BERM BREAKWATERS, TAILOR-MADE SIZE GRADED STRUCTURES

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ABSTRACT

This paper describes the 13 year experience of design and construction of rubble mound berm breakwaters in Iceland. Over twenty berm structures have been constructed since 1983. All berm breakwaters in Iceland have been constructed by local contractors who have developed considerable experience in this type of work.

The paper describes the design philosophy of berm breakwaters which aims at optimizing the structure with respect to wave load and possible yield from an armour stone quarry. The estimated yield from an armour stone quarry is used as an integrated part of the design process in an attempt to optimize the utilization of the quarry. All size grades from the quarry are used and in many cases a 100% utilization is achieved.

Breakwaters and shore protection in Iceland are mostly constructed from locally quarried rock. Rock quality is variable depending on rock type, jointing, density and degree of alteration. Porphyritic basalt and gabbro have excellent to good quality, whereas other basalts are generally good to poor. Fracture measurements are used to forecast quarry yield for rubble mound structures and are usually very close to the results of blasting. A quality assurance programme has been developed to guarantee that rocks intended for breakwaters are up to the required standard in each case.

The Icelandic Harbour Authority has developed a variant of the original berm concept with all the advantages of the berm concept although the structure is much more stable. This variant can be described as a "tailor-made size graded berm". It allows the use of even smaller stones inside the berm, than in berm breakwaters of only one or two stone classes and as a consequence larger stone classes can be used to reinforce the structure. In many cases this design approach leads to a less expensive structure than the original berm concept, although the structure is much more stable.

1. INTRODUCTION

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

In exposed sites large armour stones are needed for conventional breakwaters. Shortage of sufficiently large armour stones can be a problem, thereby requiring either an overproduction of core material, or prefabricated concrete units, which significantly increase the construction cost.

On the other hand berm breakwaters can be constructed using smaller stones and can be designed to fit the available armour stone quarry. All size grades are used and a 100% utilization of the quarry yield has been achieved in many cases.

The berm, which is a horizontal platform built above the design water level, creates a large area where the waves can propagate into 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 smaller stones to be used for the armour compared to conventional breakwaters.

Berm structures have proved to be very effective in reducing overtopping on existing breakwaters. Several breakwaters with quay on their inner side, suffering large overtopping, have been protected by berm structures. Physical model tests have proved the advantages of the berm concept compared to the conventional rubble mound structures.

Breakwaters often cause wave reflection, which can make navigation in front of them difficult. As the berm breakwater dissipates much of the wave energy with minimum reflection, it does not affect the navigational conditions. Ships navigate very close to berm breakwaters in many harbours, but so far there are no complaints regarding reflections.
2. BERM STRUCTURES IN ICELAND

There is a 13 year experience of design and construction of berm breakwaters in Iceland. The Icelandic Harbour Authority (IHA) is responsible for all investigations needed for the breakwater design, including depth soundings, geotechnical surveys, quarry investigations and in establishing the design wave climate. Often a 3D physical model test is needed both for the alignment of the breakwater and for the stability. IHA is also responsible for tendering out breakwater projects.

Over twenty rubble mound structures of the berm type have been constructed in Iceland since 1983. Fourteen of those were new structures whereas the others where improvements or repairs of existing breakwaters.

The significant design waves for these breakwaters vary from $H_s = 2\ m$ to over $6\ m$ with wave peak periods $T_p$ ranging from 10 to 20 seconds. Some of the breakwaters that have been constructed extend into deep water, $-25\ m$, and a planned breakwater will extend to $-32\ m$. Some are exposed to breaking waves.

Some examples of breakwater projects are discussed, and their location is shown in Fig. 1. The Bakkafjördur breakwater, built in 1983 and 1984, is the first berm type breakwater constructed in Iceland and probably in the world. The berm concept was introduced in Iceland through the design phase of the Helgukvík breakwater in the early eighties by its originators, (Baird and Hall, 1984). The Bolungarvík breakwater which was completed in 1993 and the Blonduos breakwater which was completed in 1994, are of the berm type. At Akranes berm type structure was built on the ocean side of an existing pier as protection but not least to reduce wave overtopping. The Kellisnes 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.

3. CONSTRUCTION / CONTRACTORS

A berm breakwater can be constructed using readily available land based methods and less specialised construction equipment compared to conventional breakwaters. Usual equipment is a drilling rig, two or three backhoe excavators, sometimes a front loader, and some trucks. Backhoes usually weight from 25 to 42 tonne and the largest can handle stones up to about 17 tonne. The number of trucks depends largely 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 is greater than for conventional breakwater design.

When the first berm breakwaters were built bulldozers were used to push stones to the berm. That resulted in breakage of stones and to many fines and that plugged the voids. Now backhoe excavators are used to place stones. Usually no underwater placement is necessary, as the front slope is steep. Good interlocking is required on the front and the edge of the berm. Experience from many breakwater project has shown that working with several stone classes and placement of stones requiring interlocking increases the construction cost insignificantly.

The construction period often extends over a two years. Experience has shown that partially completed berm breakwaters function well through winter storms, and repairs are much easier than for conventional breakwaters.

Each breakwater project is tendered out and there is competitive bidding between from 4 and up to 10 contractors.

Figure 1. Location of harbours referred to in the text. Location of other berm breakwaters are shown by dots and of the offshore Waverider buoys by the buoy symbol.
4. WAVE CLIMATE

Iceland is an island in the middle of the North Atlantic Ocean located between 63° and 67° northern latitude and 14° and 24° western longitude with very rough wave climate. The coastal waters around Iceland are, according to Young, 1994, amongst the most severe coastal waters in the world.

Cyclones from North America pass over Iceland from the SW and generate high waves when reaching the southwest coast. Deep cyclones may stagnate East or North of Iceland for several days generating high waves.

The Icelandic Harbour Authority has operated four offshore wave buoys over the past years (Viggoisson et al. 1988) and recently two new have been added. The location of the offshore Waverider buoys is shown in Fig. 1. Each year these buoys measure large waves with significant wave height over 10 m. To date the maximum measured wave height is $H_s = 16.7$ m with $T_p = 19.4$ s.

A statistical analysis of wave measurements and hindcasts over longer periods show that the yearly significant wave height around Iceland varies from 11 to 13 m and the 100 year wave from 14 to 17.5 m.

Because of the severe wave climate most harbours in Iceland are located in protected inshore areas like fjords and bays, where the wave climate is much milder. Berm breakwaters have been constructed for significant design waves of 2 to over 6 m with wave peak periods ranging from 10 to 20 s. In this wave climate of long period ocean swells, wave setup in the surf zone and infragravity waves, berm breakwaters have proved to be very good.

Heavy storms during the past few winters have caused higher wave activity around Iceland than previously observed over the last 30 years (Sigurdsson and Viggoisson, 1994).

In this light berm breakwater are to be preferred rather than conventional. Should the design waves be exceeded there is a risk of an abrupt failure on the conventional breakwater, while the berm breakwater might only experience some reshaping.

5. QUARRY SELECTION

Icelandic breakwaters and shore protection are built almost entirely from blasted blocks from basaltic lavas and occasionally gabbro or dolerite intrusions (Smarason, 1994).

Porphyritic basalt, which is the best suited basalt type for armour stone production, is usually only found at a few hundred meters interval in the lava pile. They are usually about 10 m thick and often occur in groups of a few layers although only one may be suitable for armour stone production.

Selection of suitable quarries begins with inspection of geological maps and aerial photographs in the vicinity of planned breakwaters and is continued further away from the predicted structures until successful. The search for suitable armour stone quarries is initially aimed at any prominent thick lava flows that may be accessible on low ground or at accessible benches or hillocks. Promising sites are then visited and inspected visually for geological features such as rock type, weathering forms, pores, pore fillings (amygdales), alteration and joint intensity. Further investigation is performed through pneumatic drilling and core sampling at promising sites before bidding. Quarries may be located right by the breakwater structure or up to a distance of 35 km, but most commonly at a distance of 5-15 km. Core material can sometimes be produced more economically in poorer quality rock near the structure or dredged from the sea floor, but the armour stones for the outer layers of the breakwater will have to be produced from rock of better quality at a greater distance. Possible quarry yield and rock quality may have to be weighed against transportation distance in each case to optimise cost effectiveness and durability of the structure.

6 QUALITY ASSURANCE PROGRAMME

Breakwater rocks should be as dense as possible, preferably with dry specific gravity above 2.8, although densities down to about 2.7 may be acceptable. Good resistance to abrasion and freeze/thaw action has to be ensured for breakwaters in the arctic region and tolerance to other types of weathering such as sun burn has to be assured, especially in areas exposed to rapid temperature changes from -10°C up to +20°C in a few hours.

A quality assurance programme has been developed to guarantee that rocks intended for breakwaters are up to the required standard in each case. If rock of suitable quality is not available a breakwater may be designed with blocks larger that required by the design wave condition to take into account the predicted deterioration of the armour stone rock. This has been done successfully, especially in breakwaters that are not exposed to waves of a considerable height, but shielding harbours from local wind waves.
Visual inspection by experienced geologists and engineers is in our experience more valuable than most laboratory tests of rocks, although lab tests may be helpful to obtain data regarding density, water absorption and perhaps abrasion. Few rock types are highly resistive to abrasion and those who are tend to be brittle whereas softer rocks seem more elastic and less likely to break although they may wear down faster than the harder types in the laboratory tests. Inspection of thin sections of the rock is an important method to analyse rock type, rock texture (the inner binding of crystals), as well as alteration degree of the rock.

A quality assurance programme must identify the following parameters:
- rock type
- thickness of rock layers or rock formation
- alteration/oxidation colours
- rock texture
- degree of alteration
- porosity, vesicle types and shape
- cleavage, i.e. flow banding
- specific gravity
- water absorption
- possible grading of rock formation
- weight requirement for design criteria
- visible inspection for defects such as fatal fractures
- joints and fractures
- aspect ratio

7 PREDICTED QUARRY YIELD

Spacing of joints control the possible armour stone yield from a rock. Smarason (1994) has developed a simple method to predict quarry yield prior to blasting. Scanlines of exposed horizontal and vertical surfaces as well as core samples are measured and a frequency plots made for each scanline. An average is calculated and used to work out possible weight distribution by assuming that the stone produced will be of a cubic shape. Possible irregularities are worked out as well before quarry yield prediction is issued.

Figure 2 shows three examples of predicted quarry yield which in our experience may represent excellent, good and poor quarry yield. Quarry with a yield of ≥45% over 1 tonne and 15% over 5 tonnes is considered excellent, one with ≥25% over 1 tonne and 5-10% over 5 tonnes is considered good, whereas a quarry with a yield of ≥10% over 1 tonne and ≤2% over 5 tonnes is regarded poor.

Most breakwaters in Iceland have been constructed from quarries with good quarry yield of 25-35% over 1 tonne and 5-10% over 5 tonnes. Excellent size grades has only been achieved in a few projects so far.

Quarry yield estimates have been done as a routine for most breakwater projects over the past 10 years. These estimates have usually been so close to the actual outcome of blasting, that contractors are relying on them in their bidding for the projects. The quarry yield predictions can be used to assist the buyer and the contractor to monitor the blasting job and see in advance if the quarry is being as carefully blasted as possible to produce armour blocks of the right size and shape (aspect ratio).

A quarry yield prediction at Bolungavik in NW-Iceland, where a 200,000 m³ berm breakwater was constructed in 1992-1993, proved fairly accurate. The predicted yield over 1 tonne was 34%, whereas the actual over all yield over 1 tonne turned out to be 38% (Smarason, 1994).

Figure 2. Example of predicted quarry yield, three different size grades, excellent, good and poor.
8. DESIGN BASED ON ESTIMATED QUARRY YIELD

The estimated yield from an armour stone quarry is used as an integrated part of the design. The yield curves are used to define stone classes and with the total volume of the breakwater the yield curves give the total volume of each class available for the design. The optimization process is then to use all size grades from the quarry, to use the largest stone classes in the most critical areas.

Similar process has been described as a supply-based design approach, in contrast to the demand based design (Leeuwestein et al., 1995).

In several breakwater designs described in chapter 11 the quarry production of armour stones is fully utilized down to 1 tonne.

9. THE TAILOR-MADE SIZE GRADED BERM

The above described process has led to the “tailor-made size graded” berm breakwater, where several stone classes are used. In contrast to the homogeneous berm, the size graded berm is divided into several stone classes. The largest stones are used on top of the berm and some times also at the front. Under this layer is the main armour layer, which according to the yield curves is much larger in volume. If this design needs an overproduction of small stones and core, a secondary armour is often prescribed inside the berm, next to the core. This allows the use of even smaller stones, than in berm breakwaters of only one or two stone classes, better utilization of quarry and less expensive structure.

The Helguvik breakwater (see section 11.1) was designed after the original berm concept of mainly one stone class. The lower limit of the main stone class is 1.7 tonne. The construction of the Helguvik breakwater needed an overproduction of core material to secure sufficient volume of the main stone class. A berm structure is now under construction in Keflavik only 3 km from the Helguvik breakwater. The design wave is similar and the same quarry as for the Helguvik breakwater is used. The lower limit of the secondary armour is 0.3 tonne. The design of this structure aims at fully utilizing all stones down to 1 tonne and that there will be minimum of core material left in the quarry.

There are also other positive effects of splitting the stone mass into several classes. Compared to the original berm the outer most layers have a much narrower grading. This does not necessarily mean a larger void percentage, but at least the voids are larger and the waves propagate more easily into the berm.

10. DESIGN GUIDANCE FOR ARMOUR STONE SIZE

Coastal structures can be classified by the dimension less stability parameter $H_s/\Delta D_{50}$, where $H_s$ = the significant wave height, $\Delta$ = relative stone density and $D_{50}$ = characteristic diameter of armour units (rock or concrete) = $(W_{50}/p_5)^{1/3}$ where $W_{50}$ is the median stone weight and $p_5$ is the stone density. Practical cases from Juhl and Jensen, 1993, show that the parameter $H_s/\Delta D_{50}$ 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 $H_s/\Delta D_{50} = 3-6$.

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 $H_s/\Delta D_{50}$ 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 the use of 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.

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.

11. CASE STUDIES

11.1. The Helguvik Breakwater

The berm concept was introduced in Iceland through the design phase of the Helguvik breakwater (Baird and Hall, 1984). This was in the early eighties when the berm concept was formulated.

The breakwater shelters the Helguvik Bay for a tanker terminal close to the Keflavik NATO air base. It was built in 1986 -1988 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).

In the Helgvik Bay the 50 year design wave condition was estimated as a significant wave height $H_s=5.8$ 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 an equivalent wave $H_s=5.2$ m. The design water level was $+5.0$ m.

The design can be characterised by a wide berm of only one class of armour stones of 1.7 to 7.0 tonne stones with mean weight of 3.2 to 4.2 t and stability parameter, $H_s/\Delta D_{50}$ of 2.8 / 2.6 for the equivalent wave height.

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

![Figure 3. A cross section of the Helgvik breakwater.](image)

11.2 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 World War 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 $H_s=2.8$ m with $T_p$ of 18 seconds and wind waves up to $H_s=3.0$ 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 $H_s=3.8$ 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 Viggossen, 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$^4$ to 3*10$^5$ m$^3$/s for inconvenience to danger to personnel and 1*10$^4$ to 2*10$^3$ 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 (Viggossen & 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 \( \text{m}^3/\text{m} \) which equals \( 4.2 \times 10^5 \text{ m}^3/\text{m/sec} \).

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, Fig. 4.

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 \( H_s/\Delta D_{a0} \) 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.

![Figure 4. Akranes harbour. Cross section of the berm type rubble mound.](image)

**11.3. The Blonduós Breakwater**

Blonduós 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 225 m long rubble mound berm breakwater on the north side of the pier was constructed in 1993 to 1994, Fig. 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 \( H_s \) of 4.8 m and \( T_p \) 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 \( H_s/\Delta D_{a0} \) 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 m\(^3\) of 0.4 to 1 tonne stones, while only 11,000 m\(^3\) are needed in the design.
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 $/m² (including 24.5% VAT).

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

11.4. 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 $H_s$ of 6.3 m and wave peak period $T_p$ of 17 seconds. The design water level is +3.5 m. The layout and typical cross sections of the breakwater are shown in Fig. 6. There are three major stone classes with a main armour layer of 3 to 8 tonne stones giving a stability parameter $H_s/\Delta D_{50}$ of 3.1. Under the main armour layer stones of 1 to 3 tonne are used with $H_s/\Delta D_{50}$ of 4.3. In this design the main armour layer is thinner than for the Blonduns breakwater. At the breakwater head, 8 to 14 tonne stones are used on the top of the bern with $H_s/\Delta D_{50}$ 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.

![Diagram of breakwater](image)

**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.
11.5 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 m³, 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.

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 Hₜ of 5.9 m and Tₚ of 12 s and Hₜ 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 Fig. 7. There are two 15 m wide outer berm, 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.

Armour stone class I, which has an average stability parameter Hₜ/ΔDₕₐₜ of 1.64, is used on top of the upper berm at the breakwater head. Class II, which has Hₜ/ΔDₕₐₜ 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 Hₜ/ΔDₕₐₜ 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
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

\[
\frac{q}{\sqrt{gH'}} = 5.89 \times 10^{-3} \exp(7.67 \frac{R_{25}}{H'}) \tag{1}
\]

where:
\[
q \quad \text{= mean overtopping discharge (m}^3/\text{s/m)}
\]
\[
g \quad \text{= acceleration of gravity (m/s}^2)\]
\[
H' \quad \text{= significant wave height (m)}
\]
\[
R_{25} \quad \text{= 2% uprush level}
\]
\[
R_C \quad \text{= crest elevation freeboard (m)}
\]

Under design condition the criteria established by Viggosson and Sigurardarson, 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.

11.6 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_{50} of 2.66, and class III 0.5 - 6.0 t with an average weight of 1.5 t and stability parameter 3.35. Fig. 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.
12. 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 two items, better utilization of armour stone quarry and construction methods. All size grades from the quarry are used and in many cases a 100% utilization of the quarry yield has been achieved. Less specialized construction equipment can be used and easy land based construction procedures are quickly adopted by local contractors. The berm concept has also proved to be very efficient to reduce wave overtopping on exciting breakwaters.

The design aims at minimizing 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 Bakkafoordur there has been no deformation of the construction profiles. At Bakkafoordur on the other hand, with considerable reshaping, the berm concept makes it possible to build a breakwater of rock of very poor quality.

A quality assurance programme has been developed to guarantee that rocks intended for breakwaters are up to the required standard in each case. A quality assurance programme must identify various parameters:

The design process in Iceland has evolved from the original berm concept using one or two armour stone classes, to using four or even five classes to optimize the use of the quarried rock. A supply-based design approach aims at optimizing the structure with respect to possible quarry yield and wave load, which leads to the "tailor made size graded berm" breakwater, which in many cases is less expensive than the original berm concept.

So far the berm designs that have been carried out under various conditions, wave heights and periods, breaking/non-breaking waves and good or poor 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


