Coastal Structures 2003
Portland, Oregon, ASCE

Sirevåg Berm Breakwater, design, construction and experience after design storm.

Sigurdur Sigurdarson[1],
Arnold Jacobsen[2],
Omar Bjarki Smarason[3],
Sverre Bjørdal[4],
Gisli Viggosson[1],
Christen Urrang[2] and
Alf Tørum[5]

Abstract

Sirevåg harbour is located in a narrow bay on the west coast of Norway approximately 50 km south of the city of Stavanger. 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. A new breakwater was constructed in Sirevåg, starting in January 2000 and completing in July 2001. The primary reason for the new breakwater was to give better protection of the harbour and to improve the sailing conditions in and out of the harbour.

The breakwater was designed and constructed as a statically stable Icelandic type berm breakwater for a wave height with a 100-year return period. The design 100-year recurrence wave height at the location of the breakwater was established as $H_{s,100}$ = 7.0 m. During the first winter in service the breakwater experienced a storm reaching the design level. The breakwater survived the storm without any reshaping. However stability model tests showed that there should have been a marked recession of the berm. The apparent discrepancy between the model test results and the field behaviour of the Sirevåg berm breakwater is discussed in this paper.

The construction cost for the Sirevåg berm breakwater proved to be considerably lower than reported from other projects. This is partly due to the availability of a suitable armourstone quarry and also to the maximisation of the quarry yield and the utilisation of all size grades from the quarry to the benefit the integity of the structure.

[1] Icelandic Maritime Administration, Kopavogur, Iceland, sigurdur@sigling.is
[2] Norwegian National Coastal Administration, 2nd District, Haugesund, Norway, arnold.jacobsen@kystverket.no
[3] Stapi Ltd. Consulting Geologists, Reykjavik, Iceland, stapi@xnet.is
[4] SINTEF Fisheries and Aquaculture, Trondheim, Norway, sverre.bjordal@civil.sintef.no
[5] Norwegian University of Science and Technology, Trondheim, Norway, alf.torum@bygg.ntnu.no

The statically Stable Berm Breakwater

Berm breakwaters are rubble mound structures tailored to suit specific conditions of each construction site using locally available rock. The volume of the structure and stone size can be varied to suit wave climate and each project has the aim of to maximise the utilisation of available local armour rock quarries.

A modification of the original berm breakwater has evolved into the statically stable berm breakwater. This structure is more stable than the original berm breakwater but at the same time less voluminous and is sometimes referred to as the “Icelandic-type”. It is built up of several size-graded layers, rather than just two, and the largest stone class is placed on the surface of the berm to reinforce the structure. The breakwater is designed to retain its integrity and only minor deformation of the berm is allowed under design conditions. Reshaping into an S-profile is not allowed but it is recognised that some deformation will occur with time as the result of repeated wave action.

Berm breakwaters may be divided into three categories (PIANC, 2003):

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

While a conventional rubble mound breakwater is required to be almost statically stable for the design wave conditions, berm breakwaters have different stability criteria. Traditionally berm breakwaters have been allowed to reshape, to a reshaped static stable or a reshaped dynamically stable profile, while only few stones are allowed to move on the “Icelandic type” berm breakwater. In Table 1 the two major categories of berm breakwaters are compared, the statically stable non-reshaping structure and dynamically stable reshaped structure.

Table 1. Comparison between dynamically stable reshaped BB
and statically stable non-reshaping BB.

Dynamically stable reshaped BB	Statically stable non-reshaping BB
Two stone classes	Several stone classes
Homogeneous berm	Berm of size-graded layers
Wide stone gradation	Narrow stone gradation
Low permeability	High permeability
Reshaping structures	Non-reshaping structures
Allowed erosion < berm width	Allowed recession < 2*Dn50
More voluminous	Less voluminous
No interlocking	Interlocking prescribed

The dynamically stable reshaped berm breakwater consists of only two stone classes, stones and quarry run, while the statically stable non-reshaping structure uses several stone classes. The advantage of using several stone classes is that the lighter grades can be used in specified places inside the structure. Instead of a wide stone gradation of the dynamically stable BB the statically stable non-reshaping BB has a narrow stone gradation, which means higher permeability and increased ability of the structure to absorb and swallow up the wave load on the structure. This means that the statically stable non-reshaping BB can be designed less voluminous than the dynamically stable BB.

Selection process for rubble mound structure type

The procedure followed by the designers of the Sirevåg breakwater when choosing the type of rubble mound structure can be described as follows:

1) Is it economical to design a conventional rubble mound structure following the van der Meer method. Check if all quarried material can be used in the project or sold to other projects.
2) Is it more economical to design a statically stable non-reshaping berm breakwater with the largest stone class similar to van der Meer criteria or with Ho up about 2.0. The demand for large stones is usually less in 2) than in 1). If there is a quarry available to dedicate to the project, then 2) is often more economical, usually for design wave height higher than Hs = 2 to 3 m.
3) If large stones, compared to design wave height, are not available, then go to a wider and more voluminous berm breakwater, of the statically stable reshaped type.
4) If 1) to 3) are not possible options, then check out a still wider and more voluminous berm breakwater design of a dynamically stable structure. This could be a suitable structure for a trunck section connecting an island to the shore, but is not suitable for a head section.

Quarry yield prediction as a tool at design stage

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

Often the engineer/designer has to rely on contractors or quarry operators regarding information on possible maximum quarry yields or the sizes of the largest stones obtainable from available quarries. These estimates are very often biased by the size of equipment the contractors or quarry operators are using or the actual requirement for stone sizes in previous projects. It seems to be commonly accepted that quarries only yield up to 6 to 8 tonne stones. Dedicated armourstone production is not common and therefore there are not many contractors who have much experience in this field. Guidelines for blasting for armour stones are insufficient and only a few contractors have much experience in drilling and blasting for breakwater construction. This is, however, gradually improving. Contractors are gaining experience in obtaining stone classes to the requested specifications and an increasing number of contractors are now familiar with the quarry yield prediction curves and rely on them in their tenders.

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

Table 2. Quarry yield prediction for some recent breakwater projects in Iceland, India, Norway and South Africa.

Breakwater site	Country	Rock type	Predicted Quarry Yield			Volume
			>20 t	>10 t	>5 t	(m^3)
Bolungarvik	Iceland	porphyritic basalt	2	5	11	265,000
Blonduos	Iceland	porphyritic basalt	4	9	14	100,000
Hammerfest	Norway	gneiss	4,4	10	15	3,000,000
Hornafjördur south	Iceland	porphyritic basalt	2-5	5-10	15-20	60,000
Hornafjördur east	Iceland	gabbro	5-10	10-15	15-20	100,000
Husavik	Iceland	porphyritic basalt	3-4	7-10	12-16	300,000
Karwar	India	Granite, dolerite	4	7	13	4,000,000
Coega	SouthAfrica	Quartzite	5	8	15	3,400,000
Sirevåg	Norway	anorthosite gabbro	15-17	22-25	30-33	640,000
Vopnafjördur	Iceland	porphyritic basalt	10-20	20-30	30-40	40,000

Sirevåg Berm Breakwater

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

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

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

In the preliminary design, three sets of stone classes were considered. One set was chosen based on the overall utilisation of all quarried material according to a preliminary quarry yield prediction and fulfilment of stability criteria for all sections of the breakwater (table 3).

Table 3. Stone Classes and Quarry Yield Prediction.

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

The geological investigation and quarry yield prediction included drilling of 25 cored drill holes and surface scan-lines. Three possible quarries (A, B and C) were assessed for the Sirevåg breakwater. A quarry yield prediction was carried out for the three quarries for a 640,000 m^3 breakwater. The armourstone material is anorthosite gabbro rock of good quality with specific gravity (SSD) of 2.7 and water absorption between 0.2 and 0.3. The point load index exceeds 10 MPa. The quarry yield prediction, Figure 1, for a carefully worked quarry is about 50% over 1 tonne, about 30% over 3 tonnes and about 15% over 10 tonnes. This will result in about 6% in stone class I, 20 to 30 tonnes, 10% in stone class II, 10 to 20 tonnes, 14% in class III, 4 to 10 tonnes, and 19% in class IV, 1 to 4 tonnes, Table 3.

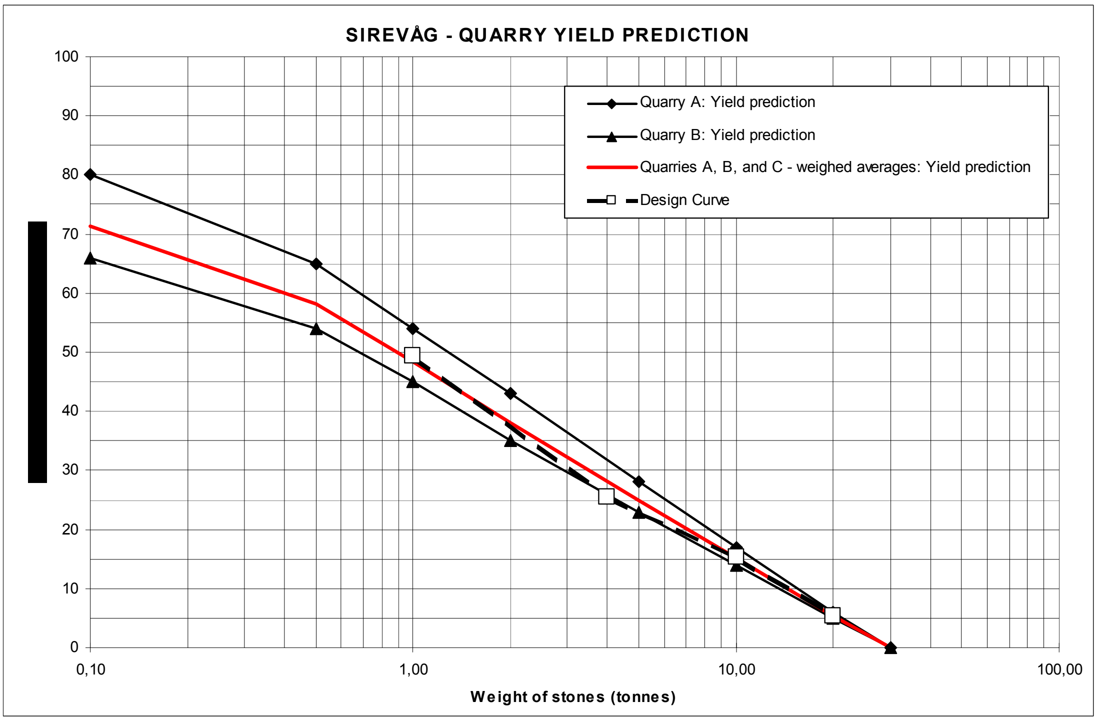

Figure 1. Quarry yield prediction and design curve for the Sirevåg breakwater.

A cross section of the outer part of the breakwater is shown in Figure 2. The design fully utilises all quarried stones over 1 tonne and a 100% utilisation of all quarried material was expected for the project.

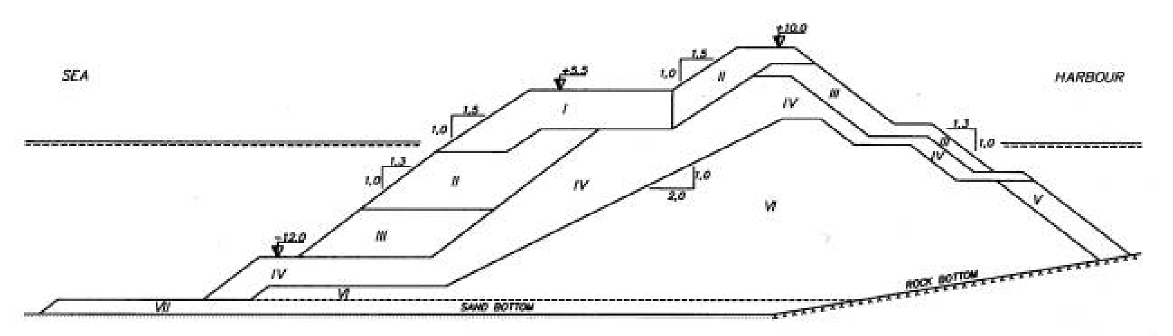

Figure 2. Sirevåg berm breakwater, cross section of the outer part.

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

The conversion from Norwegian krones to US dollars may have been done in a time of strong dollar and weak krone. Today we may have to add up to 30% on top of the USD prices. Then the overall construction cost per length metre of structure is about 22,000 USD for a structure on 20 m water depth designed for Hs = 7.0 m.

The construction of the breakwater started in January 2000 and was completed in July 2001, 3 months ahead of schedule. There were no claims from the contractor regarding contracts documents and quarry report.

The construction cost of the Sirevåg breakwater should be compared to cost figures reported in Alfageme et al. 2003, which are about six times higher. Of course this high difference in construction costs depends highly on the available armourstone quarry, but also on maximising the quarry yield and utilisation of all size grades from the quarry to benefit of the integrity and overall economics of the structure.

Monitoring program

A simple and economical monitoring program was prescribed for the Sirevåg breakwater. At the end of the construction reference points, bolts, were placed at 10 m interval on the centreline of the breakwater, each with a station number marked on it. The purpose was that it would allow an inspector to describe possible small reshaping with reference to a station number. Surveying the breakwater is not considered necessary until major reshaping or damage has taken place. In addition to the reference points along the centreline, points were placed on 30 to 50 meters interval for measuring possible settlements of the breakwater.

Station number 0 is at the landward end of the breakwater, while station 500 is at the head of the breakwater. The first 100 m of the breakwater, station 0 to 100, are built on land.

The storm of January 28 - 29, 2002

The Sirevåg breakwater was hit by a severe storm on January 28, 2002, only 6 months after it was finished. A Waverider buoy located 450 m off the breakwater head at 20 m water depth measured wave heights at half hour intervals, Figure 3. The maximum recorded significant wave height was Hs = 9.3 m and the wave height exceeded Hs = 8.0 m for a period of 3 hours.

The results from refraction computations indicate that the significant wave height at the breakwater head is about 88% of the wave height at the wave gauge. Some reflection can be expected from the breakwater trunk and adjacent rocky shore and it will affect the measured waves. In the analysis of the storm it has been estimated that the reflection can be of the order 20%, which means that the measured waves have to be reduced by further 3% down to 85% to represent the wave height at the breakwater. This means that the breakwater has been exposed to a maximum of Hs = 7.9 m and Hs = 6.8 m for a period of 3 hours, which is close to the design wave conditions of Hs = 7.0 m.

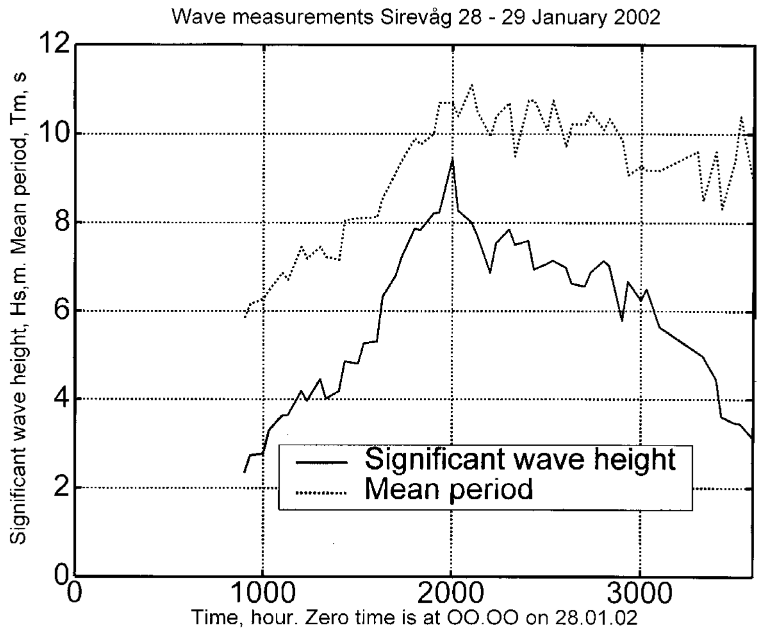

Figure 3. Wave measurements Sirevåg 28/29 January 2002. Significant wave height and mean period is given at the start of each 30 min recording, e.g. the values given at 2000 is for the time period 20:00 – 20:30.

Stability scale model tests of the Sirevåg breakwater

Stability model tests of the Sirevåg berm breakwater were not originally planned. But such tests were undertaken at SINTEF/NTNU as a diploma thesis study of students from the University of Braunschweig, Germany, (Tørum et al. 2003a), and the test results were placed in a wider context. These tests were undertaken during the construction of the breakwater.

The model tests were carried out in a wave basin in the scale 1:70. The width of the basin is 5 m and the length 40 m. The distance from the wave paddle to the model breakwater was approximately 28 m. Different stone classes were marked with different colours for better observation of motions and displacements of the stones.

The model was built on a plateau in the wave flume. From this plateau, the bottom sloped down to a maximum water depth in the flume corresponding to approximately 60 m water depth in prototype. The slope was 1:10 on the upper part and 1:30 on the lower part of the slope. The bottom slope in the prototype outside the breakwater is in the range 1:100 to 1:150. Sand was placed around the breakwater model in order to study scour and scour protection.

Stone classes I – III were scaled exactly with respect to mass, while class IV stones were retained in standard sieves and deviated slightly, 0,6 to 5,5 tonnes instead of 1 to 4 tonnes in prototype.

The model breakwater followed the drawings and design specification of the Sirevåg berm breakwater with one exception. The stones on the top of and at the front of the berm above elevation –1.0 m were placed pell-mell in the model, instead of an orderly placement that was specified in the design.

Expected reshaping of the prototype based on the model test results

As a bases for estimation of the prototype breakwater reshaping based on the model test results, HoTo has been plotted both for the model and prototype breakwater as shown in Figure 4, where $Ho=Hs/\Delta D_{n50}$ and $To=Tz\ (g/D_{n50})^{1/2}$. The HoTo values that gave mean values of Rec/D_{n50} = 1, 2 and 3 from the model tests are also shown. Based on this observation it is expected hat the apparent recession of the prototype would be in the range Rec/D_{n50} = 1 – 2 or Rec = 2 – 4 m.

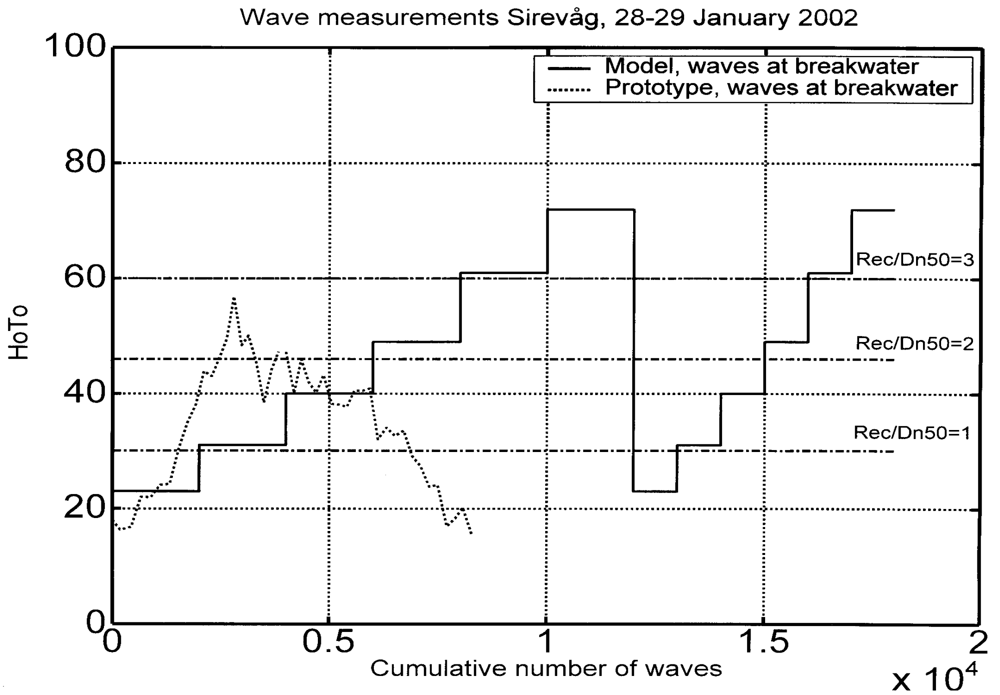

Figure 4. HoTo vs. cumulative number of waves, model and prototype. The shown recessions Rec/Dn50 = 1, 2 and 3 for given values of HoTo are mean values, 2nd order polynomial fit, taken from the test results.

Survey of the Sirevåg breakwater after the storm of 28 - 29 January 2002

Just after the storm of 28 – 29 January 2002 the breakwater was surveyed and photographed by Christen Urrang who was the chief site engineer of the Norwegian Coastal Administration during the construction of the breakwater.

The survey started at station 50, the surveyor followed the berm out the breakwater to the breakwater head, around the head and back on top of the crest structure. The survey took 3 hours. The following was reported:

- **Station 120 to 135:** 4 flat stones on berm lifted up to an upright position.
- **Station 180 to 200:** 2 flat stones on berm lifted up to an upright position.
- **Station 285:** 1 stone on berm lifted out of its position and moved inwards a distance of 3 – 4 m.
- **Station 345:** 1 stone on berm lifted out of its position and moved inwards a distance of 5 – 6 m.
- **Station 370:** 1 stone lost from the front slope. It has apparently been lifted and moved outwards and into the sea.
- **Station 380:** 2 stones lost from the front slope. They have apparently been lifted and moved outwards and into the sea.
- **Station 430:** There is a hole in the front slope. Some 1 or 2 stones have been lost.

There has been no survey or profiling of the Sirevåg berm breakwater after the storm of 28 – 29 January 2002.
In March 2003 Sverre Bjørdal of SINTEF Fisheries and Aquaculture inspected the breakwater both from land and the outer side on a boat from sea. The weather during the inspection was almost calm with very good visibility and hardly any waves. The following observations were made during the inspection:

- **Observations on the rear side and the crest.** There were no signs of any movement of the stones on the rear side of the breakwater. There were no irregularities of the stones on the breakwater crest, indicating that there had been no movements of stones. Local people reported that wave overtopping occurs in bad weather, but the overtopping water masses disappear easily into the stone layers. Biggest overtopping occurs some distance from the tip of the breakwater where the crest elevation has been lowered from + 10.00 m to +8.00 m.

- **Observations on the breakwater head.** There were no observed changes of the blocks on the berm or on the outer slope except at Area 1, where 4-5 cover blocks at the still water line had been removed. The cover stones were orderly placed and apparently well placed. The cover stones above those, which had been removed, were still in their original position. It was not possible to see whether Class II stones below – 1.0 m had been washed out. The Class II stones form the support for the Class I stones. (Area 1 has been localised from figure on the outward side of the breakwater head, 50 – 60° from normal to the trunk)

- **Observations on the outer side of the breakwater.** It was observed that in two areas, Area 2 and Area 3, some Class I blocks were removed at the still water line. The extension along the breakwater of each of these areas was 5-6 stone diameters (10-12 m). A couple of other places 1-2 stones had been removed at the still water line. The reshaping of Areas 2 and 3 seemed to be “deeper” than of Area 1, and the slope from the still water line was steeper than “normal”. How the slope is under – 1.00 m is not known, but it may be assumed that some of the Class II stones have also been removed. (Area 2 has been localised from figure between stations 380 and 400 and area 3 between stations 430 and 450).

- **Observations on the top of the berm.** Some stones had moved slightly. However, there were no signs of reshaping of the berm.

Discrepancy between the model and prototype reshaping

There is an apparent discrepancy between the model test results and the results of the prototype inspection with respect to reshaping of the berm. The reshaping is less in the prototype than might have been expected from the model test results. Possible reasons for this apparent discrepancy are discussed in the following, item by item (Tørum et al. 2003b).

Scale effects. The Sirevåg breakwater model was modelled according to the Froude model law, which assumes that the gravity force and the acceleration force are dominant forces for the fluid motion and the forces on the armour stones. However, viscous forces may play a more important role for the fluid motion and the forces in the model than in the prototype. For a Reynolds number of $2.3x10^4$ for the largest stone class of the Sirevåg breakwater, it is on the other hand unlikely to cause scale effects on the test results. Scale effects are not considered to be a major reason for the apparent discrepancy.

Model effects. Model effects are caused by the fact that a physical model is not a full replica of nature, i.e. the wave boundary conditions are different in the model from in the prototype breakwater etc. Although there is a big scatter in the results of rubble mound breakwater testing, the apparent smaller recession observed for the prototype can probably not be explained by these uncertainties.

The construction methods of the class I stones in the model and the prototype were, on the other hand, different. In the model, the armourstones on the berm were placed randomly in contrast to orderly placement in the prototype. This might be the major reason for the apparent less reshaping in the prototype than in the model.

Storm duration. The duration of the January 2002 storm was shorter than the storm duration during the model tests. Still the shorter storm duration in prototype is taken into consideration when the test results are interpreted and a recession of close to $2*D_{n50}$ is expected.

Long-crested vs. short-crested waves. The Sirevåg berm breakwater tests were carried out with long-crested waves. The waves in the prototype are, on the other hand, short-crested in open sea but are believed to have become almost long-crested close to the Sirevåg berm breakwater due to refraction effects. Therefore, the less reshaping of in the prototype is not believed to be influenced by the apparent small spreading of the directional waves in prototype.

Comparison of wave parameters in prototype and model. Possible effect of a variation in different wave parameters are discussed in (Tørum et al. 2003b). These are wave sampling variability, wave steepness, ratios $H_{1/100}/H_s$ and $H_{1/20}H_s$, skewness and at last the accuracy of the Waverider buoy. Some differences are found in these parameters, still they are not believed to be a major contributor to the discrepancy in the reshaping.

Comparison Between Prototype and Model Scale Breakwater

The expected reshaping of the prototype breakwater based on the model test result is evaluated based on the cumulative number of waves it has experienced. The apparent recession of the prototype breakwater should be in the range Rec/Dn50 = 1 – 2 or Rec = 2 – 4 m. Inspection of the prototype breakwater showed very little reshaping, except in few places where a stone or two had been removed from the edge of the berm.

Possible reasons for this apparent discrepancy can be several, but it is believed that the main reason is the difference in the construction methods, the placement of armourstones on the berm. A program has been proposed to investigate the influence of placing methods of armourstones on the berm, orderly placed versus pell-mell.

Conclusion

The Sirevåg berm breakwater is 640,000 m^3 and was constructed in 2000 to 2003. It was exposed to a heavy storm on 28-29th of January 2002, matching the 100-year design wave height criteria. There was almost no reshaping of the berm while site specific model test results and results of many more general test series on the reshaping of a berm breakwater indicated that the recession should have been 2 – 4 m. In the prototype, orderly placement and interlocking was specified in the design, while the stones on the berm were placed randomly in the model. This is believed to be the main reason for this apparent discrepancy.

It has been shown in many recent projects in Iceland and Norway that large and extra large stones can be produced with proper blast design from various rock types. Presently, international breakwater projects are being put in the construction phase without proper quarry investigations, often leading to the use of relatively more expensive concrete elements and not utilising the yield potential of large stones from locally available quarries.

Mutual understanding between the quarry geologist, designer and the engineer's representative during preparation and design phases and the contractor during the construction phase is vital for a successful execution of a breakwater project.

The statically stable berm breakwater has been shown to be an economical construction fulfilling the design criteria in one of the world's most hostile wave environments of the North Atlantic.

References

Alfageme, S., Headland, J. R., Kotulak, P. W. and Smith, E. D. (2004). "Breakopt: A Simulation System for the Probabilistic Optimization of Rubble Mound and Vertical Breakwaters". In: *Coastal Structures 2003*, ASCE.

Lamberti, A. and Tomasicchio, G.R. (1997). "Stone mobility and longshore transport at reshaping breakwaters". Coastal Engineering, Elsevier Science, vol. 29, pp 263-289.

Mathiesen, M. (1999): Sirevåg Harbour. Wave analysis. SINTEF Report No. STF60 F95029. (in Norwegian).

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., Viggosson, G., Torum, A. and Smarason O.B. (2001). "Stable berm breakwaters". In: *International Workshop on Advanced Design of Maritime Structures in the 21st Century*, Ed.: Y. Goda and S. Takahashi. PHRI, Japan, pp 152-159.

Smarason, O.B., Sigurdarson, S. and Viggosson G. (2000). "Quarry yield prediction as a tool in breakwater design". *Keynote lectures NGM-2000*. Finnish Geotechnical Society.

Tørum, A., Kohen, F. and Menze, A. (2003a). "On berm breakwaters. Stability, scour, overtopping." *Coastal Engineering*, Vol 49/3, pp 209 – 238.

Tørum, A., Bjørdal, S., Mathiesen, M. and Arntsen, Ø.A. (2003b). *The Sirevåg berm breakwater. Comparison between physical model and prototype behaviour*. Report No. IBAT/MB R1, Norwegian University of Science and Technology, Department of Civil and Transport Engineering. Prepared for the National Norwegian Coastal Administration.