Berm Breakwater Protection for the Hammerfest LNG Plant in Norway - Design and Construction

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Abstract
The island of Melkøya, Norway, was selected as the construction site for the new LNG gas process plant to service the Snøhvit gas field in the Barents Sea. About 2,300,00 m^3 of solid rock had to be blasted for levelling of the plant area to c+5 m and to construct a berm breakwater with a top level of c+12 m to protect the site. The initial design for a dynamically stable breakwater was changed to that of a statically stable, Icelandic-type, berm. The Melkøya breakwater design was model tested in a scale of 1:100. The final design was for a breakwater capable of withstanding a significant wave height up to 7.5 m with armourstones up to 35 tonnes at the most exposed locations. 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. Initial site investigations concluded that it would be necessary to import some of the larger stones from the mainland as the yield of the Melkøya gneiss was thought to be insufficient. Further quarry investigations, using recently developed techniques dependent on drill core interpretations, showed that all required rock could be quarried on site. The quarrying and breakwater construction were carried out in 9 months, from July 2002 until April 2003. Maximum weekly production of rock exceeded 100,000 m^3. The resulting quarry yield was very close to the predicted design curve.

Introduction
The Hammerfest LNG plant is being constructed to liquefy gas from the Snøhvit (Snow White) gas field in the Barents Sea for export by LNG/LPG vessels, Figure 1. The gas is transported by a 160 km long sub-sea pipeline to the island Melkøya just outside the town of Hammerfest, Northern Norway. Statoil is the main operator of the Snøhvit gas field. The construction period for the entire project is from 2002 to 2006.

About 2,300,000 m^3 of solid rock had to be blasted for levelling of the island of Melkøya area and to construct a 900 m long berm breakwater to protect the plant. Quarry investigations based on core drillings instead of a test blasting were used to estimate the possible yield from the quarrying. It was thought possible to obtain 3 to 5% of a stone class of 20 to 35 tonnes and 26 to 35% larger than 0.5 tonnes. About 1,500,000 m^3 of solid rock were required for the armourstone production. The contractor developed 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, except for the landfall of the gas pipes which was finished during the summer 2003. The maximum weekly production exceeded 100,000 m^3. The major challenges in the execution phase were the very tight construction schedule, producing the required stone classes, stockpiling in a very limited area and simultaneous construction of the breakwater.

The breakwater was designed as a statically stable non-reshaping berm structure for the 100-year storm event with significant wave height of 7.5 m. It should also withstand a wave height with 1000-year return period without total damage. Berm breakwaters were introduced in the early 1980's and initially designed as dynamically stable structures. The design philosophy of berm breakwaters has since changed and now aims at optimising not only the structure with respect to wave load, but also usage of rock yield of the designated armourstone quarry, construction equipment and the breakwater function. Modification of the original berm breakwater has been developed from the dynamically stable berm to the statically stable berm. This new development, sometimes referred to as "The Icelandic type", is built up of several size-graded layers (Sigurdarson et al. 1998). The Sirevåg berm breakwater in SW-Norway, which experienced the design wave conditions the first winter in service without any damage, is also of this modified type (Sigurdarson et al. 2003).

Figure 1. Location of the Snøhvit gas field in the Barents Sea and the Hammerfest LNG plant.

Physical model studies and environmental conditions

The entire breakwater structure for Melkøya was tested in a physical model at SINTEF in Trondheim in a scale of 1:100. This process included the following activities:

- Numerical modelling of the wave propagation and refraction in the area north of Melkøya
- Establishing input data to the physical model by applying open ocean data from The Snøhvit field in the refraction model
- Constructing a laboratory model in scale 1:100, covering an area of 1.85 x 2.7 km^2.
- Executing tests with the following scope:
  - Determining wave height and direction variations along the structure and comparing with numerical models
  - Proposing a breakwater design
  - Observing breakwater stability for proposed breakwater designs
  - Observing wave overtopping

Refraction Model

The refraction modelling was done using SINTEF's REFRAC model. This model applies a backward ray tracing technique to compute directional wave spectra at defined inshore locations. The generalized version of the model incorporates depth and current-induced refraction and criteria for depth induced wave breaking. To illustrate wave fields, the model may also be used to compute ray paths in forward ray tracing mode. The model is described in detail in Mathiesen (1984, 1996, 2002). The refraction model covered an area of approximately 3 x 4 km^2 of the area at and north of Melkøya. The model coverage area and an example of 14 s wave rays from the north are shown in Figure 2 in forward mode.

Because of the orientation of the fjord north of Melkøya, waves may only approach the site in a narrow sector centred about 355º direction. Thus, for design purposes, only waves with an offshore direction (of origin) of 360º ± 15º were considered. Previous studies by Statoil had shown that the wave height at a point approximately 3 km north of Melkøya is 0.6 times the wave height at Snøhvit for waves in the design range. Wave data for the Snøhvit field are shown in Table 1.

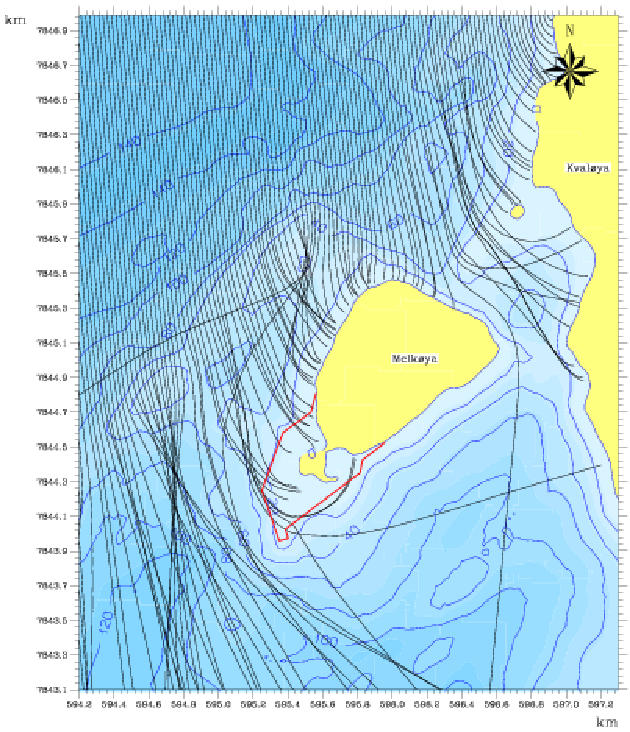

Figure 2. The model area and wave rays for 14 s period waves, offshore direction 355º. The contour of the (then) planned breakwater is shown.

Table 1. Wave data from direction north of the Snøhvit field

Parameter	Return period, years			
	1	10	100	1000
H_{m0} (m)	6.8	9.1	11.3	12.9
T_p (s)	12.6	14.1	15.6	17.0

To aid in the design and modelling process, the numerical data for each design level period were shown on a single chart. The chart for the 100-year return period is shown in Figure 3. The chart is based on a single 100-year spectrum with a peak spectral frequency of 15.5 s, and the worst cases from directions 355º – 005º.

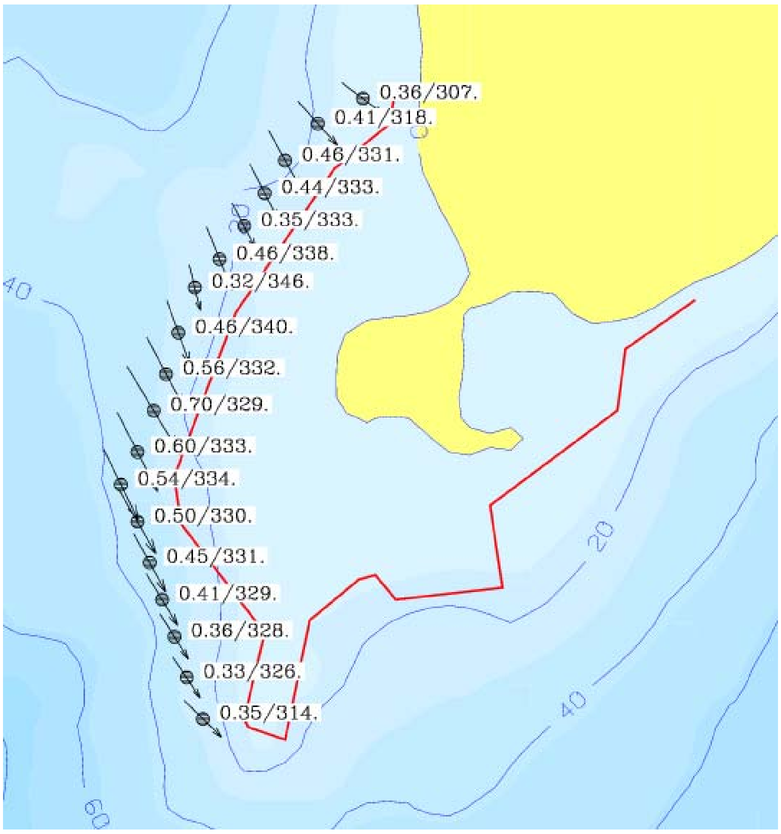

Figure 3 Aggregate design wave data for R_p = 100 years, T_p = 15.5 s along the exposed section of the breakwater. At each location, the arrow indicates the mean spectral direction, and wave height coefficient (relative to numerical model boundary) and mean direction are shown in numbers.

Physical model

The physical model of scale 1:100 is oriented along a true north-south axis, Figure 4. The depth is accurately modelled down to a depth of 55 m. Wave height in the physical model was recorded along the front of the breakwater at approximately the same positions as in the numerical model shown in Figure 3. Figure 5 shows comparisons between the physical model and the numerical model wave height coefficients starting with the northernmost location at left.

Figure 4 View of physical model of Melkøya. Waves approach from the upper right of the image. The triangular shaped structure in centre front is the Construction Jetty.

At one point it was suspected that the true wave picture was being disturbed by an unrealistically large run-up on the smooth model surface north of the contact point between the breakwater and the shore. This run-up could be observed to attack the breakwater from the rear, but it was also suspected that the return water flow created waves that would not be present in nature, since the actual shore surface is more broken up and uneven. Some tests were therefore carried out with artificial roughness elements attached from the still water level and up. This was applied on the shoreline from the breakwater and northwards. The artificial roughness consisted of a wavy, woven plastic mat, approximately 2.5 cm thick, with individual strands about 0.5 mm in diameter. As can be seen from Figure 5, the effect of the roughness was very small, and this was not used for the rest of the tests.

Between locations 0 and 200 the results from the physical model are higher than in the numerical model. In this case it is believed that the numerical model underestimates the wave heights. It is on the other hand difficult to determine which model represents more correctly the results at coordinates 500 – 600, which is the western extremity of the fill. The physical model is built down to –55 m water depth. Considerable wave refraction will occur at water depths of –55 to –150 m, which is not modelled in the physical model. The refraction diagrams from the numerical model indicate that refraction effects omitted in the physical model could have the effect of increasing the wave height on the northwest part of the structure. In the final stages of the design of the breakwater a wave height coefficient of 0.66 was chosen as a design value.

The breakwater was tested for different wave spectra (as defined by the design criteria), water levels and directions. The stability tests were performed in three stages (i) Design 1 – the preliminary design, dynamic structure; (ii) Design 2, statically stable structure; (iii) Design 3 – refinement of Design 2.

During the stability test Design 1 showed some reshaping mainly at the northeast part near coordinates 0 – 200. There was also heavy or green-sea overtopping in this area and large rocks from the armour layer were carried over the top into the plant area, clearly unacceptable in an industrial area. This design was modified as the tests progressed. Design 2, with increased berm and crest height compared to Design 1, remained stable during the 100-year storm and showed only slight reshaping in the 1000-year storm. Overtopping was reduced considerably but still the over wash was able to pick up stones and transport them into the plant area. At the end of the testing program the design was slightly adjusted in minor (but essential) details in the final Design 3. The adjustments included a wider berm of variable width at the northeast part and a 2.5 m high retaining wall at the shoreward foot of the breakwater to stem the overtopping and water penetrating the breakwater. The testing had two general criteria to meet: the static stability of the breakwater and protection against over wash. As it turned out in the tests, the reduction of over wash to an acceptable level was more difficult to meet than static stability.

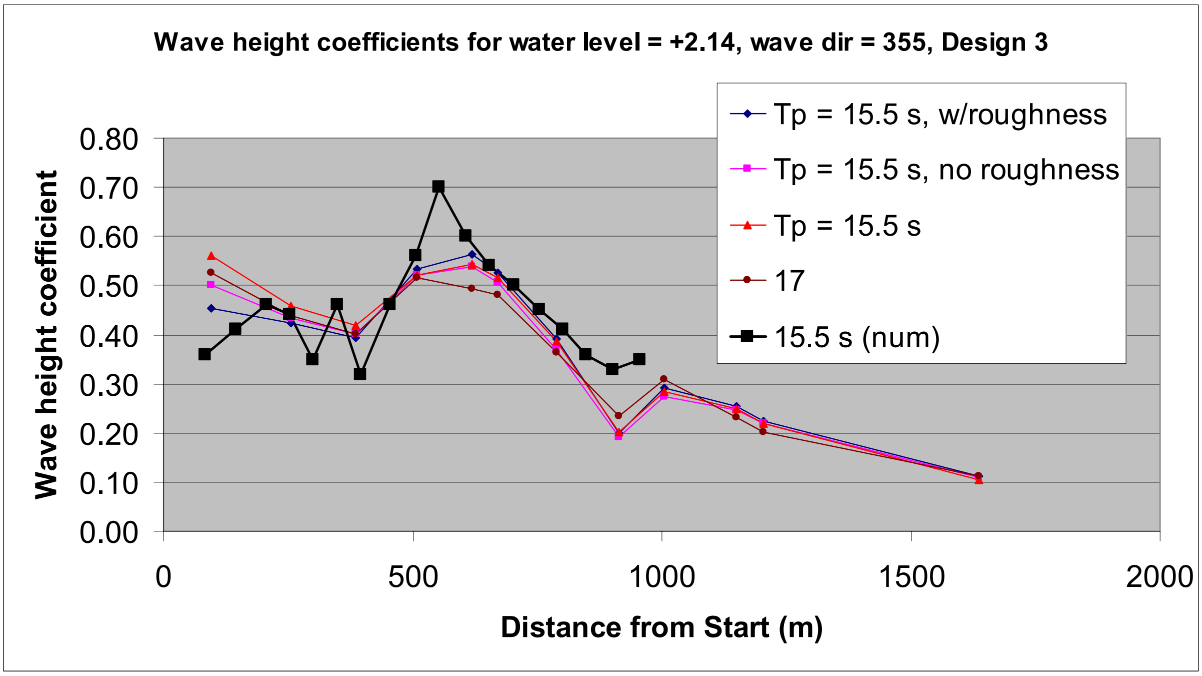

Figure 5 Comparison between physical and numerical model (solid black) results for proposed design. The water level is +2.14 m; incident wave direction is 355°. The starting point on the horizontal axis is close to the uppermost refraction point in Figure 3.

Geology of the site

A berm breakwater derives its strength and durability from the rock of which it is made. The rocks of Melkøya appear "striped" in the field, Figure 6, and such banding can be a sign of weakness, as the rock breaks more easily parallel to the banding. This type of rock is called gneiss and the banding is due to varying concentrations of the main minerals. The darker bands contain more fissile minerals and are potentially weak planes within the rock. Other planes of weakness, referred to as joints, are independent of the banding. On Melkøya the joints mostly fall into two groups; one set strikes approximately N30°E and dips about 20°E, and a second group strikes approximately N120°E and dips steeply (85°) towards the north. When quarrying takes place the rock is predisposed to break along the planes of weakness and the potential for this is assessed by carrying out quarry investigations.

Quarry investigations

A detailed geological and geophysical study was carried out on Melkøya by NGI in 1998 and the conclusion was that the rock at Melkøya was of such poor quality that an insufficient amount of armourstone could be produced in connection with the rock blasting on the island. Calculations in the middle of August 2001 showed that about 150,000 m^3 of 4 – 17 tonne large armourstone, where 50% were to be above 8.5 tonnes, had to be imported to the island from other sources and that a minimum of 430,000 m^3 of rock needed to be blasted from a good quality gabbroic quarry to obtain this quantity.

Figure 6. The rock on Melkøya is banded gneiss. Thousands of years of weathering emphasize weaknesses in the rock due to variations in mineral concentrations.

A meeting in the middle of August 2001 decided to abandon a planned test blasting on Melkøya and instead drill cored holes in the area that was to be blasted. A site visit was made by a quarry expert from Stapi Consulting Geologists and a designer from the Icelandic Maritime Administration in late August 2001 to locate 15 boreholes (C-10 to C-24) within the designated quarry areas and for a brief geological and general site investigation of the island. A visit was also made to NGI's core store in Løkken to examine drill cores (boreholes C-4 to C-9), which were drilled on Melkøya in 1997 for rock quality inspection for tunnelling. The 15 boreholes were drilled in the designated quarry areas during the first half of September 2001.

A preliminary yield prediction was made at the end of August 2001 after inspection of the cores from boreholes C-4 to C-9, for the designers to work with until the final prediction was ready. A final prediction was completed late September, after inspection of cores from boreholes C-10 to C-24 had been carried out. A quarry report was completed in October to provide information on the bedrock of Melkøya and on possible armourstone yields of the two main quarries.

The predicted quarry yield for the quarry areas on Melkøya, the Process Area Quarry (PAQ) and the Administration Area Quarry (AAQ) is shown in Table 2 where the PAQ was the largest with about 2,000,000 m^3 solid rock and AAQ with about 300.000 m^3. The yield prediction in individual stone classes from PAQ is shown in Figure 7. It was thought possible to obtain up to 5% of 20 – 35 tonne blocks under normal working conditions, but adjustments were made in the prediction allowing for the fact that the island had to be blasted out and levelled in a matter of only 10 months. It was stone class IV and not stone class I (see Table 3 for stone classes) that turned out to be most difficult to achieve, as indicated in the yield prediction.

Table 2. Quarry yield predictions for the PAQ and AAQ quarries at Melkøya, after carefully performed splitting of the largest stones over 35 tonnes.

Quarry	>0.1 tonnes	>0.5 tonnes	>1.0 tonnes	>2.0 tonnes	>5.0 tonnes	>10 tonnes	>20 tonnes
PAQ	-	31	25	20	13	8	3
AAQ	-	25	15	10	5	2	0

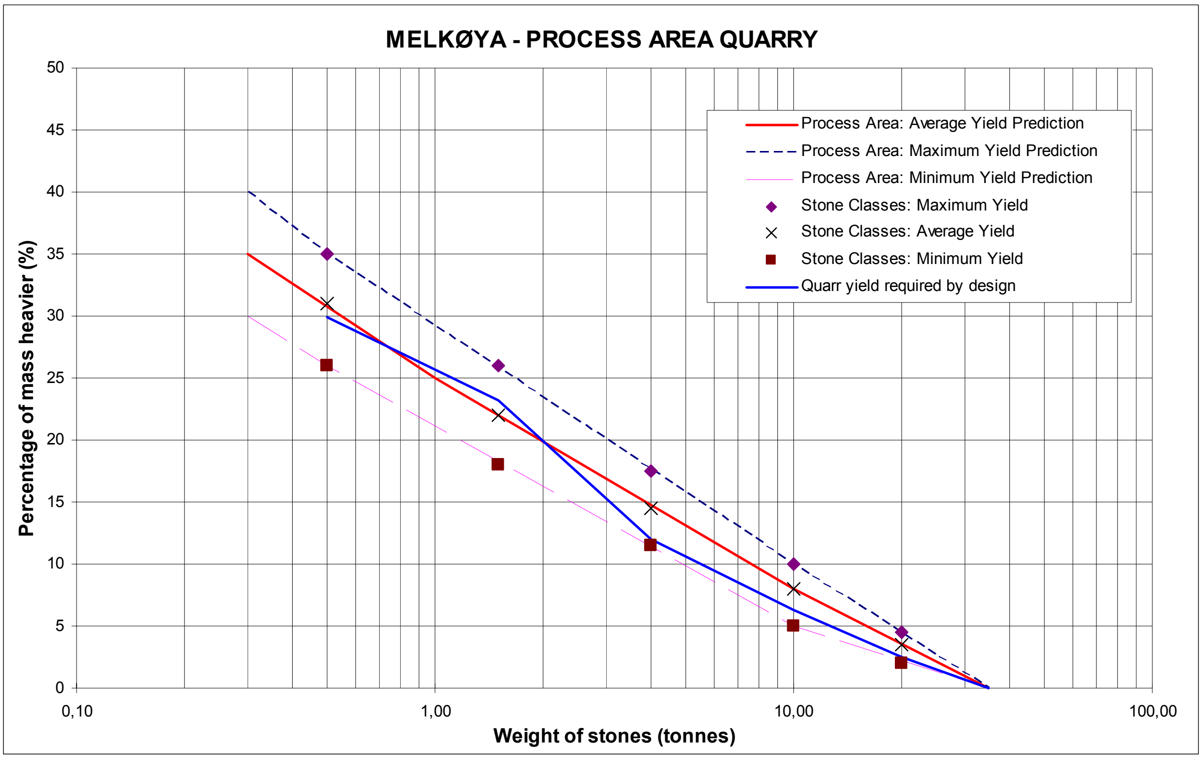

Figure 7. Yield prediction and actual quarried results in the Process Area Quarry at Melkøya. The predicted yields are used to guide the design and selection of stone classes.

Advantages of core drilling versus trial blasting in yield prediction

A small test quarry in the uppermost weathered and fractured part of a rock formation can at the best only prove the poorest possible yield of a potential quarry in that particular rock formation and says nothing about the deeper parts of the formation. Small equipment is usually used for test blasting operation, which means that the larger blocks cannot be lifted out of the blast pile, which creates a tendency to avoid a coarse blast pattern. Core drilling makes it possible to sample the proposed quarry at depth and a more accurate estimate of the quarry's yield can be made.

Test blasting was abandoned on Melkøya following the meeting with Multiconsult in August 2001 and the island was instead investigated through diamond core drilling. This made it possible to predict with confidence, a yield of up to 3 - 5% in a stone class of 20 - 35 tonnes. If test quarrying had been used the likely forecast would probably have been that quarrying of the island would produce few stones larger than 10 – 15 tonnes.

The design of the berm structure

A preliminary design of the protective berm structure proposed early in the design process was called Design 1 in the stability test. The structure had a crest height of +10.0 m and a berm height of +5.0. It was built up of two stone classes with a mean weight of 8.5 and 3.5 tonnes. For a design wave height of H_s above 6.5 m this would have resulted in a stability number, $H_o=H_s/\Delta D_{n50}$, higher than 2.7, which would classify the structure as a dynamically stable berm structure (Tørum and Sigurdarson, 2001), (PIANC, 2003).

The design basis required that the breakwater should be designed as a statically stable non-reshaping berm structure for the 100-year storm event, H_s = 7.5 m with T_p = 15.6 s, and it should also withstand a wave height with 1000-year return period, without structural damage, H_s = 8.5 m with T_p = 17.0 s.

At one stage, a light spray curtain (3 – 5 m high) on top of the breakwater was considered. The purpose of this curtain would be to reduce airborne sea spray that would eventually freeze on structures in the plant area. The idea was abandoned, however, after a numerical analysis showed that the curtain would have a minimal effect on the amount of sea spray carried over the crest, and that the net effect would be to distribute the spray over a larger area.

In July 2001 the Icelandic Maritime Administration joined the design team with SINTEF and Multiconsult-Barlindhaug.

A modified design, Design 2, had a narrower berm and higher berm and crest levels. The structure was built up of several stone classes, instead of two stone classes in the preliminary design, and based on preliminary findings from the quarry investigations at Melkøya the mean weight of the heaviest stone class was now 20 to 25 tonnes.

The final design of the breakwater consists of 8 different cross sections built up of 5 stone classes with quarry run as the sixth class (Table 3). In the final design (Figure 8) the structure was heightened to +12.0 m compared to +10.0 m in the preliminary design and the berm was heightened from elevation of +5 to +8.2 m, but at the same time the total width of the structure at elevation +5 was narrowed by 7.4 m. The width reduction resulted in a significant saving in rock volume. The total volume of stone classes I to V in the most exposed cross section was reduced by 26% in the final design compared to stone classes I and II in the preliminary design and the use of stones heavier than 6 tonnes was reduced by 39%. At the same time the final design utilised stones from 0.5 up to 35 tonnes compared to 1.5 to 17 tonnes in the preliminary design.

For a design wave of H_s = 7.5 m with T_p = 15.6 s the stability number of stone class I is H_o = 2.2 and H_oT_o = 57, which means that the structure is statically stable (PIANC, 2003). The weighted stability number for all stone classes of the most exposed cross section is on the other hand H_o = 3.0 and the H_oT_o = 92, which means that if the structure were built up with a homogeneous berm instead of a multilayer berm, it would be a dynamic structure.

Table 3. Stone Classes, weight range, mean weight and max-min weight and diameter ratio.

Stone Class	w_{min}-w_{max} (tonnes)	w_{mean} (tonnes)	w_{max}/w_{min}	d_{max}/d_{min}
Class I	20 – 35	23.3	1.5	1.14
Class II	10 – 20	13.3	2.0	1.26
Class III	4 – 10	6.0	2.5	1.36
Class IV	1.5 – 4	2.0	4.0	1.59
Class V	0.5 – 1.5	0.8	3.0	1.44

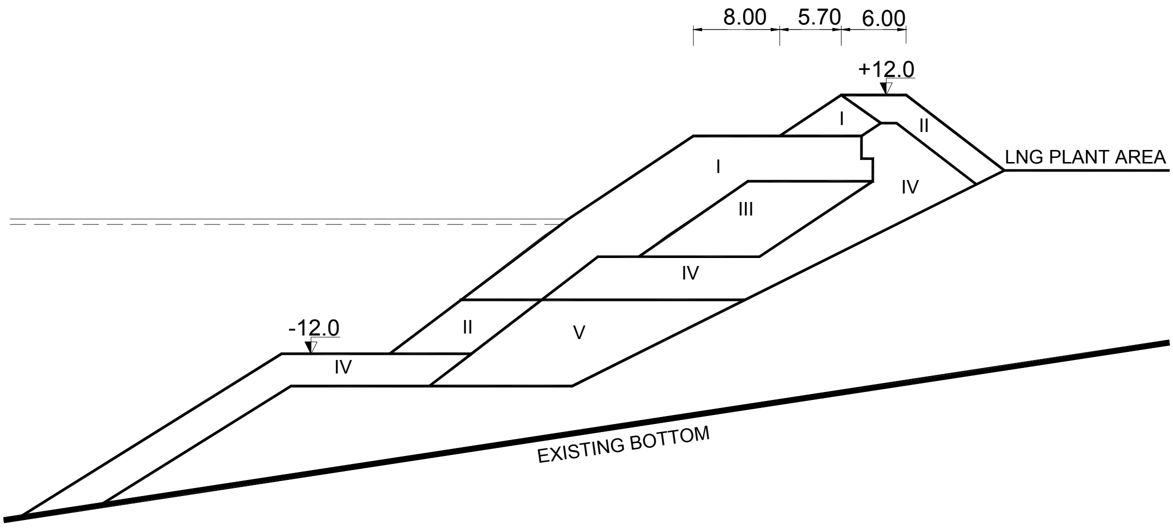

Figure 8. Cross-section of the most exposed part of the breakwater protecting the Hammerfest LNG plant.

The seabed conditions beneath the breakwater consist of a rather steep bedrock surface covered by shell sand, gravel and stones. The slope of the bedrock can only be estimated from seabed topography based on rather coarse measurements, which indicates maximum slopes of 18° from horizontal (1:3), but locally the seabed slope may be steeper. A parametric study of local stability of the breakwater toe showed that if the seabed is lower than –23 m at locations where the general seabed slope is 9 – 11° (1:6 – 1:5) the critical safety factor decreases below the factor prescribed by Design Basis (Fs=1.5). This is the case at the northwest part of the breakwater where the breakwater toe reaches a maximum water depth of –35 m with an inclination of about 11° (1:5). To fulfil the stability criteria of the lower parts of the structure it was necessary to increase the slope from 1:1.3 to 1:1.6.

The contract for Site Preparation

The contract for Site Preparation works was awarded to AFS-PIHL Group, a joint venture between AF Specialprosjekt of Norway and E. Pihl & Søn of Denmark. The construction activity was delayed by 3.5 months due to required clarifications of EU regulations. The construction schedule was to implement this delay within the final milestone. The rock excavation, breakwater construction and levelling of the island was carried out in a 9 month period from July 2002 to April 2003, except for the landfall of the gas pipes which was finished during the summer 2003, Figure 9. The maximum weekly production exceeded

100,000 m^3 of solid rock. The major challenges in the execution phase were the very tight construction schedule, producing the required stone classes (especially the two heaviest classes 10 to 20 tonnes and 20 to 35 tonnes), stockpiling in a very limited area and simultaneous construction of the breakwater.

The blasting procedure

The blasting design was based on experience from other similar projects and was further developed in cooperation with the explosives producer DYNO. The contractor's goal was to blast the rock to get fragmentation as close to the design curve as possible. This was achieved by carefully monitoring the following parameters: (i) geology; (ii) counting and weighing all stones over 4 tonnes; (iii) drill pattern and hole diameter; (iv) type and quantity of explosives.

The best results were obtained by blasting with minimum power often with a large part of the blast pile still standing, which would be considered a failure in conventional blasting. Best results were obtained: (i) with a hole spacing (E) of 3.5 m and burden (V) of 4.5 m; (ii) with an optimal bench height of 12 – 14 m; (iii) by blasting only one row at the time; (iv) by avoiding drilling parallel to the major joints; (v) by keeping the drill hole area 8 – 20 m^2; (vi) by keeping a high charge density at the bottom of holes to secure a good working platform.

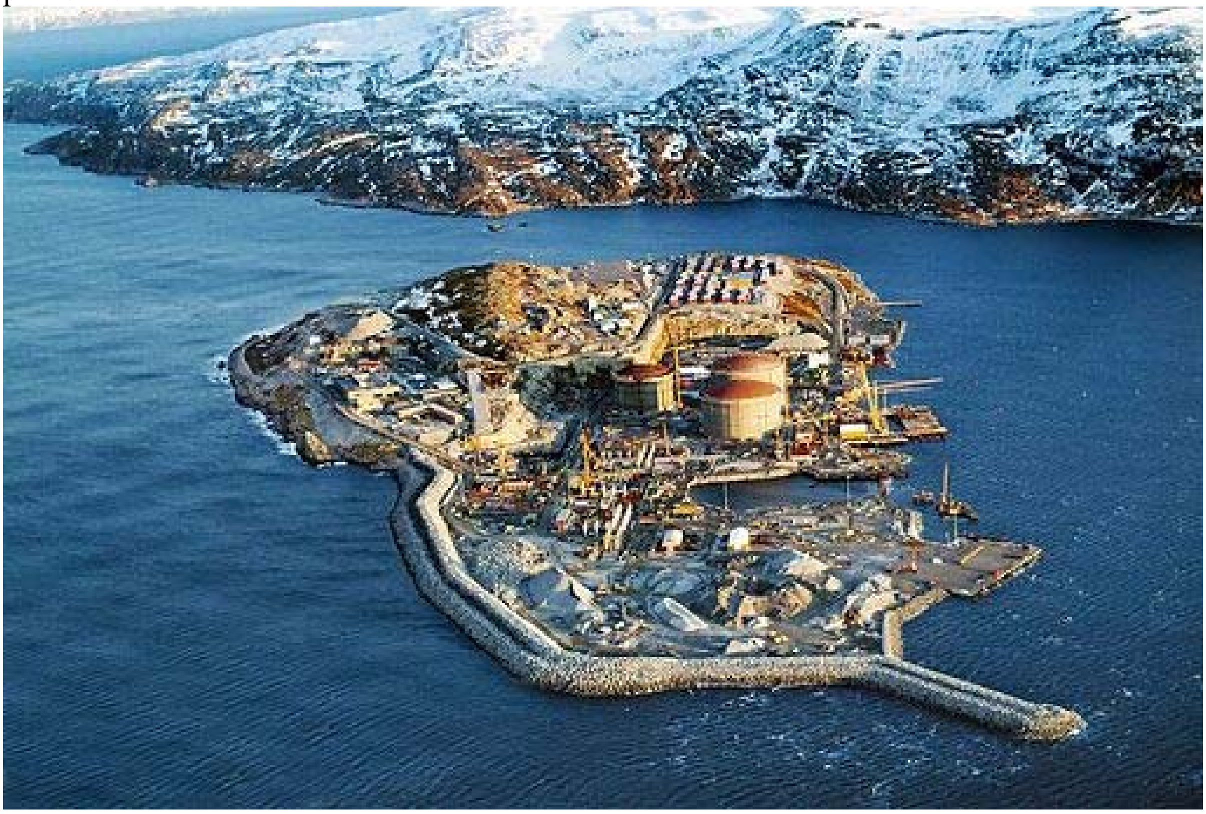

Figure 9. The Hammerfest LNG plant in November 2003 (courtesy of Statoil). The breakwater extends from the administration area in the centre left of the island to the breakwater head in the lower right hand corner and protects the plant area as well as the harbour. It is a combination of shore protection and freestanding breakwater.

The average charge was 170 – 200 g/m^3 of anfo, dynamite and slurry and the Nonel system was used for detonation. Fragmentation of each blast was registered by weighing all stones over 4 tonnes. Face mapping was used to help decide the drill pattern and use of explosives.

The holes were typically only loaded in the bottom and the middle section was left empty. Sand was used for stemming. This method is referred to as the air-deck method.

The construction equipment

The tight time frame for the project demanded a heavy machine park. But as the working area was rather small this gave the contractor problems in maintaining full productivity all the time. The quarry production and the placement on the breakwater were the main working areas. Five drill rigs were used in the quarry, three Atlas Copco D7 and two Tamrock Ranger 700. Two to three smaller rigs were used for low bench blasting and trenching. The biggest challenge was in the quarry. The armourstones and the quarry run were classified directly from the blast pile. Ten 60 – 120 tonne and six smaller excavators were used. The larger excavators were used to remove the heavier stones, 4 tons to 35 tonnes, from the blast pile and smaller ones sorted the smaller stones after removal of the larger fragments. Wheel loaders weighed, registered and carried the sorted stones to an intermediate stockpile. All armourstones heavier than 4 tonnes were weighed and registered by the wheel loader weighing system. The storing of the armourstones and quarry run was also a challenge. The biggest armourstones were produced continuously but all placed at the end of the construction and had therefore to be stored in the limited stockpile area on the island. Double handling was thus necessary, resulting in some breakage of stones. Transport of all but the biggest armour blocks was done by ten 50 – 60 tonne dumper trucks. The largest amour blocks were transported by rubber tyred wheel loaders.

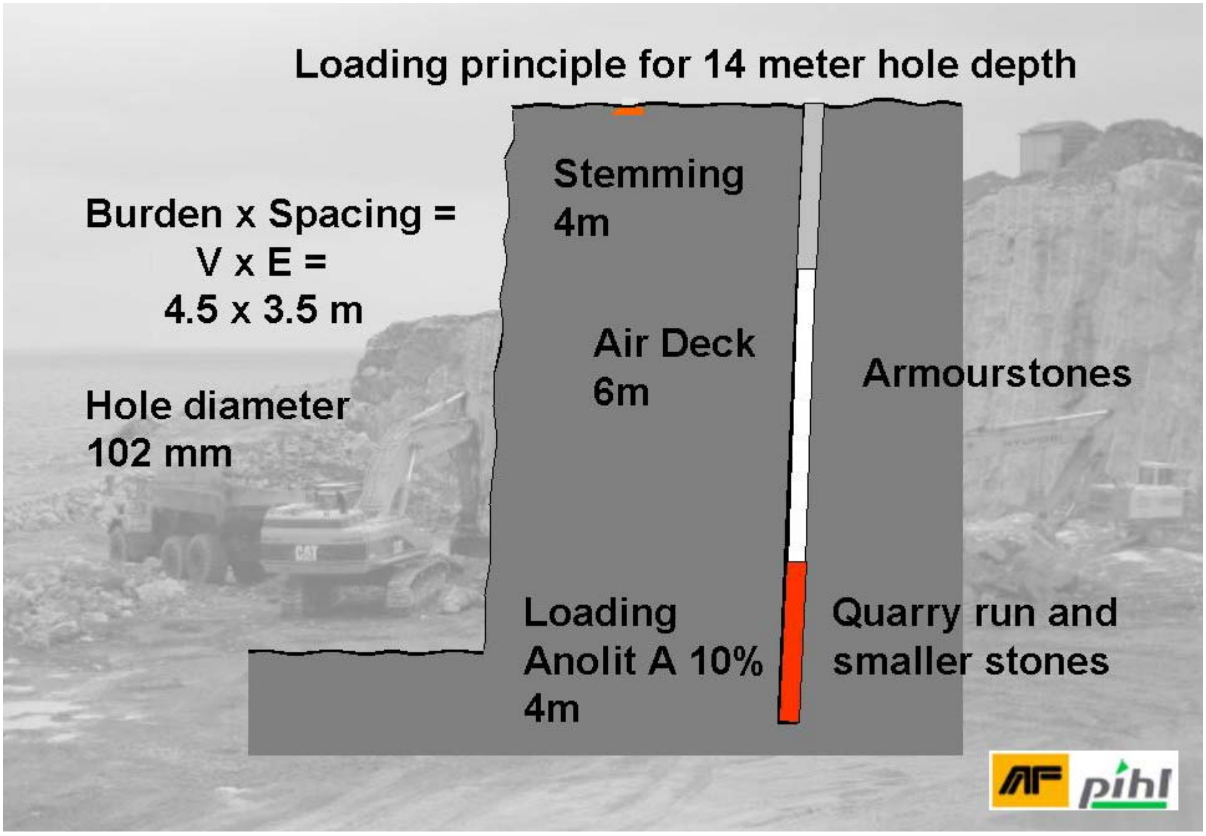

Figure 10. The blasting design by DYNO

Armourstones and quarry run were placed both from shore and from sea. Two 250 m^3 split barges were used for placing quarry run and armourstones up to an elevation of –5.0 m. The opening of the split barge is about 2.5 m allowing dumping of up to 30 – 35 tonne blocks by placing the stones in one row in the middle of the barge. Two GPS receivers, at bow and stern, were used for positioning of the barges. The rockfill inside the berm breakwater is dynamic deep compacted, since the process equipment is mainly founded on the rockfill with strong settlement tolerances.

At the start of the project there was debate as to whether to use cranes or excavators for the placement of armour blocks. In the event excavators were chosen and a 120 tonne excavator with a stone grab instead of a bucket was used. Positioning of placed stones was done with GPS receivers on the excavators. The reach of the excavators is limited making it necessary to climb on the rock berm to place the stones furthest out. Steel plates were used under the excavators to avoid breakage of the armour blocks and the undercarriage of the machine. The excavator has two major advantages over the crane, as productivity is much higher and the positioning of the armourstones is much more precise. Detailed GPS surveys of the placement were carried out afterwards from land and underwater surfaces were surveyed with echo sounders.

The armourstone production

According to the contract 2,300,000 m^3 of solid rock should be quarried at Melkøya of which the geological report recommended and the contractor subsequently planned to quarry about 1,500,000 m^3 for armourstone production. Table 4 shows required design yield based on the planned quarry for armourstone production compared to the predicted minimum, average and maximum yields. The required yield for stone classes I and II lies between the minimum and average predicted yields, required yield of classes III and V lies below the minimum yield, but the required yield of class IV lies above the predicted maximum yield. This would mean that some of the required volumes of stone class IV would have to be obtained from the remaining 800,000 m^3 of production blasting.

Table 4. Predicted yields from the Process Area Quarry compared to the design yield

Stone Class	Weight range (tonnes)	Predicted minimum yield (%)	Predicted average yield (%)	Predicted maximum yield (%)	Design yield (%)
Class I	20 - 35	2.0	3.5	4.5	2.5
Class II	10 - 20	3.0	4.5	5.5	3.8
Class III	4 - 10	6.5	6.5	8.0	5.8
Class IV	1.5 - 4	6.5	7.5	8.0	11.2
Class V	0.5 -1.5	8.0	9.0	9.0	6.7
Total I - V	0.5 - 35	26.0	31.0	35.0	30.0

By continuously monitoring the result from the blasts it was possible to get almost the exact volume of armour blocks needed. More armour blocks could have been produced, but because of the limited construction time and other logistic problem the contractor chose to produce the precise volume of the armour blocks. No stone production for this project was done outside the island.

Summary

Located in the arctic environment in the Barents Sea at 71°N the Hammerfest LNG plant at Melkøya in northern Norway is open to wave directions from 355º – 005º. The 100 year and 1000 year storm events were determined by numerical and physical models at H_s = 7.5 m with T_p = 15.6 s and 8.5 m and T_p = 17.0 s respectively.

Quarry yield prediction based on core drillings showed that it was possible to obtain up to 3 – 5% in a stone class of 20 – 35 tonnes and about 26 to 35% larger than 0.5 tonnes. This was utilised in the design of the breakwater, which consists of 8 different cross sections built up of 5 stone classes with quarry run as the sixth class (Figure 8). About 670,000 m^3 of stones from 0.5 to 35 tonnes were needed to build the 900 m long breakwater, which reaches out to a water depth of –35 m. The stability number for class I is $H_o = 2.2$ and $H_oT_o = 57$ for the 100 year case, whereas the weighted stability number for all stone classes in the most exposed cross section is $H_o = 3.0$ and $H_oT_o = 92$.

The statically stable structure was heightened but at the same time narrowed compared to a preliminarily designed dynamically stable structure, which resulted in a significant saving in rock volume. At the same time the final design utilised stones from 0.5 up to 35 tonnes compared to 1.5 to 17 tonnes in the preliminary design.

The quarrying and construction of the breakwater was completed in a 9-month period with the maximum weekly production exceeding 100,000 m^3. A flexible blast design and continuous monitoring of the blast results made it possible to get almost the exact volume of armour blocks needed for the project and import of armourstones from other sources was avoided.

Acknowledgements

The authors wish to thank Statoil ASA, the main operator of the Snøhvit gas field, and Linde AG, as the engineering contractor for the onshore plant at Melkøya, for the opportunity to publish this paper.

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