
The Mortavika Breakwater - A Harbour Exposed To Severe Weather

Tor Geir Espedal
*County Roads Office, Rogaland, Norway*

Arne E. Lothe
*SINTEF, Norwegian Hydrotechnical Laboratory, Norway*

ABSTRACT: It will be impossible to replace all the ferry routes with regular roads. However, looking at the social and economic benefits, one will soon discover that it will pay to shorten the ferry routes as much as possible. One way of shortening the ferry links is to challenge the forces of the nature and actually place the ferry harbour out towards the open sea.

The Mortavika Harbour has been placed at the outermost point of Rennesøy, and is part of the Rennesøy-Mainland Connection (RENFAST). The paper describes the hydraulic investigations programme which was required to ensure an acceptable level of service and safety and even structural integrity of the ferry terminal. The paper also includes a description of the berm-breakwater which was constructed and experiences from the construction period.

1 INTRODUCTION

Shortening of ferry routes may be an alternative to fixed connections when the construction of a fixed connection is either too expensive or is in fact impossible seen from a technical point of view.

One sees no technical solution to the crossing of the Boknafjord, which is located in the county of Rogaland. The crossing of the fjord by means of ferries has, however, been reduced to a minimum. Looking at the social-economical benefits, one prefers short ferry crossings, due to the time used by the vehicles as well as costs involved, and due to the operational costs of the ferries. It is possible to reach a level of higher capacity and more frequent sailings on a short crossing compared to a longer one, assuming that the number of ferries involved is the same.

The Boknafjord is only 8 kilometers wide at its narrowest point. The western part is rather open, resulting in heavy seas making their way into the fjord. For this reason it has been rather difficult to locate suitable places for a ferry terminal, well-protected from the North Sea. Up to 1991 the ferries crossed the Boknafjord from Mekjarvik to Skudeneshamn, a 25 kilometre long distance.

The ferry terminal at the northern side of the fjord was moved to Arsvågen in 1991, shortening the ferry distance to 20 kilometers.
When the Rennfast-connection opened November 30 1992, the ferry distance was further shortened to 8.3 kilometers. Rennfast is the mainland connection for the five major islands in the community of Rennesøy, with approximately 3000 inhabitants. At the same time the Rennfast project also contributes to a shortening of the ferry connection across the Boknafjord.

The location of the the ferry terminal on the southern side of the Boknafjord was of great importance to the ferry distance. There are no natural harbours on the northern part of Rennesøy, which would make it possible to cross the fjord at its narrowest point. There was, however, a shallow area close to the northern part, where one might construct a breakwater. In 1986 the initiative was taken to implement a project involving consideration as well as a laboratory programme concerning the possibilities for constructing a breakwater-protected ferry terminal at Mortavika, at the northern part of the island of Rennesøy.

2 HYDRAULIC INVESTIGATIONS PROGRAMME

It was recognized at the early stages of the planning that the site would be exposed to severe weather, and that an extensive programme of investigations would be required to ensure an acceptable level of service and safety and even structural integrity of the ferry terminal.

In the process, it was also recognized that the success or failure of the whole RENNFAST project would depend on this vital link in the coastal road transport system.

At the time when the planning of the Mortavika area began, extensive experience had already been gained through the planning, construction and operation of the Arsvågen terminal at the opposite end of the ferry route.

It was clear from the beginning that the whole idea of a ferry terminal at Mortavika would depend on the viability of a berm breakwater design, and every part of the planning was aimed at this.

The main elements of the investigation program have been:

A. Field measurements of waves and seiching at the site (Oct 87 - Feb 88).

B. Numerical model analysis of seiching and seiching patterns for three different harbour lay-outs.

C. 2-dimensional model tests in a wave tank to determine breakwater stability and composition of breakwater cross-section scale (1:50 scale).

D. 2-dimensional model tests in a wave tank to investigate a possible “composite berm”, consisting of two types of coarse, blasted rock (1:50 scale).

E. 3-dimensional model tests to determine wave processes in front of the breakwater, wave action inside the harbour, variations in wave attack along the breakwater, navigation and effects of dredging (1:100 scale).

F. A comparison between hindcast wave data from location in the open ocean outside the Boknafjord and locally measured data, the resulting transfer function was used to determine design wave height.

G. A process simulation where the possible gain in moving the open sea part of the ferry crossing into less exposed waters was investigated by simulation of a process in which two parallel transport systems were operated in the proposed new and the (then) existing ferry route.

In addition to this, extensive literature studies and examinations of other berm breakwater projects were carried out, and separate investigations of details such as erosion protection, fendering systems, suitability of candidate vessel, and evaluation of the need for back up ferry terminals were carried out.

2.1 Establishing the design wave height

The winter of 1987/1988, when the wave recording were made, is generally considered to be a below average winter in terms of weather exposure.

The resulting wave height at the breakwater is particularly sensitive to the offshore wave direction since a linear wave propagation to the site is only possible in a sector of 25° ± 20°. Waves in this sector is generally a transient phenomenon, since waves tend to be persistent about SSW and NW directions in the part of the North Sea. Sea states in the intermediate sector (i.e. waves that would propagate with little attenuation to the site) are generally of short duration.

A data set containing hindcast wave data from a period in outer part of Boknafjord was made available through the Norwegian Meteorological Institute (MI). The set covered the period 1955 - 1980. A model for a direction dependent wave attenuation was made and tested on the period when measurements were taken: 1987 - 1988. The result is shown in Figure 2, where the percentage of observations are plotted vs wave height. The figure shows that the model will overestimate the occurrence high waves, and this is taken as a safety margin in the following.
A 100 year return period was used for the design sea state, and it was assumed that this sea state would occur for an offshore direction of 240° or 270°.

The resulting design sea state was then established as a significant wave height $H_m = 6.7$ m and a peak spectral period of 15.4 s. The maximum wave was assumed to be 13.0 m.

2.2 Seiching in the harbour basin

Seiching is a phenomenon which is known to occur in other parts in the region, and numerical model analysis was carried out to obtain an optimum design. Three different designs were tested, of which the final design gave the strongest seiching. The strongest seiching occurred for a period of 160 s. It was found, however, that this seiching had a frequency of occurrence which would still make the design acceptable, since the later model tests strongly indicated that wave transmission in the two other designs would be unacceptably high.

In this case, well defined criteria existed for acceptat seiching forces, since the terminal from which the ferries operated until the opening of RENNFAST subject to seiching, and extensive measurements and modelling had been performed in order to eliminate the seiching. Figure 3 shows the current vector field and the surface elevation contour in the harbour basin at 170 s period (slightly above the resonant peak at 140 s for clarity).

2.3 Physical model tests

Two physical models were constructed for the investigations; one 2-dimensional wave tank model at 1:50 scale in a 5 x 54 m wave tank with water depth 1.3 m, and one 3-dimensional model to a 1:100 scale covering an area of 150 m2 in the model.

The scope of the physical model tests were different. The purpose of the 2-D tank model was to obtain data for the structural integrity and survival under design conditions of the breakwater. Parameters and processes studied included:

- rock size and gradation
- thickness of composite layers of rock
- overtopping of the breakwater
- reshaping and response to wave attack
- breakwater head design and reinforcement
- effects of high water level
- stability of toe and resistance to rolling down steep slopes of the underwater cliff which the breakwater rests on
- volume flux along the breakwater; all under design wave conditions.

In order to attain a greater degree of utilization of the blasted rock from the quarry, a design was tested in which a part of the berm was replaced by core material. In the final design, a smaller grade of rock was used i
the submerged part of the berm. The concept of a composite berm is not known to have been used before.

The purpose of the 3-dimensional model tests was to optimize design layout and geometry of the entire harbour under normal operating conditions.

The model was used to establish transfer coefficients for wave heights for all together 5 alternatives. This set of coefficients applied to the time series of wave heights emanating from the time series of offshore wave data 1955 - 1988 and the directional attenuation model. With a set of criteria for acceptable wave heights at berth, estimates of down time and service rates for each alternative were given.

The model tests showed that the harbour entrance would be the critical element in the operation of the terminal. With a maximum permitted wave height of 2.0 m in the 90 m wide entrance, the estimated service rate would be 99.6 % on a year round basis.

For lack of acknowledged criteria for wave heights in entrances, the 2.0 m value was used in the further studies.

2.4 Service rate estimates

Based on records from the ferry operations over the Boknafjord, the authors have estimated that a 99.5 % service rate will be accepted by the public as close to "full service", i.e. with approximately 44 hrs per year allowed for down time with various causes.

Assuming that modern dual-direction ferries would be available, analysis of current and wind data showed that waves at the harbour entrance would be the critical element. Based on this, a service rate of 99.6 % was estimated.

3 CONSTRUCTION OF THE BERM-BREAKWATER

The breakwater is 380 metres long, with a crest height of 8 m, determined from the design wave height. The breakwater is constructed as a berm type, where the 16 metre-wide berm in front of the breakwater body is supposed to absorb the energy of the waves before reaching the actual body. Contour line 0 has a width of 53.0 metres, while, at its broadest point, it is 95 metres, measured at the foot of the breakwater. At the deepest point one had to fill to contour line -21. The theoretical volume of the breakwater is 380 000 cubic metres.

The breakwater is constructed around an inner core, consisting of rocks varying from 0 - 1.5 tons. Several layers with varying rock sizes, according to where the force of the waves would hit the breakwater, were constructed around the core. All in all the breakwater was divided into 11 regions, and each region had its specific construction. Each region was constructed by up to 4 different layers of blocks with varying weight. It was necessary to divide the breakwater into so many regions and layers in order to utilize the available masses as much as possible. In connection with the building of the ferry terminal it was necessary to bias a certain area in order to have necessary place for cars in queue. The construction plans included the costs of up to 50 000 cubic metres of blocks one reckoned one had to buy, because one did not expect to produce what was needed on the spot. However, as time went by, one managed to get a blasting programme rolling, which enabled a block share (of blocks weighing more than one ton) of more than 50 per cent. Through optimalization of the construction and the contents of various weight fractions of the breakwater, it was possible to produce all the masses for the breakwater without having a mass surplus greater than 30 000 cubic metres.

From laboratory tests, one discovered that region number 2 and 4 were the most exposed ones on the outside of the breakwater. For this reason the breakwater consists of four different layers in these regions.

The greatest rocks were placed between contour line -1 and contour line 2 in region 2 and 4. This is the area where the energy of the wave forces is strongest. Rocks of class I, with a minimum weight of 5.5 tons and an average weight of 8.0 tons, were placed in this area. There was no upper limit concerning the weight of the blocks, and blocks up to 22-23 tons were placed here. The other regions are built in the same way, with various demands to the size of the blocks.

Figure 4 shows the ground plan of the breakwater, section of the breakwater head, classification in classes of the blocks and a description of the demands for the rock classes in the various regions.
Tests performed in the laboratory show that any breach in the breakwater would occur first at the inside of the breakwater in region 10, due to overtopping. For this reason one decided to strengthen the inside of region 8 and 10, having the same demands to the weight of the blocks as on the outside, but the blocks were orderly placed in order to ensure the stability. However, also other regions were subject to an orderly placed inside, but not with such a stringent demand to the weight of the blocks.

3.1 Demands to quality

It was very important that the quality demands had to be followed, in order to make the berm-breakwater capable of withstanding the enormous forces of the waves acting here. Perhaps the most important one of all the quality demands was concerning the content of fine substance in the block layers. In order to make the breakwater work as a berm-breakwater, a deciding factor is to have a void ratio of at least 35 per cent in the block layers. Water, making its way into the breakwater, will then have a free outlet between the blocks. Therefore one did not accept contents of any masses with less weight than 1.0 ton in the block layers. However, in the core masses of the breakwater one permitted the use of all masses from 0 - 1.5 tons, with the exception of masses containing topsoil, clay, sil and sand. A filter layer between the core masses and the block layers was also used.

All of the blocks were counted and weighed. Four groups of blocks were sorted out from the stone quarry

1. Blocks weighing more than 8 tons.
2. Blocks weighing from 5.5 tons to 8.0 tons.
3. Blocks weighing from 3.3 tons to 5.5 tons.
4. Blocks weighing from 1.0 tons to 3.3 tons.

From these groups one did pick out stones in order to meet the demands of the actual class of stones. The number of blocks and the weight of each load were
registered. This was done at a manned weighing station by measuring and recording each load.

The quality control of the demands concerning the average weight was done for 200 tons at a time. One did not tolerate any deviation from the demands concerning rocks in class I.

When it comes to rocks in class II, one did accept a deviation of 5 per cent from the demand of average weight. Concerning rocks in class III, one accepted a deviation of 8 per cent, while a 10 per cent deviation was accepted in rocks of class IV.

An additional demand was that all the blocks should be clean and without any fissures. One should also avoid using blocks where the proportion of the height and the width was higher than 3:1. The density was set at a minimum of 2.7 tons/cubic metre.

For each 30 000 cubic metres being blasted one did pick out a representative basis for testing purposes (30-50 kilos), where one estimated the density and where the rock material was analyzed. When the rocks were loaded, one carried out a visual judgement of the weight of the blocks and compared them to previous weighed and marked reference blocks. The visual power of judgement was regularly checked by means of spot tests.

3.2 Geometrical control

The actual placing of the blocks on the breakwater was carried out by a CAT 245 using a hydraulic hand. One by one the blocks were dropped or thrown out. It is not believed that this method of dumping has caused significant breakage of the rocks. Each rock was handled with rough equipment enough times for weaknesses to materialize before the dumping. In case of breakage before dumping, rocks were always brought back or entered as two blocks at lesser weight.

![Fig. 5 A block thrown into the berm-breakwater by a CAT 245 using a hydraulic hand.](image)

The principle of the berm breakwater relies on the fact that the blocks are placed in a disordered pattern. In this way the void ratio gets as high as possible. All the blocks are taken out to the core masses on the breakwater, and there they are either tipped onto the core masses or taken out on "working piers" which are extended on the shoulder approximately each 30 meters. These working-piers had to be taken away, complete, all the way down to the contour line-3, due to the strict demand concerning fine materials in the breakwater blocks. Subsequently the excavator, by means of a hydraulic hand, took the blocks one by one and either dropped or threw the blocks to their right position in the breakwater.

This way of working made strict demands to the geometrical control concerning the construction of the breakwater. Because of the scope of the work and its complexity, one decided to use electronic measurements. One made use of a small boat with an echo-sounder and a PC-registration unit. The position was estimated simultaneously by means of continuous surveying from an onshore surveying station. The registrations were carried out so frequently that it was possible to design a cross section of any place of the breakwater. In practice it turned out that one took profiles for each 5 meter longitudinal direction of the breakwater and for each class of blocks being placed. The actual designs of the profiles were arranged in such a way that one could compare planned and executed profile. In addition it was important to have access to the results of the measurements 12 hours after they had been completed. In this way one hoped to correctives to be able to fill in any missing masses in a region. Following given demands concerning admissible geometrical deviations, the control would give the permission to continue the work once the demands had been met.

As time went by, it turned out that the driver of the excavator developed a very good sense of how many blocks were needed to stay within the demands concerning deviation.

3.3 Experiences from the construction period

In order to get a sufficiently high percent of blocks, it was important to have an onshore blasting program which really could ensure a high share of blocks. Several programmes were tried, but one came up with a blasting pattern with 3.0 meters setting and 5 metres distance between the holes where the berths heights exceeded 4.0 metres. The result was a very low consumption of explosives, 0.29 kilos per cubic metre.