THE ICELANDIC-TYPE BERM BREAKWATER FOR LARGE DESIGN WAVE HEIGHTS

by

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ABSTRACT

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 H_s = 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 significant wave height of and exceeding 8.0 m. The paper describes some recent breakwater projects in Iceland and Norway and a feasibility study for a breakwater with a design wave height H_s = 8.0 m. A cost comparison will be presented for three alternatives, an Icelandic-type berm breakwater using two different armourstone quarries and an Xbloc breakwater. The paper also shows results of safety optimization calculations for mild depth limited wave climate and for a situation at deep water.

1. THE ICELANDIC-TYPE BERM BREAKWATER

Berm breakwaters have basically developed in two directions. On one hand are the structures built using a homogenous berm, usually of one stone class, that are allowed to reshape, sometimes referred as dynamic berm breakwaters. On the other hand are the more stable structures with steep upper and lower slopes, built of several stone classes, where only a few stones on the berm are allowed to move. 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 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, which is a “tough” structure, has proved to be a cost and technically efficient structure, if suitable rock is available.

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.

2. WAVE HEIGHT LIMITS

Until now the Icelandic-type berm breakwaters have been designed for wave heights up to H_s = 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 at lower latitudes.

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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, see Sigurdarson et al. 2003, 2005a, 2005b and 2006.

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

3. OPTIMUM SAFETY LEVELS

In order to come to optimum safety levels for breakwaters a procedure has been followed in numerical simulation for identification of minimum cost safety levels, Sigurdarson et al. (2007). Before such a numerical simulation can be performed, design rules should be available and also a description of the behaviour of the structure under (very) extreme wave conditions. The mentioned procedure of numerical simulation gives amongst others the following items:

- Design structure geometries by conventional deterministic methods, corresponding to various chosen design wave heights;
- Definition of repair policy and related cost of repair;
- Definition of a model for damage accumulation and consequences of complete failure.

The objective is to identify the most economical safety levels over the lifetime of the structures. The procedure is to calculate the lifetime cost of a number of structures, which are deterministically designed to different safety levels and to identify the safety level corresponding to the lowest cost. The optimisation was performed with Monte Carlo simulations. The failure modes considered are the recession of the front of the berm and the rear side erosion (Van der Meer and Veldman, 1992). Three limit states are considered:

- Serviceable limit state (SLS) corresponds to the limit of damage not affecting the function of the breakwater.
- Repairable limit state (RLS) corresponds to moderate damage.
- Ultimate limit state (ULS) corresponds to very severe damage.

Two cases are considered, a shallow water case with 11 m water depth and a deep water case with 20 m water depth. Only the shallow water case will be presented here.

Results of shallow water case

The results of the cost optimization simulations for the shallow water case are shown in Figure 1. The total cost as a function of the design return period is given for various design stability numbers H_o, where $H_o = H_s/\Delta D_{n50}$, with H_s = significant wave height, Δ = relative mass density and D_{n50} = nominal diameter.

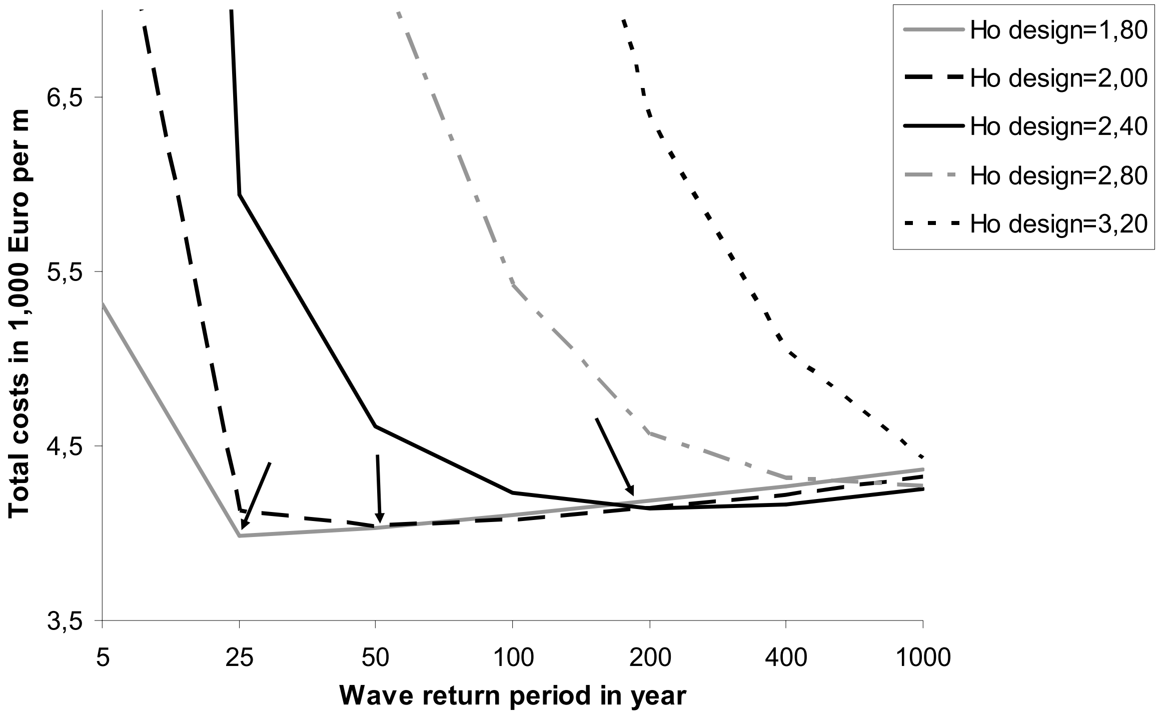


Figure 1. Shallow water case, total cost as a function of design return period for various design stability numbers. The arrows point to the minimum values.

The optimum safety level has a flat minimum towards higher return periods, but rather steep increase in cost towards the lower return periods. Design for a low stability number is more economical than to design for a high stability number. The most economical design corresponds to H_o^{design} = 1.8 and a design return period of 25 years. But as the minimum is very flat, there is only 3% increase in total cost if designed for 100 years return period instead of 25 years. For the stability number of H_o^{design} = 2.0 the design return period of 50 years is the most economical, but there is only a 3% increase in total cost if designed for 200 years return period.

Results of deep water case

The results for the deep water case are given in Figure 2. This graph shows the same characteristics as for the shallow water case with flat minimum towards the higher stability numbers and steep increase towards the lower stability numbers. The most economical design corresponds to H_o^{design} = 1.8 for 100 years return period and H_o^{design} = 2.0 for 200 years return period.

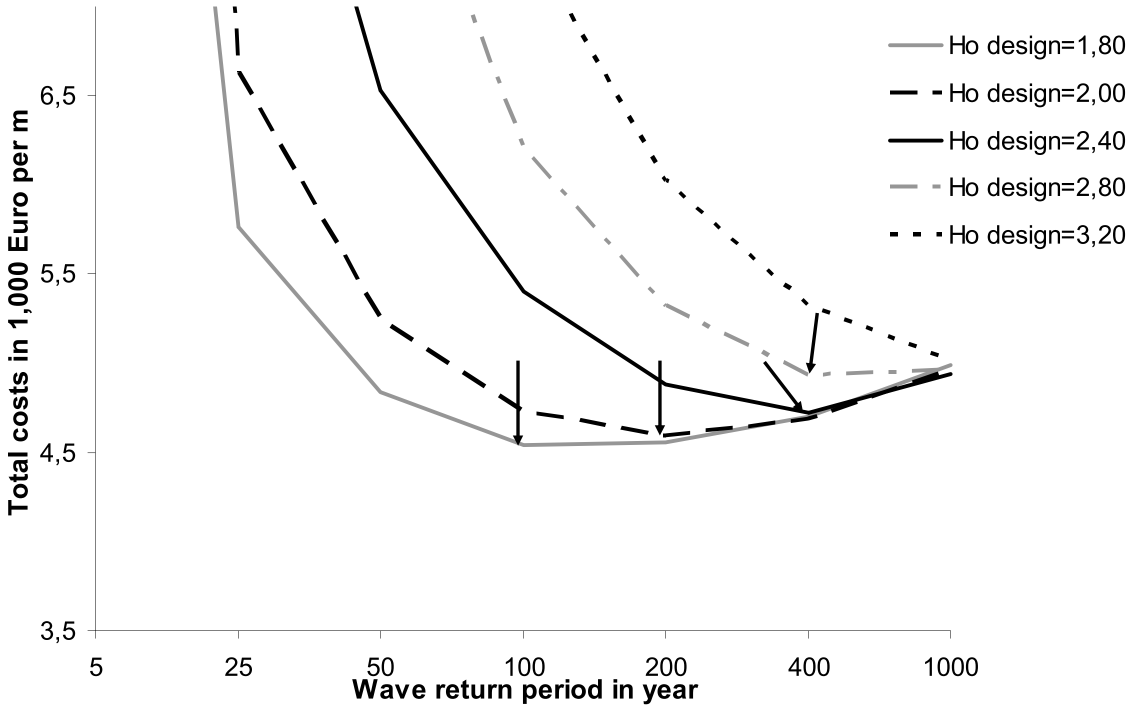


Figure 2. Deep water case, total cost as a function of design return period for various design stability numbers. The arrows point to the minimum values.

A comparison of Figures 1 and 2 indicates that the shallow water case is less expensive than the deep water case, which can be explained by the higher probability of extreme wave heights in the deep water case. The optimum safety level is reached for lower return period for the shallow water case than for the deep water case. But the assumption has to be noticed, that in the calculations all rock sizes are available. For the shallow water case with 50 years return period the stability number of 2.0 corresponds to a mean weight of class I stones of 12 tonnes, while for the deep water case with 200 years return period the stability number of 2.0 corresponds to a mean weight of class I stones of 25 tonnes.

Comparison with the design of recently constructed Icelandic-type berm breakwaters

Table 1 lists some recent berm breakwater projects in Iceland and Norway (Sigurdarson et al. 2006). In all cases the breakwaters have been designed for a wave height with 100 years return period. In four out of six cases the stability number of the largest stone class has been close to 2.0, or in the range H_o = 1.9 to 2.2.

Breakwater project	Design return period (y)	Design wave H_s (m)	Design water depth (m)	Class I (tonnes)	Design H_o
Sirevåg	100	7.0	19	20 – 30	2.1
Húsavík	100	6.8	13	16 – 30	1.9
Grindavík*	100	5.1	10	15 – 30	2.0
Hammerfest	100	7.5	25	20 – 35	2.2
Thorlákshöfn	100	5.7	9	8 - 25	1.9

* Class I stones on the Grindavik breakwaters are only used on a limited part of the breakwater heads. The data here corresponds to class II stones used on the most exposed trunk section.

Table 1. List of recently constructed Icelandic-type berm breakwaters.

It is important to be aware of that the design return period and the stability number are dependent variables. From the results of the safety optimization calculatins it can be concluded that for the depth limited case the wave height with 100 years return period is about 5% higher than for 50 years return period and about 11% higher than for 25 year return period. If the Sirevåg berm breakwater had been designed for waves with 50 or 25 years return periods instead of 100 years, but with unchanged stone classes, the stability number of the largest stone class would be 2.0 and 1.9 respectively. That is very close to the optimum safety level for the shallow water case with H_o^{design} = 1.8 and design return period of 25 years. For the Hammerfest case, on the other hand, which is a more deep water case it was difficult to come closer to the optimum safety level of H_o^{design} = 1.8 for 100 years return period as that would have needed a stone class with a mean weight of 46 tonnes instead of the 25 tonnes that were used. However, the simulations are not valid for such a case because the unit price for such big rocks is underestimated in the simulations.

It can be concluded that the design of these recent breakwaters follows the general recommendation drawn from the safety optimization calculations. With the required stone sizes available it has been possible to design the berm breakwaters with low stability numbers close to the optimum safety levels.

4. HÚSAVÍK BERM BREAKWATER COMPLETED IN 2002

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 capable of entering the harbour. The chosen layout consisted of a new 350 m long outer breakwater with a 130 m long quay with 10 m water depth.

The Húsavík Berm Breakwater was designed for H_s = 6.8 m and T_p = 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 H_o of 1.9 and H_oT_o of 52 (where $T_o = T_m\,(g/D_{50})^{0.5}$ with T_m = mean period). The rock type is basalt of good quality with specific gravity of 2.9. To get the best utilisation of the quarried material, a decision was taken to use 5 stone classes for the breakwater, see 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, located 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 achieved a higher yield than predicted, by avoiding the weaker and fractured zones in the quarry. The largest stone class was 16 to 30 tonnes with a mean weight of 20.7 tonnes. The construction was completed in 2002. Until now the structure has once experienced wave conditions close to the design conditions. No reshaping has occurred.

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%

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

5. A NEW BERM BREAKWATER AT HÚSAVÍK

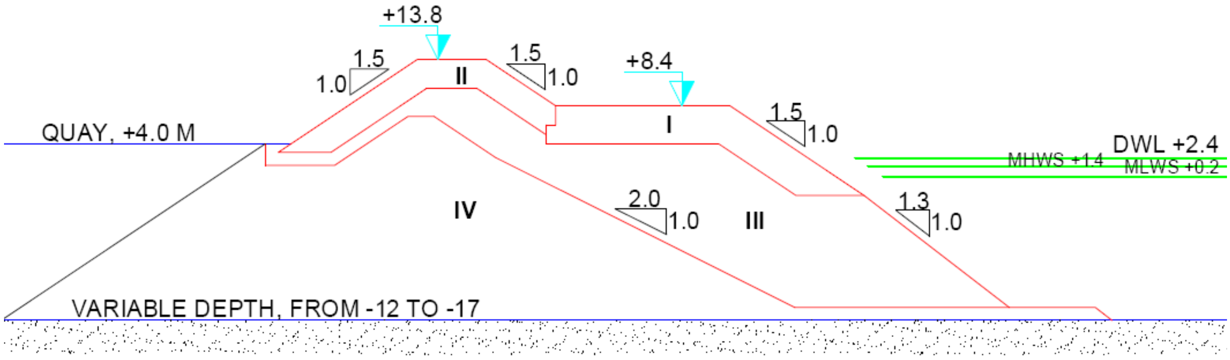
In an ongoing feasibility study for an aluminium smelter close to Húsavík, a new more exposed breakwater has been proposed. A preliminary estimate of the design wave conditions with 100 years return period is H_s = 8.0 m; a peak period of T_p = 16.6 s and a design water level of +2.4 m CD. The water depth at the outer part of the breakwater, which is about 400 m long, is -12 to -17 m CD. The preliminary functional design criteria for the breakwater are given in Table 3. These include stability criteria and overtopping criteria.

Return period (years)	H_s (m)	T_p (s)	Stability criteria	Overtopping criteria (l/s per m)
1	5	13.2	No damage	< 0.1
100	8	16.6	Start of damage	< 1
10,000	9.5	18.5	No failure	< 10

Table 3. Preliminary functional design criteria for a new Húsavík breakwater

Two types of cross sections are considered for the breakwater, an Icelandic-type Berm Breakwater and a rubble mound structure protected by one layer of Xbloc armour units, see Figures 3 and 4.

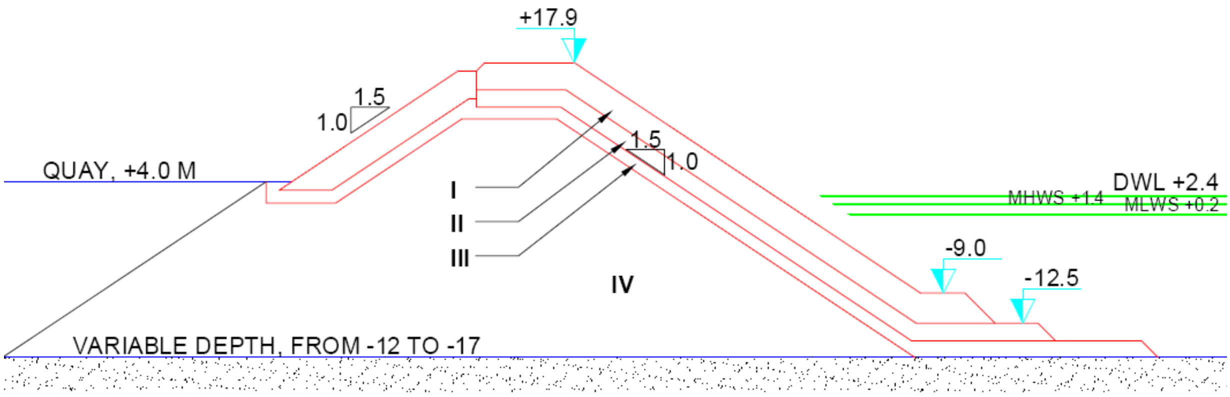
Three possible armour stone quarries have been identified for the project, see Table 4. Quarry A is the same quarry that was opened for the breakwater construction in 2002 but quarries B and C have been proposed for the new breakwater project. Hauling of material from quarry A is along public road with lorries or trailers that have to fulfil weight limitations, while a special road has to be constructed for quarries B and C and mining trucks can be used.



STONE CLASSES

CLASS	WEIGHT	MEAN WEIGHT
I	20 - 50 TONNES	30 TONNES
II	10 - 20 TONNES	13.3 TONNES
III	UNDEFINED STONE CLASSES, 1 - 20 TONNES	
IV	QUARRY RUN	

Figure 3. Icelandic-type Berm Breakwater cross section design for Húsavík



STONE CLASSES

CLASS	WEIGHT
I	28.8 TONNES Xbloc
II	2 - 6 TONNES ROCK
III	0,5 - 2 TONNES ROCK
IV	QUARRY RUN

Figure 4. Xbloc cross section design for Húsavík

Quarry	Distance (km)	Road	Preliminary quarry yield prediction		
			0.3–10 tonnes	10-20 tonnes	20-50 tonnes
A	25	Public road	35%	6%	4%
B	20	Quarry road	34%	7%	9%
C	6	Quarry road	18%	3%	0%

Table 4. Possible armourstone quarries for the new breakwater at Húsavík

The feasibility study included three possible alternatives:

- •1. Icelandic-type berm breakwater constructed of material from quarry A trucked with trailers carrying 14 tonnes of material.
- •2. Icelandic-type berm breakwater constructed of material from quarry B trucked with large mining trucks carrying 30 tonnes of material. A 20 km mining road will be constructed.
- •3. Xbloc breakwater constructed of Xbloc elements cast at the site and material from quarry C trucked with large mining trucks carrying 30 tonnes of material. A 6 km mining road will be constructed.

The fourth alternative will also be included in the feasibility study, an Xbloc breakwater as in Alternative 3 but the Xbloc units will be cast abroad where labour and cement prices are lower than in Iceland and then transported by sea to the construction site. This part of the feasibility study is not finished.

Preliminary results of the feasibility study are listed in table 5. Volumes of different stone classes for the three alternatives are given for a 400 m long section of breakwater at a 16.5 m water depth. As the quarry yield for the large stones in alternative 1, 20 to 50 tonnes, is lower than the need for this stone class there will be considerable leftovers in the quarry that can not be used for other purposes. Table 4 also lists unit prices for the different elements of the breakwater, total cost for quarry roads for alternatives 2 and 3, as well as cost per meter breakwater and total cost for the 400 m long section. The unit price for rocks is a weighted mean price for the different classes of armourstones.

The results of the study are that alternative 2, an Icelandic-type berm breakwater built from quarry B, is cheapest, total cost 18.4 million USD. Alternative 3, an Xbloc breakwater, Xbloc units cast at site and other material from quarry C, is about 15% more expensive with a total cost of 21.2 million USD. Alternative 1 is most expensive with a total cost of 31.4 million USD about 70% higher than alternative 2. As the difference between alternatives 2 and 3 is relatively small these two alternatives will be considered for further study.

	Alternative 1	Alternative 2	Alternative 3
Armour stones 20 – 50 t (m^3)	61,600	61,600	
Armour stones 6/10 – 20 t (m3)	45,200	45,200	32,000
Armour stones 0,3 – 6/10 t (m3)	256,400	256,400	161,600
Quarry run (m3)	507,600	507,600	654,800
Leftovers in quarry (m3)	670,000	0	0
Xbloc, volume concrete (m3)			35,400
Weighted unit price rock (USD/m3)	38	27	19
Unit price quarry run (USD/m3)	24	15	11
Unit price leftovers/quarry (USD/m3)	8		
Unit price Xbloc (USD/m3 concrete)			284
Cost for quarry road (million USD)	0	1.1	0.3
Cost per m (thousand USD/m)	**78.6**	**46.1**	**52.9**
Total cost (million USD)	**31.4**	**18.4**	**21.2**

Table 5. Alternatives 1 to 3, comparison of volumes and cost for 400 m long section, at a water depth of 16.5 m. All prices in USD, exclusive VAT.

6. CONCLUSIONS

Developed through a number of projects over the last two decades the Icelandic-type berm breakwater has been designed for wave height up to H_s = 7.5 m. Prototype experience exists where a breakwater has been exposed to the design wave for 10 hours with reshaping not exceeding design criteria.

Safety optimization calculations for the Icelandic-type berm breakwater show that low stability numbers for the largest stone class give the optimum safety level. As a consequence of a flat minimum of the optimum safety levels it is preferable to choose rather conservative design.

A feasibility study for a breakwater exposed to a wave height of H_s = 8.0 m with a return period of 100 years is presented. It includes three alternatives. In alternatives 1 and 2, the Icelandic-type berm breakwater is constructed from material originating from two different quarries with different hauling methods and different quarry yield predictions. The third alternative is an Xbloc breakwater using an armourstone quarry closer to the site than in alternatives 1 and 2, but with lower yields in the heavy stone classes. The preliminary cost estimate shows that alternative 2, with the Icelandic-type berm breakwater, is cheapest. However, the Xbloc breakwater is only 15% more expensive. Both these alternatives will be looked at further.

Coming to design wave heights around 8 m, the Icelandic-type berm breakwater design comes to physical limits. Will enough very large stones be available? And if so, will the berm breakwater still be cheaper than the alternative with only small rock covered with concrete Xblocs?

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