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WAVE HEIGHT LIMITS FOR THE STATICALLY STABLE ICELANDIC-TYPE BERM BREAKWATER

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The Icelandic-type berm breakwater has been developed through a number of breakwater projects over the past two decades for a design wave height up to Hs = 7.5 m. Some of the structures have experienced waves close to or even exceeding design wave conditions and reshaping has been within the design criteria. Since the year 2000 several projects have made use of extra large stones, a class of stones heavier than 15 to 20 tonnes. This has been made possible through reliable quarry yield prediction. Recently several projects have called for modification of the design for waves exceeding 8.0 m. The paper describes some recent breakwater projects in Iceland and Norway and a project that calls for a design for a wave height Hs = 9.2 m.

ICELANDIC-TYPE BERM BREAKWATER

Berm breakwaters have basically developed in two directions. On the one hand are the dynamic structures built using a few stone classes that are allowed to reshape. On the other hand are the more stable structures built of several stone classes, where limited profile reshaping is allowed. These structures have been 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 construction equipment, transport routes and required functions. These breakwaters are fairly simple to construct, usually they are built of locally quarried material and quarry yield prediction is used as a tool in the breakwater design procedure.

The Icelandic-type berm breakwater is built up of several narrowly graded armour classes with the larger classes placed at the most exposed locations within the breakwater cross section. These narrowly graded armour classes have a higher porosity than wider graded armour classes and therefore higher permeability which increases the stability of the structure. Taking advantage of this the Icelandic-type berm breakwater is a less voluminous structure than the dynamic reshaping berm breakwater. The Icelandic-type berm breakwater also provides a more efficient use of the quarry yield.

Although the Icelandic-type berm breakwater is constructed with several stone classes experience has shown that they are fairly simple to construct. That is reflected in the bidding prices for breakwater projects.

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RECENT BREAKWATER PROJECTS IN ICELAND AND NORWAY

Until now the Icelandic-type berm breakwaters have been designed for wave heights up to Hs = 7.5 m. Some of these structures have already experienced waves close to or even exceeding the design conditions. This is partly due to the fact that the frequency of storms at higher latitudes is much higher than at lower latitudes. Higher storm frequency means that breakwaters at higher latitudes encounter high wave conditions more frequently than breakwaters in lower latitudes.

The authors of this paper have been involved in a number of breakwater projects. A list of some of the more recent breakwater projects in Iceland and Norway follows with the construction period and design wave height for the most exposed section of the breakwater.

- Sirevåg berm breakwater, Norway, 2000 to 2001, Hs = 7.0 m.
- Húsavík berm breakwater, Iceland, 2001 to 2002, Hs = 6.8 m.
- Grindavík berm breakwater, Iceland, 2001 to 2002, Hs = 5.1 m.
- Hammerfest berm breakwater, Norway, 2002 to 2003, Hs = 7.5 m.
- Vopnaförður breakwater, Iceland, 2003 to 2004, Hs = 4.0 m.
- Þorlákshöfn berm breakwater, Iceland, 2004 to 2005, Hs = 5.7 m.

These projects are described in the text below.

Sirevåg berm breakwater

Sirevåg is a small fishing harbour located in a narrow bay on the southwest coast of Norway approximately 50 km south of the city of Stavanger. It is open to westerly waves. The area outside the harbour is an open coast with no reefs or shoals that give shelter from the waves. The wave height reducing effects are refraction and in shallow water also wave breaking.

The Sirevåg Berm Breakwater, completed in 2001, is designed for Hs = 7.0 m and Tp= 14.2 s (Sigurdarson et al. 2003). Four stone classes were used for the structure in addition to the core of quarry run, Table 1. The largest stone class is 20 to 30 tonnes with a mean weight above 23.3 tonnes corresponding to stability parameter Ho of 2.11 and HoTo of 48.

The rock armour is anorthositic gabbro of good quality with specific gravity (SSD) of 2.68. The main quarry was located on the northern side of the Sirevåg bay and the material was transported by a split barge to the breakwater. Two smaller quarries were located adjacent to the breakwater. The predicted 5.6% of stones in class I (20-30 tonnes) was easily achieved.

The breakwater has already experienced two storms where the wave height reached the design wave height for several hours, (Sigurdarson et al. 2003) and (Tørum et al. 2005). Measurements from a storm in January 2002 from a Waverider located only 450 m from the breakwater have been transferred to the breakwater site. They show that it was exposed to wave heights above Hs = 6.8 m for more than 3 hours with a maximum significant wave height during that period of Hs = 7.9 m. The reshaping during this storm was very modest. The intensity of the second storm in January 2005 has been estimated based on wave hindcasting correlated with measurements further away. During this storm the breakwater has been exposed to wave height of Hs ≈ 7.0 m for more than 6 hours continuously. During the second storm the reshaping continued but is still within the design criteria. The recession of the berm on the breakwater trunk is less than 2 stone diameters and less than 3 stone diameters on the breakwater head.

Table 1. Stone Classes and Quarry Yield Prediction for the Sirevåg breakwater

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%

Húsavík berm breakwater

The Húsavík harbour located on the northeast coast of Iceland is exposed to northerly waves. As the harbour entrance was rather wide wave agitation and ship movements in the harbour often exceeded the acceptable criteria. Several proposals were studied in a 3D physical model study. These included lengthening of the existing outer breakwater, which limited the size of ships entering the harbour. The chosen layout consists of a new 350 m long outer breakwater with a 130 m long quay with 10 m water depth.

The Húsavík Berm Breakwater is designed for Hs = 6.8 m and Tp = 15.5 s. The largest stone class is 16 to 30 tonnes with a mean weight of 20.7 tonnes corresponding to a stability parameter Ho of 1.9 and HoTo of 52. The rock type is basalt of good quality with specific gravity of 2.9. To get the best utilisation of the quarried material it was chosen to use 5 stone classes for the breakwater, Table 2. The total volume of the breakwater is about 275,000 m^3, about 140,000 m^3 of armourstones and 135,000 m^3 of quarry run.

A new armourstone quarry was opened for the project 25 km from the construction site, where all armourstones heavier than 1 tonne were quarried. Smaller armourstones and quarry run was quarried in the existing quarry at a distance of 5 km from the construction site. The quarry yield prediction proved to be fairly accurate and the contractor got a higher yield than predicted by avoiding the weaker and fractured zones in the quarry. The construction was completed in 2002.

Until now the structure has once experienced wave conditions close to the design conditions. No reshaping has occurred.

Table 2. Stone Classes and Quarry Yield Prediction for the Húsavík breakwater

Stone Class	w_{min}-w_{max} (tonnes)	w_{mean} (tonnes)	w_{max}/w_{min}	d_{max}/d_{min}	Expected quarry yield
I	16.0–30.0	20.7	1.9	1.23	5%
II	10.0–20.0	12	1.6	1.17	5%
III	4.0 – 10.0	6	2.5	1.36	9%
IV	1.0 – 4.0	2	4.0	1.59	14%
V	0.3 – 1.0	0.5	3.3	1.49	12%

Figure 1. From the construction of the Husavik berm breakwater. Surveying of class I armourstones, 16-30 tonnes, before building up the crest structure.

Grindavík berm breakwater

Grindavík is a fishing village on the southwest coast of Iceland. This coastline is one of the most exposed ones worldwide. The harbour is protected by rocky shallows. The entrance to the harbour used to be through a narrow channel with limited depth and two difficult bends or turns. Many fishing boat accidents occurred in the area, often with loss of lives. Also, there were severe limitations as to the utilisation of the harbour as larger ships had to sail in on high tide. This even led to ships bypassing the harbour altogether. After thorough physical model tests proposals were made for a new straight entrance channel with berm breakwaters on each side protecting the inner part so as to shorten the time ships were to be exposed to breaking waves. Dredging of the entrance channel was finished in the summer of 1999. Dredging was done through multilayer compound lava from a minimum depth of –2.5 to a finished channel depth of –9.5 m.

The most exposed part of the breakwaters is designed for Hs = 5.1 m and Tp = 18 s. The design utilizes four stone classes, Table 3, with the largest stone class 15 to 30 tonnes, with a mean weight of 20 tonnes, corresponding to a stability parameter Ho of 1.5 and HoTo of 47. As the heads of the two breakwaters are located very close to the navigational channel no movements of stones was allowed in this area. This explains the low stability parameter of the largest stone class, which is only used in the breakwater head.

The rock type is young Holocene basalt lava of good quality with a specific gravity of 2.85 (SSD). The western breakwater is 290 m long and the eastern one 310 m. The total volume of the two breakwaters is about 160,000 m^3, about 110,000 m^3 of armourstones and 50,000 m^3 of quarry run. The high percentage of armourstones in this project is due to the fact that the water depth under the large parts of the breakwaters is rather small, less than -2 m.

All material came from a new armourstone quarry that was opened for the project 2.5 and 5 km from the two breakwaters. The quarry yield prediction proved to be fairly accurate. Still the largest stone class did not come out as expected and some adjustment had to be made during construction. The less exposed areas of class I were replace with a lighter stone class, approximately 12 to 20 tonnes.

The quarry was worked down to about 4 m below groundwater level and the most competent part of the quarried rock lay within the lower half of the lava. The effect of the water on the blasted lower part of the bench height was underestimated in the yield prediction, resulting in a lower yield than predicted for the largest stone class. As there was room for design adjustments it was decided not to encourage the contractor to lower the water level in the quarry through pumping to obtain a higher fraction of class I.

The two breakwaters have on few occasions experienced severe storm situations but not very close to the design conditions. No reshaping has occurred.

Table 3. Stone Classes and Quarry Yield Prediction for the Grindavík breakwaters

Stone Class	W_{min}-W_{max} (tonnes)	W_{mean} (tonnes)	W_{max}/W_{min}	d_{max}/d_{min}	Expected quarry yield
I	15.0 – 30.0	20	2.0	1.26	5%
II	6.0 –15.0	9	2.5	1.36	9%
III	1.5 – 6.0	3	4.0	1.59	17%
V	0.3 – 1.5	0.7	5.0	1.71	20%

Hammerfest berm breakwater

The Hammerfest LNG plant was constructed on the island of Melkøya outside Hammerfest in Northern Norway to liquefy gas from the Snøhvit (Snow White) gas field in the Barents Sea for export by LNG/LPG vessels (Sigurdarson et al 2005). The island is located in a fjord and waves may only approach the site in a narrow sector centred about northern direction.

The initial design for a dynamically stable breakwater was changed to that of a statically stable, Icelandic-type berm breakwater. The Melkøya breakwater design was model tested in a scale of 1:100. The breakwater was designed to be statically stable and non-reshaping for the 100-year storm event. It should also withstand a wave height of the 1000-year return period without total damage. The 100-year storm event for the most exposed section of the breakwater is Hs = 7.5 m and Tp = 15.6 s. The largest stone class is 20 to 35 tonnes with a mean weight above 25 tonnes corresponding to a stability parameter Ho of 2.2 and HoTo of 57.

The rock type is banded gneiss of good quality with a specific gravity of 2.69. About 2,300,000 m^3 of solid rock was blasted for levelling of the island of Melkøya for the plant area and to construct a 900 m long berm breakwater to protect the plant. A geological report recommended quarrying about 1,500,000 m^3 for armourstone production. Quarry investigations based on core drillings instead of a test blasting as originally planned were used to estimate the possible yield from the quarrying. A quarry yield prediction for the project was expressed in three curves, minimum yield, average yield and maximum yield. The yield prediction for the largest stone class varied from 2.0 to 4.5% with an average yield of 3.5%. A design curve was based on the predicted yields and the contractor developed and adjusted the blasting designs with the goal of achieving fragmentation as close to the design curve as possible. The rock excavation, breakwater construction and levelling of the island was carried out in a 9 month period from July 2002 to April 2003.

The breakwater has yet to experience storms close to design conditions.

Table 4. Stone Classes and Quarry Yield Prediction for the Hammerfest breakwater

Stone Class	W_{min}-W_{max} (tonnes)	W_{mean} (tonnes)	W_{max}/ W_{min}	d_{max}/ d_{min}	Expt. average quarry yield
I	20.0–35.0	25	1.75	1.20	3.5%
II	10.0–20.0	13.3	2.0	1.26	4.5%
III	4.0 – 10.0	6.0	2.5	1.36	6.5%
IV	1.5 – 4.0	2.0	2.7	1.39	7.5%
V	0.5 – 1.5	0.8	3.0	1.44	9.0%

Vopnaförður breakwater

Vopnafjörður is a fishing village in northeast Iceland. The harbour is open to waves from the northeast. The harbour is partly sheltered by older breakwaters and islands. When the largest company in the village needed a new quay for large purse-seiners located in the least sheltered part of the harbour it was clear that a new breakwater had to be built. 3D physical model tests were undertaken and various proposals were tested. The outcome of the study was to close the gap between two small islands with a breakwater. Esthetical issues played an important role in the design of the breakwater. These included lowering of the breakwater from the original plans and a naturally curved alignment. The local population had strong opinions on the alignment of the breakwater and where and how it should connect to the islands at each end. About 7% of the population of the community, which is not only the fishing village but also the rural areas surrounding it, attended an open meeting in preparation of the project. The plans were accepted but the discussion continued through the execution phase and during construction of the breakwater some modifications had to be made.

The design wave height for the breakwater after being reduced due to obliquity is Hs = 4.0 m with Tp = 15 to 17 s. As the local armourstone quarry was known to give high yields of large stones the breakwater was designed as a modified two layer rubble mound structure. The modifications originated in the experience with design of berm breakwaters and had the aim to maximise the utilisation of quarried material. The structure is rather low, with crest elevation of +6.0, when the design water level is +2.5 m. Three stone classes were used for the project, Table 5. The primary armour layer, two layers of class I stones, extends from -2.0m up to the crest. The rear side is protected with two layers of class II stones and class II stones are also used to support the class I stones on the front side. Class I and II have stability parameters Ho of 1.3 and 1.9 respectively and HoTo of 40 and 68.

The breakwater is about 350 m long, mainly built on a water depth of 8 to 9 m CD, with a total volume of about 124,000 m^3, about 56,000 m^3 rock armour and 68,000 m^3 quarry run. The construction took place in 2003 and 2004.

Although yield predictions of the local armourstone quarries in porphyritic basalt of Tertiary age had already been carried out in previous projects the quarry had to be investigated through core drilling to ensure quantity of material and rock integrity.

The breakwater has on a few occasions been exposed to high waves but not of the magnitude of design conditions. No reshaping has occurred.

Table 5. Stone Classes and Quarry Yield Prediction for the Vopnafjörður breakwater

Stone Class	w_{min}-w_{max} (tonnes)	w_{mean} (tonnes)	w_{max}/w_{min}	d_{max}/d_{min}	Expected quarry yield
I	8.0 – 25.0	13.7	3.1	1.46	16%
II	3.0 – 8.0	4.7	2.7	1.39	15%
III	1.0 – 3.0	1.7	3.0	1.44	17%

Þorlákshöfn berm breakwater

Þorlákshöfn is located on the south-west coast of Iceland where the 400 km long sandy coast of southern Iceland meets the rocky coast of the Reykjanes peninsula. The development of the harbour started in the 1960's with the construction of two caisson breakwaters. After the volcanic eruption in the Westmann Islands south of Iceland in 1973 the harbour of Þorlákshöfn was expanded to accommodate a part of the fishing fleet of the Westmann Islands (Viggosson, 1990). The southern breakwater was extended by 200 m as a rubble mound structure and a new 380 m long rubble mound northern breakwater was constructed. The breakwaters were partly protected by armourstones 6 to 12 tonnes in weight and partly by 9 tonnes concrete dolos units. A new quarry was opened for the project close to the harbour. These are the only breakwaters in Iceland where concrete armour units are used. In total 2220 dolos units where placed between elevation -4 m and +8 m on the seaward side of the southern breakwater. On top of the doloses, from elevation +8 to the crest at +12 m, the breakwater was protected by two layers of 6 to 12 tonnes stones. The southern breakwater has been monitored since its construction and broken doloses counted and registered by production numbers. The total broken doloses in the first 25 years in service is less than 5% (Einarsson et al. 2002).

Since the extension of the harbour in 1976 several new quays have been constructed in the harbour. By replacing dissipating beaches with vertical steel sheet pile structures the wave agitation in the harbour has increased. This, as well as plans for new industry called for new studies of the harbour. Both numerical wave models and a physical 3D model were used for this purpose. Both models showed a prominent wave height gradient outside the southern breakwater. This meant that extensions of the breakwater did not increase the calmness in the harbour. Therefore it was decided to construct a new basin enclosed by a new northern breakwater with an opening through the old northern breakwater and to use the existing harbour entrance. To increase the stopping distance inside the harbour entrance a part of the caisson breakwater from the 1960's had to be removed. This was no loss as the service life-time of this part of the breakwater was near its end, located in the most exposed part of the harbour and with several holes from ship collisions.

The most exposed part of the breakwaters is designed for Hs = 5.7 m and Tp = 17 s. The design utilizes four stone classes, Table 6, with the largest stone class 8 to 25 tonnes, with a mean weight of 13.7 tonnes, corresponding to a stability parameter Ho of 1.9 and HoTo of 60.

A new armourstone quarry was opened for the project about 2 km from the construction site. The rock is an olivine tholeiite basalt lava of Holocene age consisting of several 1 – 12 m thick flow units. The quarry site was chosen through core drilling where the lava was found to have solidified as one 8 – 12 m thick layer. The specific gravity of the rock is 2.8. The rock is of good quality for breakwaters but fails a Los Angeles test for road construction. The yield prediction was easy to achieve during the construction and the contractor had to blast specifically for core material at times during the construction due to the high yield of the quarry.

The breakwater has not yet experienced storms close to design conditions. No reshaping has occurred.

Table 6. Stone Classes and Quarry Yield Prediction for the Þorlákshöfn breakwater

Stone Class	W_{min}-W_{max} (tonnes)	W_{mean} (tonnes)	W_{max}/W_{min}	d_{max}/d_{min}	Expected quarry yield
I	8.0 – 25.0	13.7	3.1	1.46	14%
II	3.0 – 8.0	4.7	2.7	1.39	13%
III	1.0 – 3.0	1.7	3.0	1.44	13%
IV	0.3 – 1.0	0.5	3.3	1.49	12%

DESIGNING FOR WAVES EXCEEDING Hs=8.0 M

In several projects that the authors have been involved in the design wave height has exceeded Hs = 8.0 m. Among them are several feasibility studies as well as the Laukvik breakwater which will be described later in the paper. The following sections will describe the possibilities for developing the Icelandic-type berm breakwater for these wave heights.

Use of larger stone

Recent breakwater projects in Iceland and Norway (Smarason 2005) have required a considerable quantity of 20 – 35 tonne rock in the most exposed areas of the rubble mound structures. Experience has shown that good quarries in basalt, gabbros and gneiss can yield 3–6 % in a stone class of this size relatively easily. The yield curves indicate that it might be possible to increase the percentage in this stone class considerably by extending it to include 50 tonne rocks, thus avoiding unnecessary breakage through splitting of the largest stones. Compared to the Hammerfest project this will increase the possible design wave height from 7.5 m to 8.0 m.

In the case of larger stones, larger and more specialized equipment is needed for sorting the stones from the blasting pile, transportation and placement on the breakwater. A stone class with stones up to 50 tonnes requires an excavator of 150 to 170 tonnes to place the stones instead of 110 to 120 tonne excavator for the 20 – 35 tonne stone class. This may increase the construction cost somewhat but insignificantly for a medium to large size project.

Another possibility is also to use more narrowly graded stone classes, as that increases the stability. This is, however, not practical when armourstones are acquired through quarrying as this reduces the quarry yield considerably. On the other hand when armour stones are acquired as a by-product from dimension stone quarries this could be considered.

Stones with higher density

One way to achieve this is to use high density rock armour, if available. If we assume that the positive effect of increased density follows the relative buoyant density Δ in the power of 3, then using a stone class of the same weight as in the Hammerfest project, but specific gravity of 3.1 t/m^3, instead of 2.69 t/m^3, we can design for a significant wave height of about 8.9 m if we assume the same stability parameter and for stone density of 3.0 a significant wave height of about 8.6 m.

On the other hand Helgason et al. (2005) showed that the theoretical assumption of lift and drag dominance giving power of 3.0 for the relative buoyant density (Δ) in most stability formulae seems not to hold for natural rock, at least not for a structure with a front slope of 1:1.5. This actually indicates that the positive effect of increased block density is overestimated in most stability formulae for the slope of 1:1.5. For a structure with a flatter slope the results show a fairly good coherence with the assumed power of 3.0 for the relative buoyant density (Δ) in most stability formulae.

Helgason et al. (2005) also states that the effect of increased density is correctly described by the Hudson formula (and the van der Meer formula) for slopes of 1:2.0 and most likely also for flatter slopes. It might be concluded that the power of Δ in armour stability formulae cannot generally be set to 3.0 as it depends on the slope angle and the type of armour.

Concrete armour units

Instead of reinforcing the berm with extra-large stones it is also possible to reinforce it with concrete armour units. The obvious question is why not use a fully conventional rubble mound structure protected with concrete elements. There could be several design criteria as overtopping, crest height and wave reflection. In all cases the statically stable Icelandic-type berm breakwater reinforced with stones or concrete armour units has an advantage over the conventional structure.

Similarly the Japanese horizontally composite breakwater with a berm of concrete blocks reduces the reflection and overtopping as well as the loading due to impacting waves. The composite-berm rubble mound breakwater of the Azores (Melby 2005) is an innovative structure, but it is different as it has a reshaping berm to support a traditional concrete armour layer.

Flatter front slope of the berm

Initially berm breakwaters were built with a steep lower slope, the front slope of the berm, even as steep as 1:1. These structures were allowed to reshape. With the development of the Icelandic-type berm breakwater the lower slope became more gentle and developed from 1:1.3 to 1:1.5. This has increased the stability of the berm. It is clear that the stability will still increase with gentler slope.

Bjørdal et al. (2004) tested an alternative design for the Laukvik breakwater where the designed profile is similar to the reshaped profile of a reshaping berm breakwater, the so-called S-profile. The profile had an upper slope of 1:1.5 and a lower slope of 1:4. No reshaping was observed on this profile in contrast to some reshaping on a profile of the same stone sizes and with upper and lower slope of 1:1.5. The disadvantage of this design is that it is much more difficult to construct.

Allow more reshaping

The statically stable Icelandic-type berm breakwater is a narrow and not a voluminous structure. The design criteria has been keep the stability number Ho below 2.0 in the regime of statically stable non-reshaping structures according to the PIANC (2003) guidelines. Still both Sirevåg and Hammerfest breakwaters have stability numbers above 2.0 or 2.1 and 2.2 respectively. We have already prototype experience from the Sirevåg breakwater that reshaping is less than design criteria after being hammered with the design wave height for 10 hours.

If we design for more reshaping the structure needs to be wider and more voluminous. The models presented by van der Meer (1988), Hall and Kao (1991) and Tørum et al. (2003) all relate characteristic profile parameters and hydraulic and structural parameters.

Tandem breakwater

It is possible to limit the wave height at the structure by a submerged reef or mound in front of the main breakwater called a tandem breakwater. As the waves break on the reef the wave impact on the main breakwater is reduced. The width of the reef and its distance from the main breakwater depend on the wave length. The effect of the reef diminishes with increased tidal difference. It is necessary to enable drainage of the reservoir between the reef and the main breakwater. Unless there are special requirements, as e.g. low crested structures, the tandem breakwater is not usually chosen as it needs usually more material to construct and is therefore more costly.

LAUKVIK BREAKWATER

Laukvik fishing harbour is located on an exposed location in Lofoten in northern Norway and is open to heavy waves from northwest, Bjørdal (2003) and Bjørdal et al. (2004). Construction of the Laukvik breakwater, which is the most exposed breakwater in Norway, started early in the sixties as a classic Norwegian rubble mound breakwater with a single layer of cover block. The blocks were well interlocked above the sea level. In the construction phase the breakwater was damaged several times or every year from 1968 to the completion in 1971. However, during a heavy storm from the northwest in 1972, the breakwater was severely damaged. A new design was chosen for the reconstruction in 1984, based on two concrete caissons covered by a step formed concrete superstructure down to the water level. Below the water level stone blocks supported the concrete cap. During the winter 1992/1993 the breakwater was damaged. It was repaired, but suffered a further damage under a storm of long duration in the winter 2001/2002. The damages included washing out and undermining of the caisson on the breakwater head. Temporary repairs after the 2002 storm included the bolting together of stones with steel bars, Figure 2.

Figure 2. Temporary repairs after the 2002 storm included the bolting together of stones with steel bars.

The design wave, at 20 m water depth outside the breakwater, has been established as Hs 100 = 9.2 m with Tp = 16 s in 2002. The waves will refract into the breakwater and break as plunging breakers. Based on model tests in 2002/2003 an Icelandic-type berm breakwater was introduced, built up of several layers of armour stones. The largest stone class was 20 – 30 tonnes rendering a stability parameter Ho = 2.2 and HoTo ≈ 70 (Bjørdal 2004).

After this design was proposed geological investigations of nearby quarries have been undertaken. A quarry in a monzonite rock of good to excellent quality with specific gravity of 2.72 is located within 2 km of the breakwater. The predicted yield over 1 tonne is about 40% and about 5% should fall in a 20 – 30 tonne class, whereas 10% could be obtained in an extra large stone class of 20 – 50 and 13% if the class was 20 – 60 tonnes, Figure 3.

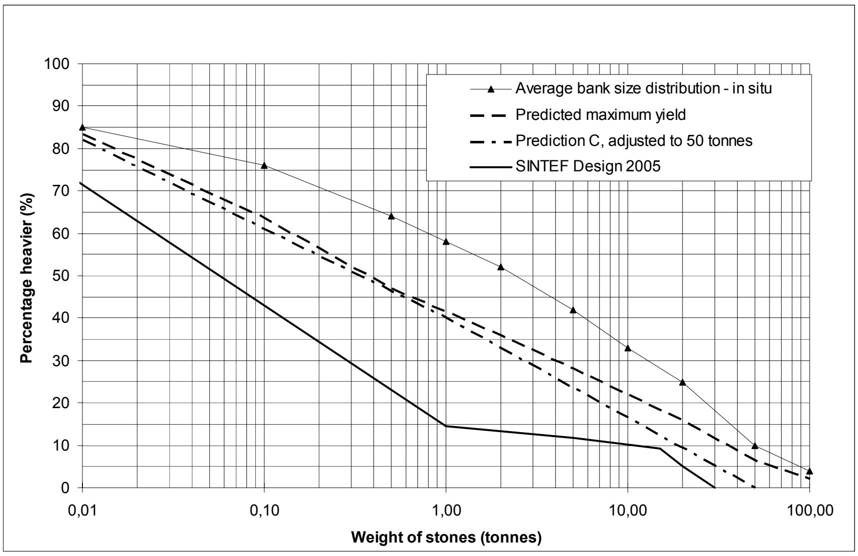

Figure 3. Quarry yield prediction for the proposed quarry for the Laukvik breakwater based on core drillings.

Based on this new information an alternative design was introduced where 30 – 40 tonnes blocks were placed on the breakwater head. However, the cost of this breakwater is presently considered too high for the small fishing port of Laukvik. Other possible, cheaper solutions have to be considered.

Calculations of the total costs, including repairs, will also be made to see if it will pay economically to repair weaker and cheaper designs and to accept more frequent repair work. This work is going on and reanalysing the wave conditions outside Laukvik is a part of this work. So far the design wave seems to increase up to 9.4 m in front of the breakwater.

CONCLUSION

Developed through a number of projects over the last two decades the Icelandic-type berm breakwater has been designed for wave height up to Hs = 7.5 m. Prototype experience exists where a breakwater has been exposed with the design wave for 10 hours with reshaping not exceeding design criteria. Further development above Hs = 8.0 m is possible through various adjustments depending on the site conditions. The challenge of an economical design for the Laukvik project where the design wave height is Hs = 9.2 to 9.4 still exists.

REFERNCES

Bjørdal, S., Vold, S, Viggosson, G. and Sigurdarson S. 2004. Design of Berm Breakwaters for Extreme Wave Conditions, *Proceedings of 29th International Conference on Coastal Engineering*, ASCE, 3740-3752.

Bjørdal, S. 2003. Laukvik Fishing Harbour, Rehabilitation of the Breakwater – Model Tests, *PIANC-AIPCN Bulletin No 113*.

Einarsson, S., Sigurdarson, S., Viggosson, G., Smarason, O.B. and Arnorsson, J., 2002. Berm Breakwaters – Design, Construction and Monitoring. Breakwaters '99, *Proceedings of an International Symposium on Monitoring Breakwaters*, ASCE.

Hall, K. and Kao, S. 1991. A study of the stability of dynamically stable breakwaters. *Canadian Journal of Civil Engineering, Vol. 18*, 916 – 925.

Helgason E. and Burcharth H.F. 2005. On the use of high-density rock in rubble mound breakwaters. *Proceedings of 2nd International Coastal Symposium in Iceland*, Icelandic Maritime Administration.

Melby, J.A. 2005. Composite-Berm Rubble Mound Breakwater. *Proceedings of Coastlines, Structures and Breakwaters 2005*, Institution of Civil Engineers, London, 520-529.

PIANC 2003. State of the Art of Designing and Constructing Berm Breakwaters. *Report of Working Group 40 of the Maritime Navigation Commission, International Navigation Association*, Brussels 2003.

Sigurdarson, S., Loftsson, A., Lothe, A.E., Bjertness, E. and Smarason, O.B. 2005. Berm Breakwater Protection for the Hammerfest LNG Plant in Norway - Design and Construction. *Proceedings of Coastlines, Structures and Breakwaters 2005*, Institution of Civil Engineers, London, 349-362.

Sigurdarson, S., Jacobsen, A., Smarason, O.B, Bjørdal, S, Viggosson, G, Urrang, C. and Torum, A. 2003. Sirevåg Berm Breakwater, design, construction and experience after design storm. *Coastal Structures 2003*, ASCE, 1212-1224.

Smarason, O.B. 2005. Optimum Quarry Yields for Extra Large Armourstone. *Proc of 2nd International Coastal Symposium*, Icelandic Maritime Admstr.

Tørum, A., Løset, S. and Myhra H. 2005. On berm breakwaters. Sirevåg berm breakwater – comparison between model and prototype reshaping. *Proceedings of 2nd International Coastal Symposium in Iceland*, Icelandic Maritime Administration.

Tørum, A., Kuhnen, F. and Menze, A. 2003. On berm breakwaters. Stability, scour, overtopping. *Coastal Engineering, 49*, 209 – 238.

Van der Meer, J. 1988. Rock slopes and gravel beaches under wave attack; *Doctoral thesis, Delft University of Technology*.

Viggosson, G., 1990. Rubble Mound Breakwaters in Iceland, *Journal of Coastal Research, SI 7*, pp.41-61.

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