Development of an Icelandic-type berm breakwater for the Oakajee port project in Western Australia

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Introduction

The state government of Western Australia has nominated Oakajee Port and Rail (OPR) as the successful proponent for the development of a new deep water port for the export of bulk iron ore at Oakajee, Western Australia. The project will comprise: an initial phase of up to two cape class bulk ore loading berths; breakwater; dredged approach channel and manoeuvring area; and possible expansion, including a further cape class bulk berth and seven Panamax class bulk / container berths. A lee breakwater might be added at a later stage.

JFA Consultants (JFA) West Australian based specialist coastal, port and harbour engineers are managing the planning, investigations and design associated with the development of the breakwaters, the dredging and reclamation aspects of the project. The Icelandic Maritime Administration (IMA) is assisting with the breakwater design aspects, assisted by Stapi in regards to quarry investigations. HR Wallingford (HRW) are providing technical input into various aspects of the project (through JFA) including detailed review of the breakwater selection process appropriate for this location

Initial development phase

The initial phase of the development for the proposed breakwaters at Oakajee involved a mass armoured-type breakwater design due to the aggressive wave climate precluding the use of a traditional two layer statically stable breakwater design. Further analysis supported by the Rock Manual (CIRIA, 2007), however, suggested that such breakwaters should preferably be non-reshaping statically stable. The non-reshaping statically stable breakwater (also known as the Icelandic-type berm breakwater) uses selected armourstone in the potentially mobile areas dimensioned to be statically stable.

The Icelandic-type berm breakwater concept has been in development over the past 25 years and nearly 40 such structures have been constructed worldwide over a wide range of wave climates, water depths and tidal conditions. Sigurdarson et al (1998) showed some of the early berm structures constructed in Iceland as well as results from physical model studies at DHI showing reduction in erosion and recession of the berm of the Icelandic-type berm breakwater. Tørum & Sigurdarson (2001) presented the results of the PIANC Working Group 40 on Berm Breakwaters. Sigurdarson et al (2005) presented the berm breakwater protecting the Hammerfest LNG plant in northern Norway. With a 100 year design wave height of $H_s = 7.5$ m a statically stable Icelandic-type berm structure was designed and constructed with a largest stone class of 20 to 35 tonnes armourstone. Quarry yield prediction of 3 to 5% in this class proved to be accurate. Compared to the originally designed dynamic stable structure, the Icelandic-type was raised but at the same time narrowed, resulting in a significant saving in rock volume.

Safety optimisation calculations for the Icelandic-type berm breakwater show that low stability numbers for the largest stone class give the optimum safety level (Sigurdarson et al., 2007). As a consequence of a flat minimum of the optimum safety levels it is preferable to choose rather conservative design. The Icelandic-type berm breakwater should be designed for a low stability number, if possible $H_o < 2.0$ ($H_o = H_s/\Delta D_{n50}$). Optimum safety levels correspond to H_o of 1.8 and 2.0 and return periods of 25 and 50 years. With only 2% additional cost design for 100 years return period practically avoids repair.

Sigurdarson et al (2008) presented a new formula for the recession of the Icelandic-type berm breakwater, where the recession is a function of H_oT_{op}. The start of recession is estimated at $H_oT_{op} = 35$ Compared to earlier methods, the new formula has much less scatter. The formula can be used as design equation for Icelandic type berm breakwaters, up to $H_oT_{op} = 70$. Beyond that condition the structure is close to dynamic stability, which is not acceptable for Icelandic type berm breakwaters

The paper describes the development of an Icelandic-type berm breakwater for the project, Figure 1, designed on the basis of no appreciable damage for successive 100 year return period design storm events. Owner functional requirements may permit the outer breakwater to overtop, but not heavily, but the inner sections, the causeway, that will carry conveyors and probably have vessels moored behind so wave overtopping will be more strictly limited.

Recent revisions to the anticipated construction programme suggest that rapid construction is desired to limit delay in income. The owner may tolerate some operational expenditure, if balanced by lower capital expenditure.

The key to the use of the Icelandic berm breakwater design is in its ability to match the predicted quarry yields of the potential quarries. In parallel to the design process, work has been done on to assess available quarries in and around the Oakajee area. Preliminary quarry assessments have determined that large rock armour for the Icelandic-type design can be obtained from potential quarry within a reasonable distance, 5 to 10 km, from the construction site.

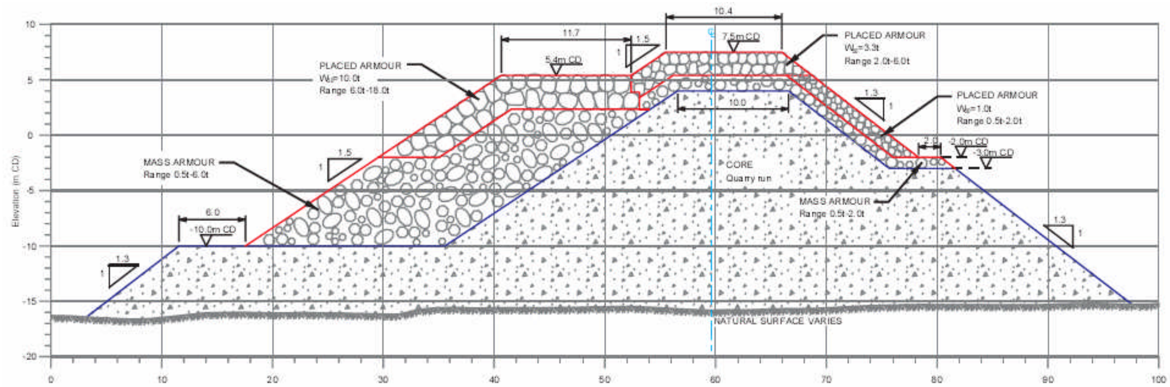

Figure 1 Typical cross section of the Icelandic-type berm breakwater

The Oakajee site

The Oakajee site is on a relatively open coast approximately 24 km north of Geraldon, which is located about 400 km north of Perth, Western Australia, Figure 2. The site at Oakajee is exposed to swell and storm waves from SSW through W to NNW with the most direct exposure around 240°N. The long period wave exposure causes considerable issues with mooring at the Geraldton Port. The tide range is small, as are tidal currents.

The outer part of the breakwater is expected to be on water depths exceeding 14 m with the breakwater head at water depths 24 m. Storms at 1:100 year return give $H_s \approx 5$m with $T_p \approx 12$-17 s in 20 m of water, so even the larger storm waves are unlikely to be depth-limited except at the inner sections of the breakwater. Cyclonic waves from around 290°N might exceed $H_s \approx 5.5$ m with $T_p \approx 10 - 12$ s, but these will be substantially oblique to the main breakwater, and "direct cyclonic hits" at this site will be rare.

The seabed is generally rocky with limited sand pockets. Rock is available from quarries in the hinterland with the primary restrictions of rock type / density, durability, maximum armour size, and haul distance all varying with quarry source selected. Access to the site from land is relatively unrestricted, with no urban development or geographical features imposing any substantial difficulties, so new haul roads may be considered.

The first design phase

During the first design phase in April 2008 the information about the availability of large armourstone for the IceBB was based on a desk study. It concluded that there was a realistic possibility that large rock armour in the 10-20 tonne and 20-30 tonne stone classes could be obtained from granulite quarries within a reasonable distance from the Oakajee port site.

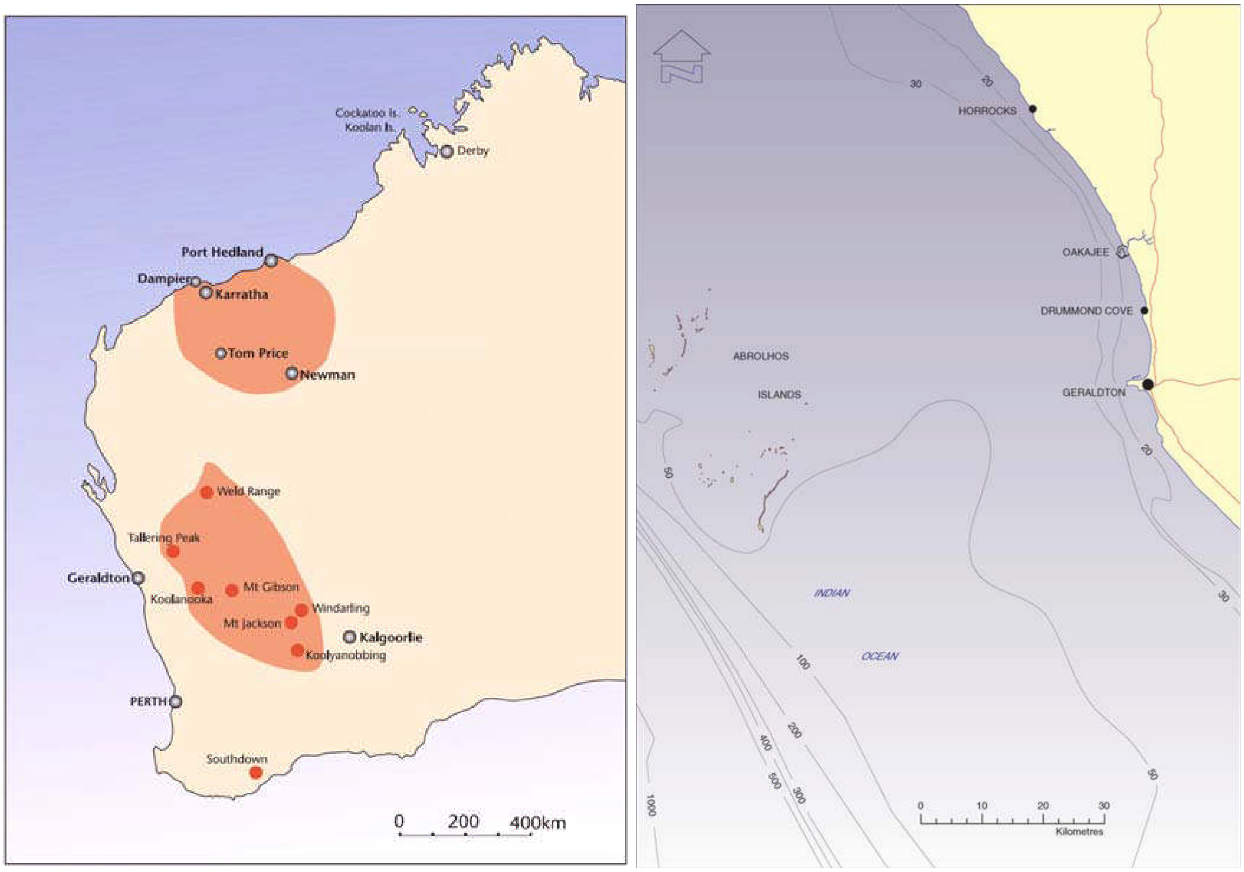

Figure 2 Left: Iron Ore Deposits, Western Australia (Macquarie presentation to OPR, 2008). Right: Location of the Oakajee site north of Geraldton.

This was confirmed with drilled rock cores from the GPP quarry site, a nearby prospective granulite quarry. A quarry yield prediction that assumed 30% of the quarried material heavier than 1 tonne and 10% heavier than 10 tonnes. The prediction was adjusted to a maximum stone size of 30 tonne. During the site visit a new site worth investigating through seismic surveys was localised. This site has been called Quarry Site D. Compared to the GPP quarry site which is about 16 km from the breakwater, the Site D is much closer or about 6 km. This has significant influence on the construction cost as the breakwater construction includes large volumes of material from the quarry.

The first drilling phase at Quarry Site D took place in November 2008. A quarry yield prediction based on 7 out of the 9 boreholes predicts 35% of the quarry yield over 1 tonne and about 12% over 10 tonne.

Table 1 shows the proposed stone classes in the first design phase and Table 2 the matching of the required volumes of the different stone classes of the first design phase and the supply of material from the quarry, based on the quarry yield prediction of the first drilling phase in Quarry Site D. In these calculations it is assumed that 2.1 million m^3 of quarried material are needed for the breakwater, 1.5 million m^3 of core and 600,000 m^3 of rock in four different stone classes. The last column of Table 2 shows that it is possible to quarry large volumes of armourstone in the quarry not used in the design.

At the time of the writing of this report the second drilling phase in Quarry Site D is ongoing, but limited results are available. Therefore the results from the first drilling phase will be used here.

The cores from the first drilling phase in Quarry Site D show that the joint spacing increases with depth which means that the possibility to produce large armourstone increases with depth. Figure 3 shows a preliminary bench yield prediction for Quarry Site D based on the first drilling phase, different yield curves for the fresh rock above and below the elevation +50 m. The surface elevation of the proposed quarry site varies between +60 and +80 m.

Table 1 Proposed stone classes in the first design phase for the Oakajee breakwater

Stone Classes	W_{min} (tonn)	W_{max} (tonn)	W_{50} (tonn)	W_{max}/W_{min}	D_{max}/D_{min}	D_{50} (m)
I	18.0	30.0	22.0	1.7	1.19	2.01
II	6.0	18.0	10.0	3.0	1.44	1.55
III	2.0	6.0	3.3	3.0	1.44	1.07
IV	0.5	2.0	1.0	4.0	1.59	0.72

Table 2 Matching of required volumes from the first design phase and the quarry yield prediction of the first drilling phase in Quarry Site D

Stone Classes	W_{50} (tonn)	Volume in brkw (m^3)	Quarry yield prediction %	Necessary quarrying %	Unused vol/ needed vol (+/-) (m^3)
I	22.0	15,000	4.0%	84,000	69,000
II	10.0	80,000	8.0%	168,000	88,000
III	3.3	185,000	10.0%	210,000	25,000
IV	1.0	320,000	13.0%	273,000	-47,000
V	Quarry run	1,500,000	65.0%	1,365,000	-135,000
Total		2,100,000			

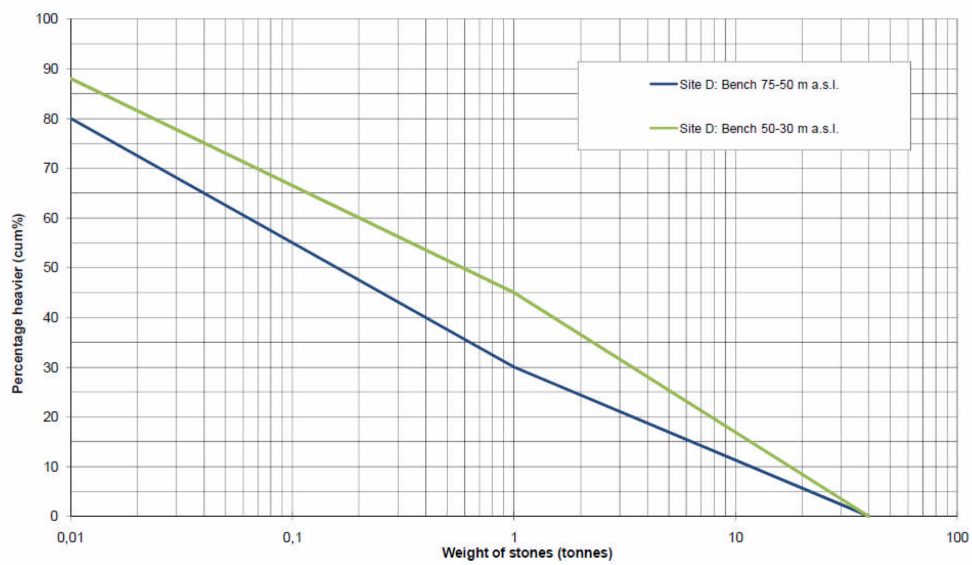

Figure 3 Preliminary bench yield prediction for Quarry Site D, based on 1st drilling phase.

The second design phase

The aim of the second design phase is to review the design of the breakwater with respect to the revised design wave conditions, to utilize better the possibilities of producing large armourstone from the granulite/quartzite quarry in Site D and to take into account new research and prototype experience of the Icelandic-type Berm Breakwater.

The second design phase included a cooperation of the breakwater designer and an experienced breakwater contractor. This cooperation resulted in improvements in the design of the breakwater and enabled more effective construction methods.

During the design process, the design wave height at the trunk of the breakwater increased about 13 – 17% and therefore it was necessary to increase the stone size. In the first design phase, stone Class I was only used for the breakwater head and Class II for the trunk. In the second design phase, stone Class I is used to protect the trunk of the breakwater as well as the breakwater head. To increase the yield in Class I, the upper and lower limits have been widened. Table 3 presents the proposed stone classes for the Oakajee breakwater in the second design phase.

Table 3 Proposed stone classes for the Oakajee breakwater

Stone Classes	W_{min} (tonnes)	W_{max} (tonnes)	W_{mean} (tonnes)	W_{max}/W_{min}	D_{max}/D_{min}	D_{50} (m)
I	15.0	35.0	21.7	2.3	1.33	2.00
II	6.0	15.0	9.0	2.5	1.36	1.49
III	2.0	6.0	3.3	3.0	1.44	1.07
IV	0.5	2.0	1.0	4.0	1.59	0.72

Table 4 shows the various design parameters for the breakwater and causeway, the design wave height and period, the design water level, the type of cross section, either Icelandic-type berm breakwater or conventional 2 layer structure, and the stone class of the main armour. Figure 4 shows the location of the different structures, the breakwater, the causeway, the tug harbour, the reclamation bund and the landfill bund.

Table 4 Design wave height for the breakwater

	Brkw-head	Breakwater trunk	Outer causeway	Outer causeway	Inner causeway
Design wave height (m)	5.1m / 5.5m	5.6m	3m<H_s<4.4	2m<H_s<3m	H_s<2m
Peak period	16.0s/10.7s	17.8s	15.9s	15.9s	15.9s
Type of cross section	IceBB	IceBB	IceBB	IceBB	Conv. b/w.
Main armour, stone class	I	I	II	III	III

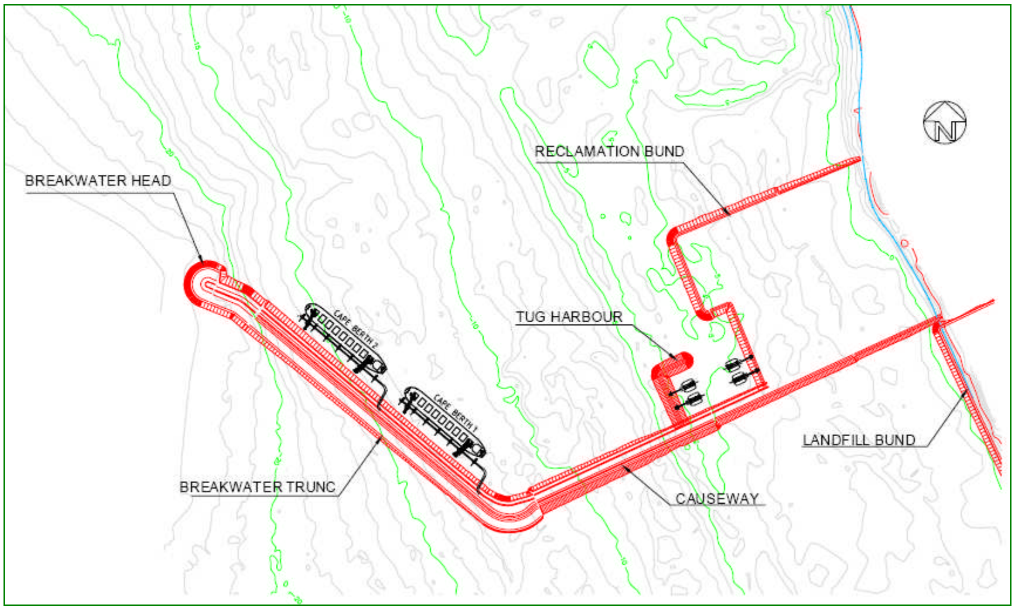

Figure 4 The different structures of project

PIANC (2003) classifies the berm breakwaters as follows, where the dimensionless stability parameters H_o and T_{om} are defined as $H_o=H_s/\Delta D_{n50}$ and $T_{om}=T_m(g/D_{50})0.5$, Table 5

Table 5 PIANC (2003) classification of berm breakwaters

Type of breakwater	H_o	H_oT_{om}
Statically stable non-reshaped berm breakwater	< 1.5-2	< 20-40
Statically stable reshaped berm breakwater	1.5-2.7	40-70
Dynamically stable reshaped berm breakwater	>2.7	>70

Table 6 shows the stability parameters for different stone classes, different parts of the breakwater and for the breakwater head, different wave type, cyclonic and non-cyclonic waves. It can be seen that with reference to the stability parameter H_o the cross sections will in all cases be grouped as *Statically stable non-reshaping berm breakwater* in the PIANC (2003) classification, but with reference to the stability parameter H_oT_{om} the cross sections enter the regime of *Statically stable reshaped berm breakwater* as the H_oT_{om} value exceeds the value of 40.

For the breakwater head we see that with reference to the H_o stability parameter the cyclonic wave conditions are more critical, but with reference to the H_oT_{om} parameter the non-cyclonic conditions are more critical.

Table 6 Stability parameters for the main armour on different parts of the breakwater

		H_s (m)	T_m (s)	Stone class	$w_{50\%}$ (t)	D_{n50} (m)	H_0	T_{0m}	H_0T_{0m}
Breakwater head	Non-cyclonic	5.1	12.3	I	21.7	2.0	1.56	27.2	42
Breakwater head	Cyclonic	5.5	8.2	I	21.7	2.0	1.68	18.2	31
Breakwater trunk	Non-cyclonic	5.6	13.7	I	21.7	2.0	1.71	30.3	52
Causeway outer	Non-cyclonic	4.4	12.2	II	9	1.5	1.80	31.3	57
Causeway inner	Non-cyclonic	3.0	12.2	III	3.3	1.1	1.72	37.0	64

The preliminary volumes needed for the different structures and different stone classes are presented in Table 7. In total, close to 2.7 million m^3 of rock is needed for the breakwater, 350,000 m^3 for the reclamation bund and tug groyne and 100,000 m^3 for the land fill bund. It is anticipated that over 500,000 m^3 of quarry run from the overburden, weathered rock, will be used for these structures. The result is that about 2.6 million m^3 of material in different stone classes are needed from the armourstone quarry.

Table 7 Volumes needed for different structures and different stone classes

Stone Classes	Break-water (m^3)	Reclm Bund/ Tug Groyne (m^3)	Land Fill Bund (m^3)	Quarry run from overbrd (m^3)	Needed from quarry (m^3)
I	110,000				110,000
II	220,000				220,000
III	230,000	40,000			270,000
IV	350,000	60,000	20,000		430,000
V	1,790,000	200,000	80,000	-500,000	1,570,000
	2,700,000	300,000	100,000	-500,000	2,600,000

It is assumed that about 2.1 million m^3 will be quarried from benches above elevation +50 m and 500,000 m^3 from benches below elevation +50 m. The quarry yield prediction for benches above and below elevation +50 presented in Figure 3 is now used to calculate the volumes in different stone classes, Table 8. The last column shows the unused volumes in the different stone classes. About 90,000 m^3 of Class I stones have not been used in the design and about 30,000 m^3 in Class II. In these calculations it is assumed that the excess material in the heavier stone classes can be used for lighter classes. This means that about 120,000 m^3 of excess material in Classes I and II can be used to fulfill the need for material in Classes III and IV.

Table 8 Volumes in different stone classes from the upper benches and lower benches and matching of the total quarrying to the volumes needed for the breakwater

Stone Classes	QYP - above +50m	Quarrying above +50m	QYP - below +50m	Quarrying below +50m	Unused vol/ needed vol (+/-)
	(%)	(m^3)	(m^3)	(m^3)	(m^3)
I	7%	150,000	11%	50,000	90,000
II	9%	190,000	12%	60,000	30,000
III	8%	170,000	14%	70,000	-30,000
IV	13%	270,000	15%	80,000	-80,000
V	63%	1,320,000	48%	240,000	-10,000
		2,100,000		500,000	

Recession of the Icelandic-type Berm Breakwater

Recently, increased attention has been paid to the recession of the berm on the Icelandic-type berm breakwater; see Figure 5 for a definition of a berm recession Rec. PIANC (2003) presented recession data of many (research) projects, with partly Icelandic-type berm breakwaters. A large scatter is present due to various influences which could not be made more specific. Some of them would be the definition of wave height (at the toe or more at deep water), placement of rock (dumped or carefully placed), way of measuring recession, etc.

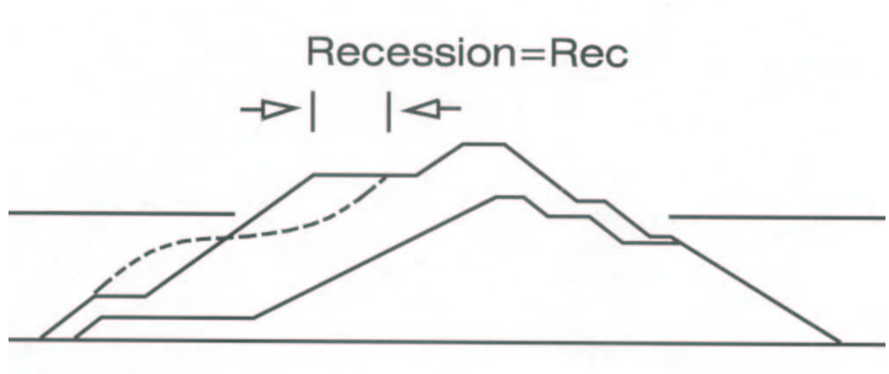

Figure 5 Recession of the berm on a berm breakwater

PIANC (2003) gave a third order polynomial function for the recession. Sigurdarson et al. (2007) changed this formula to a simple power function with a more accurate description of the reliability, but deleting depth dependence:

$$\begin{aligned} & Rec/D_{n50} = 0.037\,(H_oT_{om} - Sc)^{1.34} \\ \text{with:}\quad & Rec/D_{n50} = 0 \text{ for } H_oT_{om} < Sc \qquad (1) \\ \text{and:}\quad & \mu(Sc) = 20 \text{ and } \sigma(Sc) = 20 \end{aligned}$$

The recession is described as a function of the significant wave height and the mean wave period.

Sigurdarson et al. 2008 presented a paper that focuses on the stability and recession of the Icelandic-type berm breakwater. The PIANC (2003) recession data does not represent well the Icelandic-type berm breakwater. Requirements for the type of data needed to get more reliable results for the Icelandic-type are defined and three data sets from wave flume tests are identified. All those datasets are based on experiments on Icelandic-type berm breakwaters where the recession of the berm occurs in the narrow graded Class I rocks and is not influenced by the smaller rock classes. The construction of the rock of Class I should be done with care.

Moreover the analysis is focused on the part of the recession process that is relevant for Icelandic-type berm breakwaters, that is relatively early in the damage process, HoTop < 70. From the analysis of those data sets, a new formula for the recession of the Icelandic-type berm breakwater is presented, Equation 2, where the mean recession along the tested berm breakwaters is a function of HoTop, the significant wave height and the peak period of the waves.

$$\begin{aligned} & Rec/D_{n50} = 0.032\,(H_oT_{op} - Sc)^{1.5} \\ \text{with:}\quad & Rec/D_{n50} = 0 \text{ for } H_oT_{op} < Sc \qquad (2) \\ \text{and:}\quad & \mu(Sc) = 35 \text{ and } \sigma(Sc) = 5 \text{ and } H_oT_{op} < 70 \end{aligned}$$

During a site visit to the Sirevåg breakwater in September 2008 the breakwater was inspected. Since its construction in July 2001, the breakwater has experienced two storms close to or even exceeding the design wave conditions (Sigurdarson et al. 2003 and Tørum et al. 2005). The outer part of the breakwater and the breakwater head have suffered recession. An estimate of the recession is presented in Figure 6 where a recent aerial photo has been inserted into the original design drawing. On the breakwater trunk the max recession is of the order 2.1 to 6.2 m. With a mean diameter of Class I armour stones of 2.05 m, this corresponds to a recession of about 1 to 3*Dn50. On the breakwater head the maximum recession is about 8.4 m which corresponds to about 4*Dn50. These are preliminary results as a scanned copy of the aerial photo was used. Therefore the results can change slightly.

Table 9 presents calculations of recession of the Oakajee and Sirevåg breakwaters with the recession formulas presented in Sigurdarson, et al., 2007 and 2008. For the trunk section of the Oakajee breakwater the recession is calculated for waves from two different directional sectors, both for the sector 225-240° with the maximum wave height and also for the sector 240-255° with a slightly lower wave height but slightly longer period. According to these calculations, the recession at the trunk during the 100 year conditions, can be expected to be 4.6 to 5.9 times the mean diameter of the Class I armourstone. The recession of the trunk is less or about 3 diameters from the non-cyclonic conditions and about 1 from the cyclonic conditions.

The calculations for the Sirevåg breakwater indicate that a recession of about 5 to 7.5 stone diameters can be expected. Comparing the calculations for the Sirevåg breakwater with the estimates from the aerial photo in Figure 6 of about 1 to 3 diameter on the trunk and 4 diameters on the head, we see that both recession formulae over predict the recession.

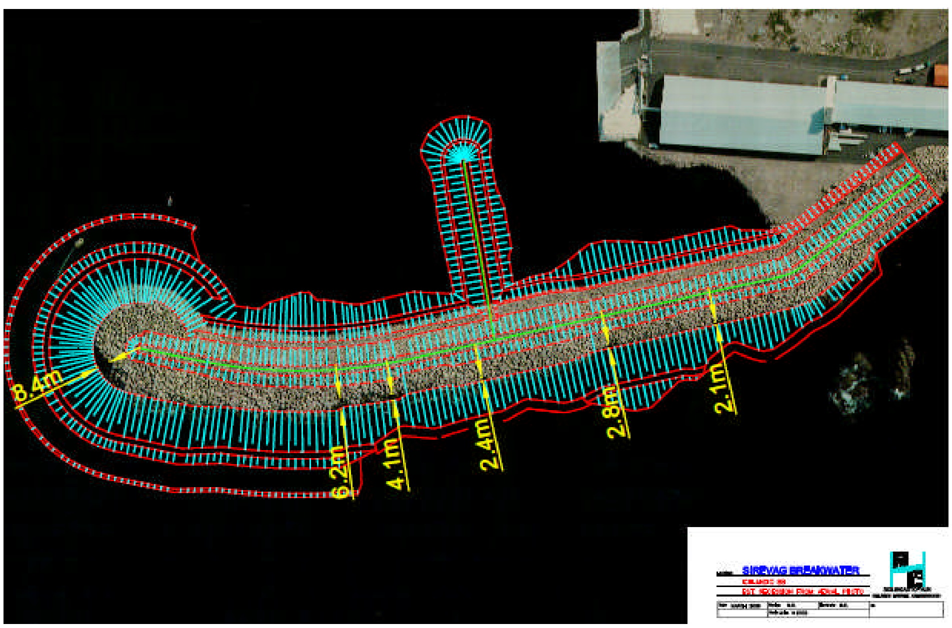

Figure 6 Sirevåg breakwater, estimated recession from an aerial photo

Table 9 Calculated recession of the Oakajee and Sirevåg breakwaters

	Waves					Recession - Rec/D_{n50}	
	RP (years)	Wave type	Dir. (°)	H_s (m)	T_p (s)	Eq. - 2007 (Rec/D_{n50})	Eq. - 2006 (Rec/D_{n50})
Sirevåg brkw		Estimated storm		6.7	16	6.6	5.0
Sirevåg brkw		Estimated storm		7	16	7.5	5.4

Planning of the breakwater construction

Constructability

Some concern has been raised regarding the constructability of the breakwater. There are four main challenges in the breakwater construction:

1. Production of large armourstone in the quarry.
2. Quarry production maintained to meet breakwater construction schedule.
3. Building of the breakwater in the constant swell conditions.
4. Limited construction time.

Production of armourstone

Extensive core drilling is taking place and the preliminary quarry yield prediction gives a good indication that it will be possible to get the necessary size and volume of armourstone. This is however only possible if a correct blasting technique is used. It has been pointed out that a wide drilling pattern and a minimum quantity of explosives will be necessary. Through the production period it is important to register the result of each blast to be able to monitor the percentage of each stone class. If the volume of armourstone achieved from the blasting does not follow the quarry yield curve then changes in the blasting pattern should be considered.

Table 10 presents a list of some of the more recent Icelandic-type Berm Breakwater projects in Iceland and Norway. Including are the construction period, design wave height for the most exposed section of the breakwater, largest rocks used, total volume and bottom depth at the deepest section of the breakwater.

Table 10 Recently constructed Icelandic-type berm breakwater

Project / Location	Construction year	H_s	Largest rocks	Total Volume	Deepest section
Sirevåg (Norway)	2000 – 2001	7.0m	20.0t <w< 30.0t	620,000m^3	-18m
Húsavík (Iceland)	2001 – 2002	6.8m	16.0t <w< 30.0t	270,000m^3	-12m
Grindavík (Iceland)	2001 – 2002	5.1m	15.0t <w< 30.0t	170,000m^3	-5m
Hammerfest (Norway)	2002 – 2003	7.5m	20.0t <w< 35.0t	3,000,000m^3	-35m
Vopnafjörður (Iceland)	2003 – 2004	5.0m	8.0t <w< 28.0t	140,000m^3	-9m
Thorlákshöfn (Iceland)	2004 – 2005	5.5m	8.0t <w< 25.0t	230,000m^3	-5m
Landeyjarhöfn (Iceland)[1)]	2008 – 2009	6.1m	12.0t <w< 30.0t	600,000m^3	-9m
Helguvík (Iceland)[1) 2)]	2008 – 2009	5.0m	15.0t <w< 25.0t	350,000m^3	-28m

1) Landeyjahöfn and Helguvík are under construction.
2) Helguvík, extension of existing breakwater.

In all of these projects, the placement of rocks has been done by large excavators. In Sirevåg, Húsavík and Hammerfest split barges were used to place quarry run and rock in the lower layers of the breakwater cross section. In Hammerfest a large crane was also used to place quarry run material to the lower berms. Split barges will be used for Landeyjahöfn and the extension of the Helguvík breakwater.

Conceptual quarry plan

It is recognised that the quarry output is an important consideration in achieving the construction timeframe. Initially a construction period of 2 years is considered, including a 6 months period of mobilisation, which leaves 18 months for the quarrying and breakwater construction. The approach of balancing quarry output to construction requirements requires a production capacity of 6 to 7,000 m^3 of material each day. This is achieved through developing a quarry with multiple operating locations and needs 300 to 400 m of working face to achieve the blast and outload cycle to meet the above demand. This can be done in multiple faces, a single long face or multiple benches.

Breakwater construction methodology

The breakwater is of the IceBB type and many of these have been built before under different conditions. Some of these breakwaters are exposed to higher design wave conditions, but what distinguishes the Oakajee site from the others is the constant swell, that will be present during the construction period.

The method that is being proposed is a combination of building the breakwater from land with conventional methods and from sea with a split barge, when the swell is not too high. The construction time is important and linked to the start of the dredging activity. A construction period of 2 years has been proposed but some time saving can be achieved with increased production in the quarry. It is possible to increase the production in the quarry by using more equipment for classifying materials. If limited space in the quarry is a problem an intermediate transport of unclassified rock mass can be done and the additional classifications equipment will be used outside the quarry area. For example at the lay down area close to the breakwater.

Analysing the constructability of the IceBB breakwater included drawing up phase plans for the construction of the breakwater. In the beginning it focused on land based construction methods, using a 120 tonne excavator to place armourstone above a level of -2 m and introducing a 300 tonnes crane for the placement of quarry run and various stone classes to the core, subsea berms and lower parts of the bulk placed stone classes. The crane would use rock skips to place material. All armourstone on the surface of the breakwater, placed stones in two layers, are assumed to be placed with an excavator to secure interlocking between the individual stones.

Cost comparison

Compared to other international breakwater projects, the construction prices for the Icelandic-type Berm Breakwater are generally lower. This is partly due to the fact that the design through the years has been developed in close cooperation with contractors performing the work. The cross sections have developed taking the construction methods into account, setting reasonable construction tolerances and defining stone classes that can be achieved without too costly quality control. On the other hand recent development towards berm breakwaters with flatter berm slopes and wider horizontal berms at low levels demanded longer reach with armourstone, lower productivity in placing material on the breakwater, which increased the construction cost. Maximizing the utilisation of all stone fragmentations from the armourstone quarry is also important for the economy of the structure.

An alternative design using single layer concrete armour units (Core-Loc), in place of the Icelandic design has also been developed. The paper will present cost comparison between the Core-Loc and the Icelandic-type berm breakwater.

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