta D_{n50}$ = 3-6.

The design process in Iceland has evolved from using one or two armour stone classes to using four or even five classes to optimize the use of the quarried rock. Experience shows that it is desirable to have the stability parameter $H_S/\Delta D_{n50}$ of the main armour layer in the range of 1.7-3.0 for breakwater trunk sections and 1.7 - 2.3 for breakwater round-heads. Usually the main armour layer consists of one or two layers of stones on top of the berm and on the slope from berm up to crest.

Lower values of the stability parameter represent breakwaters where the armour stone quarries have allowed using larger stones in relation to the design waves or where larger armour stones are used to compensate for poor quality of rock.

The aim is to minimize stone movements and make a statically stable structure, not a dynamically stable one. When stones start to roll and hit each other there will be high abrasion and spitting of stones occurs. The aim is not a dynamically stable structure, where as the berm concept is used as the structure can tolerate some reshaping in contrast to conventional type structure.

The design approach is a tailor-made structure with respect to wave load and possible quarry yield.

## 6 CASE STUDIES

### 6.1 The Helguvik Breakwater

During 1986 - 1988 a berm breakwater was built in the Helguvik Bay for a tanker terminal close to the Keflavik NATO air base, Fig. 2. The breakwater was built by Icelandic contractors for the U.S.Navy with a design contract with Bernard Johnson Inc. which retained W.F.Baird and Associates to assist in the development of the design for the breakwater (Baird and Woodrow, 1988). The design took place in the early eighties when the berm concept was formulated (Baird and Hall, 1984).



STONE CLASSES

I   $1,7 \leq W \leq 7,0\ t$   $\overline{W} = 3,2\ t - 4,2\ t$
II   $0,5 \leq W \leq 1,7\ t$   $\overline{W} = 1,0\ t$
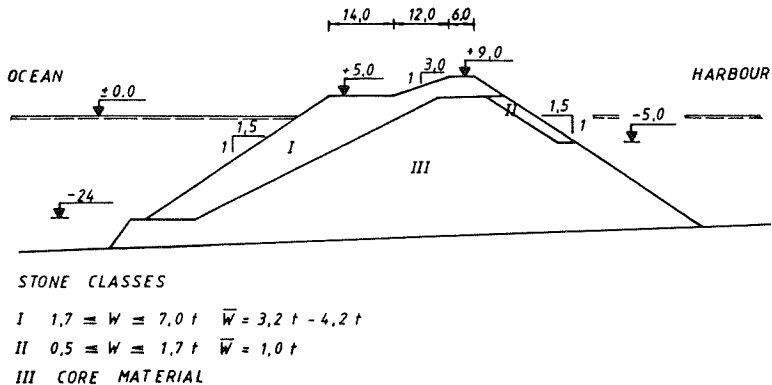III   CORE MATERIAL

Figure 2. A cross section of the Helguvik breakwater.

In the Helguvik Bay the 50 year design wave condition was estimated as a significant wave height Hs=5.8 m and a peak wave period $T_p$=9.6 s, with waves attacking the breakwater at a 45° angle. Reducing the wave height due to oblique wave attack according to de Waal and van der Meer 1992 for runup for short crested waves results in a equivalent wave Hs=5.2 m. The design water level was +5.0 m. The berm design is a relatively simple one with only one class of armour stones of 1.7 to 7.0 tonne stones with mean weight of 3.2 to 4.2 t. The stability parameter $Hs/\Delta D_{n50}$ has been assessed as 2.8 / 2.6 for the equivalent wave height. As the peak period is shorter than for most breakwaters in Iceland a higher stability parameter than is normal provides an adequate stability.

Recent inspection of the breakwater showed no deformation or reshaping of the construction profile.

### 6.2 The Akranes Breakwater

Akranes is located on the north side of the Faxafloi Bay, north of Reykjavik. The harbour is protected by a 330 m long main breakwater. Originally the breakwater was made of 3 Phoenix-type concrete caissons from the Normandy landing operation in WorldWar II, which were towed up to Iceland in 1950. Later the caisson breakwater was protected by a conventional rubble mound and a wave screen on top of it and extended by a conventional rubble mound breakwater. Due to shortage of funds the construction was done in segments during the years 1976 to 1980.

The harbour is exposed to a combination of ocean and wind waves, which have been estimated as ocean waves Hs = 2.8 m with $T_p$ of 18 seconds and wind waves up to Hs=3.0 m with $T_p$=8 seconds. Although the two wave situation occur simultaneously they are not at its peak at the same time. The estimated wave height with 50 year return period is Hs = 3.8 m.

During 1980 to 1984 three large storms hit Akranes. The breakwater head and the outermost 55 meters of the breakwater were washed out and down below high water level. There were damages several places along the breakwater outside the wave screen, specially at the intersections between different construction phases (Bruun, 1985, and Viggosson, 1990).

A 3D hydraulic model test was conducted in 1985 at the scale of 1:45 to optimize the rubble mound protection located in front of the wave screen. The wave conditions, duration and different water levels of the storms as well as the overtopping and damages were reproduced very accurately in the model.

At that time very little data was available to quantify the admissible overtopping discharge. Japanese researchers (Fukuda et al, 1974) had published a guidance on permissible volumes of overtopping discharge in relation to inconvenience or danger to personnel and vehicles. It is necessary to mention that the overtopping phenomenon is highly irregular both in time and space. The admissible overtopping, expressed in $m^3$/s per m length, are of the order $4*10^{-6}$ to $3*10^{-5}$ $m^3$/s for inconvenience to danger to personnel and $1*10^{-6}$ to $2*10^{-5}$ $m^3$/s for inconvenience to impassable for vehicles. These rates were assumed to be to conservative for the overtopping volumes under design wave conditions. Recently an extensive research on overtopping has been done in European hydraulic laboratories with the same conclusion (Franco et al, 1994). Under design condition the following criteria was established (Viggosson & Sigurdarson, 1986):

*For wave conditions with 50 year return period and a 3 year return period of a water level, the overtopping during 20 minutes should not exceed 0.5 m³/m which equals 4.2\*10⁻⁴ m³/~.sec.*

The above criteria, which are about 10 times less conservative than those proposed by Fukuda, correspond approximately to one wave overtopping the breakwater during a 20 minute period under the design conditions.

The improvement was investigated by testing 13 cross-sections of conventional and berm type. The tests resulted in lower and less voluminous cross-section for the berm type as compared to the conventional 1:2 mound, since the uprush was lower. The crest elevation was +11.0 m compared to +11.5 and the total volume of core and rocks along the 160 m segment was 24,500 $m^3$ compared to 29,500 $m^3$. In addition a better utilisation of the quarry was obtained by the berm type. In the model the 3D effects were quite large, as part of the overtopping was due to a wave reflection from adjacent rocky beach with a small bay. It was found that by not closing the little bay the crest elevation of the berm cross section could be lowered to +10.0 m, Figure 3.

The main armour layer, two layers of stones on top of the berm and one on the slope from the berm

to the crest, consists of 5.0 to 8.0 tonne stones with mean weight of 3.0 tonne and a stability parameter $Hs/\Delta D_{n50}$ of 1.67. Under this layer 0.5 to 6.0 tonne stones are used with a mean weight of 1.5 tonne and a stability parameter of 2.3. The rubble mound was built in 1991 and there has not been any reports of overtopping since. A recent inspection showed no deformation of the construction profile.
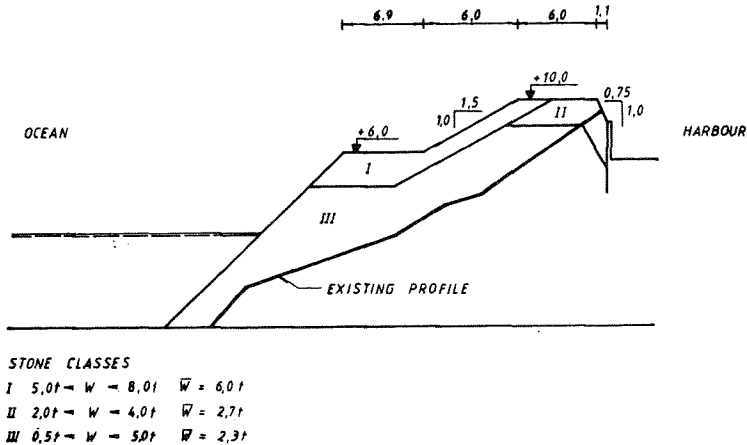


STONE CLASSES

I   $5.0t \to W \to 8.0t$   $\bar{W} = 6.0t$
II  $2.0t \to W \to 4.0t$   $\bar{W} = 2.7t$
III $0.5t \to W \to 5.0t$   $\bar{W} = 2.3t$

Figure 3. Akranes harbour. Cross section of the berm type rubble mound.

### 6.3 The Husavik Breakwater

Husavik is a fishing village located on the northern shore of Iceland. The Husavik harbour is exposed to north-westerly waves, with a design significant wave height Hs of 4.0 m and wave peak period Tp of 15.5 seconds. The design water level is + 2.7 m. The existing pier, a 10 m wide concrete caisson, is used for export of container cargo. Due to heavy overtopping of waves and increased turnover of containers, there was an urgent need to improve conditions at the berth. The solution was to build a berm type rubble mound on the ocean side of the pier and increase the width up to 30 m, in order to minimize the overtopping and at the same time to stabilize the old caissons , Figure 4.

The berm design is relatively simple with only one stone class of 1 to 5 tonne stones in the main cross section, giving a stability parameter $Hs/\Delta D_{n50}$ of 2.6. Larger stones are used on the outer corner up to the existing pier due to concentration of wave energy at the roundhead. Smaller stones are used on the landward end.

Construction of the rubble mound was extended over a period of two years due to lack of funds. In the second phase, which was built in 1990, the top section was built up to elevation +6.0m with the core withdrawn or lowered several meters down to elevation +1.0 m to increase the width of the armour at the upper part of the berm. In December 1992 Husavik was hit by a storm estimated as one of the worst in this century. Still there have been no reports of overtopping or reshaping since the construction.
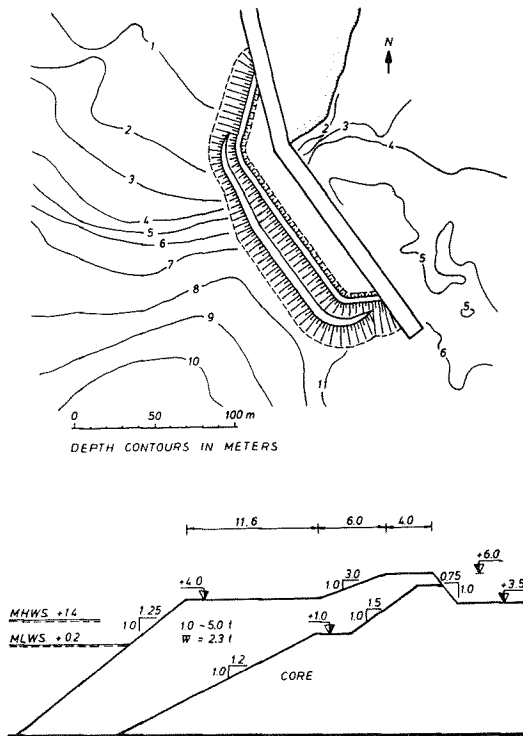
Figure 4. Husavik harbour. Layout and cross section of the berm type rubble mound.

## 6.4 The Blonduos Breakwater

Blonduos is also located on the north shore of Iceland. The existing harbour facility consists of a pier on a coast which is open for northerly and southerly waves. As a first phase in building a safe harbour for fishing vessels up to 30 m in length, a 225m long rubble mound berm breakwater on the north side of the pier was constructed in 1993 to 1994, Figure 5. The area between the breakwater and the existing pier will give shelter for fishing vessels against all wave directions.

The design wave is a combination of long swells from north east and local wind waves from north, which gives Hs of 4.8 m and Tp of 12 seconds. The design water level is +2.5 m.

The total volume of the breakwater is 95,000 m$^3$, of which 55,000 m$^3$ are stones larger than 0.4 tonne and 40,000 m$^3$ is core material. The main armour is 1 to 6 tonne giving a stability parameter Hs/$\Delta$D$_{n50}$ of 2.8. At the breakwater head, 6 to 10 tonne stones are used on top of the berm which corresponds to a stability parameter of 2.0. To utilize the quarry better smaller stones, 0.4 to 1.0 tonne are used inside the berm.

The quarry yield is expected to be 20% of armour stones between 1 and 6 tonne and 4% between 6 and 10 tonne. The total required volume of 1 to 6 tonne stones is 37,000 m$^3$, which requires production of 185,000 m$^3$ of blasted rock. This production will give about 7400 m$^3$ of 6 to 10 tonne stones whereas the design uses 7,000 m$^3$. This means that the quarry is fully utilized for material

down to 1 tonne.  On the other hand, the production will give about 28,000 m³ of 0.4 to 1 tonne stones, while only 11,000 m³ are needed in the design.



DEPTH  CONTOURS  IN  METERS



STONE CLASSES

I    6.0 - 10.0 t
II   1.0 - 6.0 t
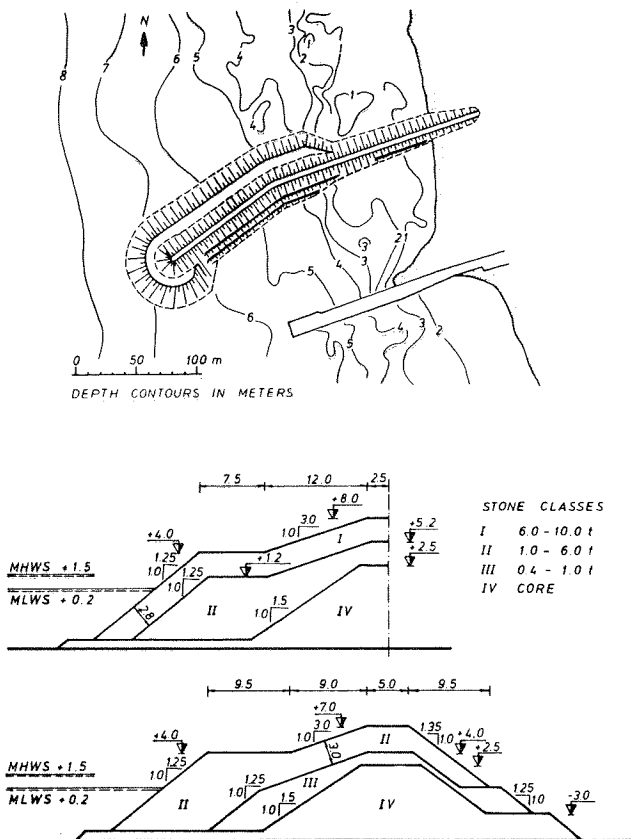III  0.4 - 1.0 t
IV   CORE

Figure 5.  The Blonduos breakwater.  Layout and cross section of the breakwater.  The upper cross section is from the breakwater head and the lower from the trunk section.

The smaller stones inside the berm are not large enough to withstand the design wave condition. Although some reshaping of the berm is expected, this layer is not expected to be exposed directly to wave attack, but it gives the necessary void for the waves to propagate into.

The core is available at the construction site while the rock has to be carried 26 km distance along public roads which limits the size of trucks used.  The cost estimate was divided as follows: 15% of the total price is for blasting and sorting in the quarry, 65 % for transport and 20 % for placement of the rock at the construction site.   The total overall unit price to the contractor was about 20 $/m³ (including 24.5% VAT)

The breakwater was hit by a severe storm  last January without any displacement of stones.

## 6.5 The Bolungarvik Breakwater

Bolungarvik is an active fishing harbour located on the north west shore of Iceland, the so called West fjords. The existing harbour facilities consist of a main breakwater, 215 m long pier which gives little shelter from northerly waves. It is the main loading/unloading facility for trawlers, capelin boats and coasters. Due to large wave action at the leeside of the breakwater and huge wave overtopping it was closed down frequently during the winter time.

A hydraulic model study was conducted in 1985 to optimize the layout and the length of an addition to the breakwater/pier. Several lengths were tested and the optimal solution was to construct a berm type breakwater on the ocean side of the existing pier which extended 90 m further. Very little was gained making the breakwater longer, both with respect to wave agitation into the harbour as well as safety in approaching the harbour in breaking waves. This allowed widening of the existing pier to 60 m creating ample space for storage of cargo and other functions. Construction started in 1992 and completed 1993.

The design wave consists of north-westerly swells with a significant wave height Hs of 6.3 m and wave peak period Tp of 17 seconds. The design water level is +3.5 m. The layout and typical cross sections of the breakwater are shown in Figure 6. There are three major stone classes with a main armour layer of 3 to 8 tonne stones giving a stability parameter $Hs/\Delta D_{n50}$ of 3.1. Under the main armour layer stones of 1 to 3 tonne are used with $Hs/\Delta D_{n50}$ of 4.3. In this design the main armour layer is thinner than for the Blonduos breakwater. At the breakwater head, 8 to 14 tonne stones are used on the top of the berm with $Hs/\Delta D_{n50}$ of 2.4.

All material was produced in a quarry located 5 km from the construction site. The total volume of the breakwater is 200,000 $m^3$, with equal parts of core and armour stones. The cost estimate was divided as follows: 25% for the production in the quarry, 50% for transport and 25% for placement of the rock at the construction site. The total overall unit price to the contractor was 16 $\$/m^3$ (including 24.5% VAT)

Last January a severe storm hit Bolungarvik. Although there were no wave measurements in the area the storm is believed to be close to the design condition. Inspection of the breakwater after the storm showed that only very few stones had moved and none more than its length. An interesting feature is that at the uppermost part of the breakwater, constructed as a conventional structure the voids are filled with smaller stones and stones of some kilograms up to about 100 kg have been rolling up and down the slope causing considerable abrasion.

## 6.6 The Keilisnes Breakwater

The Keilisnes harbour is intended to serve a proposed aluminium smelter, located on the Northern site of the Reykjanes Peninsula. Two sites for the harbour were tested in a hydraulic model, where movements of moored ships up to 60,000 DWT were measured. The first location was on an open coast where the breakwater reached down to 32 m water depth and the second location in a small bay with a water depth of 20 m. Based on the hydraulic model tests, the first location was selected for further development of the harbour facilities.

The proposed harbour at Keilisnes consists of a 500 m long breakwater, with a total volume of 1,750,000 $m^3$ , and a single 225 m long and 37.5 m wide quay at the leeside. The breakwater reaches down to a water depth of -32 m and the crest elevation is +14.0 m.
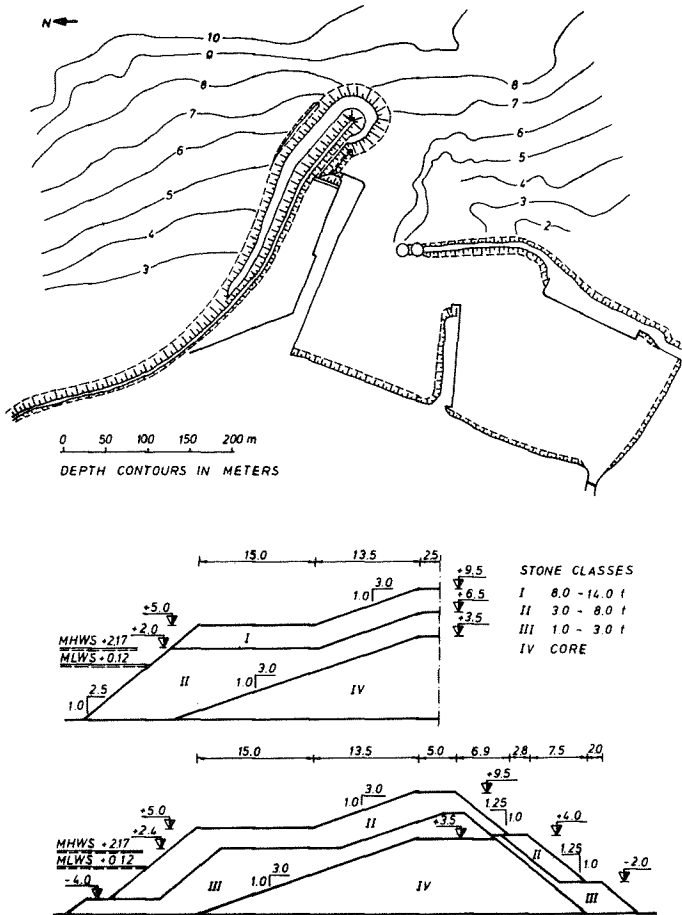
**Figure 6.** The Bolungarvik breakwater. The layout and cross section of the breakwater. The upper cross section is from the breakwater head and the lower from the trunk section.

A suitable quarry is found about 7 km from the construction site yielding sufficient amounts of large armour stones. The majority of the core material will be taken from the smelter site adjacent to the harbour.

The breakwater demonstrated very good stability during the hydraulic test programme, successfully surviving repeated 50 year design storms with Hs of 5.9 m and Tp of 12 s and Hs of 5.3 m and Tp of 16 s. The final reshaped profile developed into a S-shape which was stable when the berm had eroded up to the upper slope of the breakwater.

Typical cross sections are shown in Figure 7. There are two 15 m wide outer berms, the first at elevation -10 m and the second at elevation +6 m. The first berm functions as a platform which supports the upper armour stone layers and will catch stones that roll down as the breakwater develops a S - shaped profile. The function of the upper berm is to dissipate wave energy and is built

from the largest stones available from the quarry. The base material consists of a dredged gravel up to elevation -16 m at the outward slope of the deep water cross section and up to -5 m elevation in the middle of the structure. A total of 276,000 m³ are required of 1 to 20 tonne armour stones, 561,000 m³ of core material and 913,000 m³ of dredged gravel.
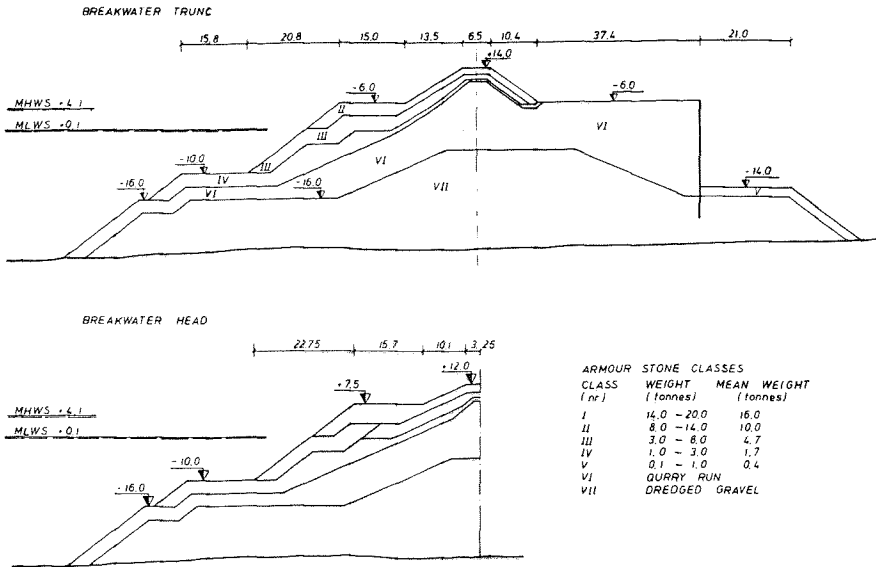


Figure 7. Cross sections of the proposed Keilisnes breakwater.

Armour stone class I, which has an average stability parameter $Hs/\Delta D_{n50}$ of 1.64, is used on top of the upper berm at the breakwater head. Class II, which has $Hs/\Delta D_{n50}$ of 1.98 is used as a main armour layer on top of the upper berm at the breakwater trunk section. Smaller stones, Class III with an average stability parameter $Hs/\Delta D_{n50}$ of 2.54 are used inside the upper berm.

The hydraulic testing programme included overtopping measurements for different significant wave heights, peak periods and water levels (Sigurdarson and Viggosson, 1994). The overtopping was measured on the inner part of the slope from crest to quay at +9.3 m height. By using the same approach as De Waal and Van der Meer, 1992, did for overtopping measured at the front end of the crest, the upper limits of the measured overtopping discharge gave the following formula

$$\frac{q}{\sqrt{gH_s^3}} = 5.89x10^{-5} \exp(7.67\frac{R_{u2\%} - R_c}{H_s}) \qquad (2)$$

where
- $q$ = mean overtopping discharge (m³/s/m)
- $g$ = acceleration of gravity (m/s²)
- $H_s$ = significant wave height (m)
- $R_{u2\%}$ = 2% uprush level
- $R_c$ = crest elevation freeboard (m)

Under design condition the criteria established by Viggosson and Sigurdarson, 1986, was used. The lower limits of the overtopping criteria from Fukuda et al 1974 of inconvenience to personnel or vehicles were used with safe working condition on the breakwater, which are to be fulfilled 98% of

the time. And the higher limits danger to personnel or impassable for vehicles were used with the safe stay at berth criteria for ships, which is not to be exceeded more than once a year.

Although a crest elevation of +12 m is sufficient to satisfy the stability criterion of the rear side of the breakwater (Van der Meer and Veldman, 1992), the overtopping volumes exceeded considerably the overtopping criterion. On the other hand, by increasing the crest elevation to +14.0 m, acceptable levels of discharge are achieved. It is interesting to notice that the safe stay at berth and the 50 year design conditions are equally critical with respect to the overtopping criterion.

### 6.7 The Bakkafjordur Breakwater

The berm breakwater at Bakkafjordur was built in 1983 and 1984 from stones of a very poor quality quarried at the breakwater site. The design wave condition were Hs = 4.8 m with Tp=12.0 s. The berm was constructed of two stones classes, class II 2.0 - 6.0 t with an average weight of 3.0 t and stability parameter $Hs/\Delta D_{n50}$ of 2.66, and class III 0.5 - 6.0 t with an average weight of 1.5 t and stability parameter 3.35, Figure 8. Samples of stones were tested in freeze/thaw test and gave very poor results. Although abrasion tests have not been performed the stones are believed to have little resistance to abrasion. Deterioration of the stones has accelerated a dynamic development of the profile. In the winter 1992/93 the breakwater is believed to have experienced waves close to the design load. The berm was eroded up to the trunk, an unstable S-profile had developed and there was erosion up to the crest of the structure. Repair took place in 1993 and in spite of the poor quality of the rock it was decided to use the local quarry again. The crest structure was rebuilt of stones larger than 2 tonnes and the berm of stones from 0.5 to 2 tonnes. Although the breakwater may need some maintenance every 10 years or so it is considered to be the most economical solution to use the original quarry.
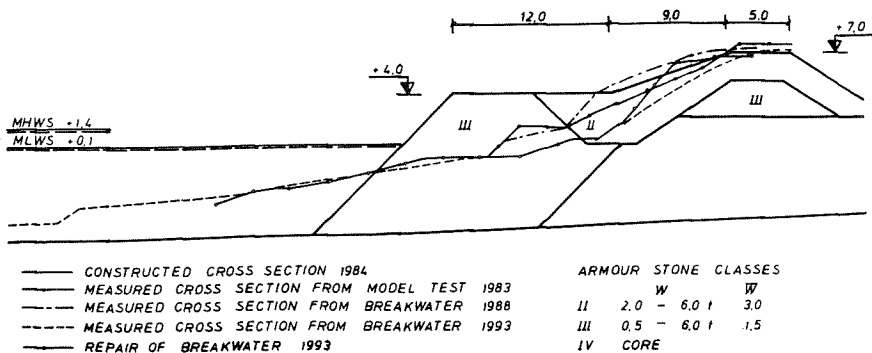


Figure 8. Cross section of the Bakkafjordur breakwater, constructed 1984, measured during model test, measured at breakwater 1988 and 1993 and the repair cross section from 1993.

### 7. CONCLUSIONS

In Iceland berm breakwaters have proved to be an economical solution to improve existing harbour facilities and to build new breakwaters. The economy lies particularly in the use of smaller rock than would be possible for conventional breakwaters and achieving up to a 100% utilisation of the quarry yield. Less specialised construction equipment can be used and easy construction procedures are quickly adopted by local contractors. The berm concept has proved to be very efficient to reduce wave overtopping on exciting breakwaters.

The design aims at minimising the deformation of the berm during design conditions. All the mentioned breakwaters have experienced wave conditions which are close to the design conditions since their construction and except for Bakkafjordur there has been no deformation of the construction profiles. At Bakkafjordur on the other hand with considerable reshaping the berm concept makes it possible to build a breakwater of rock of very poor quality.

A comprehensive quality assurance programme is necessary during the construction phase to ensure that the armour stones meet the requirements of quality and durability.

The design process in Iceland has evolved from using one or two armour stone classes to using four or even five classes to optimize the use of the quarried rock.

So far the berm designs that have been carried out under various conditions, wave heights and periods, breaking/non breaking waves and good or bad quality of rock, promise to be a success, showing a good performance in the severe winter storms that Iceland has experienced in the past few years.

## REFERENCES

Baird, W.F. and Hall, K.R., (1984). The design of breakwaters using quarried stones. 19th International Conference on Coastal Engineering, Houston, pp. 2580-2591.

Baird, W.F. and Woodrow, (1987). The development of a design for a breakwater at Keflavik, Iceland. Seminar on Berm Breakwaters: Unconventional Rubble-Mound Breakwaters, Ottawa.

Bruun, P., (1985). Design and construction of mounds for breakwaters and coastal protection. Elsevier Science Publishers.

De Waal, J. P. and Van der Meer, J. W. (1992). Wave Runup and Overtopping on Coastal Structures. 23rd International Conference on Coastal Engineering, Venice, pp 1758-1771.

Franco, L., de Gerloni, M. and Van der Meer, J.W., (1994) . Wave Overtopping on Vertical and Composite Breakwaters. 24th International Conference on Coastal Engineering, Kobe.

Fukuda, N., Unom, T. and Irie, I., (1974), Field Observations of Wave Overtopping of Wave Absorbing Revetment. Coastal Eng. in Japan, 17: pp. 117-128.

Juhl J. and Jensen, O. J. (1993). Practical experience with berm breakwaters. Draft worked out under the MAST I programme.

Sigurdarson, S. and Viggosson, G. (1994). Berm Breakwaters in Iceland, Practical Experiences. Hydro-Port'94, Int. Conf. on Hydro-Technical Eng. for Port and Harbor Construction, Japan.

Sigurdarson, S., Viggosson, G. and Halldorsson, A. (1994). Rubble Mound Breakwaters and Shore Protection. Hornafjordur Int. Coastal Symposium, Icelandic Harbour Authority.

Smarason, O.B. (1994), Quarry investigations for rubble mound structures in Iceland. Hornafjordur International Coastal Symposium, Icelandic Harbour Authority.

Van der Meer, J. W. (1988). Rock Slopes and Gravel Beaches under Wave Attack. Doctoral thesis. Delft University of Technology. The Netherlands.

Van der Meer, J.W. and Veldman, J.J. (1992). Singular points at berm breakwaters: scale effects, rear, roundhead and longshore transport. Elsevier, Journal of Coastal Engineering, 17, p. 153-171, Amsterdam.

Viggosson G. (1990). Rubble Mound Breakwaters in Iceland. Journal of Coastal Research, SI 7, pp. 41-61, Fort Lauderdale, Florida, Spring 1990.

Viggosson, G. and Sigurdarson, S., (1986), Akranesharbour. Model study of the breakwater. Icelandic Harbour Authority (in Icelandic).