STABILISATION OF THE TIDAL ENTRANCE AT HORNAFJORDUR, ICELAND

Gisli Viggosson[1], Sigurdur Sigurdarson[1] and Bjorn Kristjansson[1]

Abstract

Hornafjordur is a tidal entrance on the south-east coast of Iceland, a shore with a heavy littoral drift. An extreme storm hit the coast in January 1990. It caused large shoaling to encroach upon the entrance channel from both sides, and a breakthrough occurred between the rock headland Hvanney and the South Barrier. The inlet was closed for several weeks.

A rubble mound shore protection was built on its west side in 1991 to restore the South Barrier. A curved breakwater of berm type was constructed during the summer 1995. Due to severe wave action and strong current a berm structure with toe protection was chosen.

The paper presents the experiences gained on the tidal inlet based on hydrodynamic and hydraulic models, both calibrated to the field data.

Introduction

The inlet is the entrance channel to the town of Hornafjordur, an active fishing harbour. The entrance has a rock headland on its west side and rock reefs which

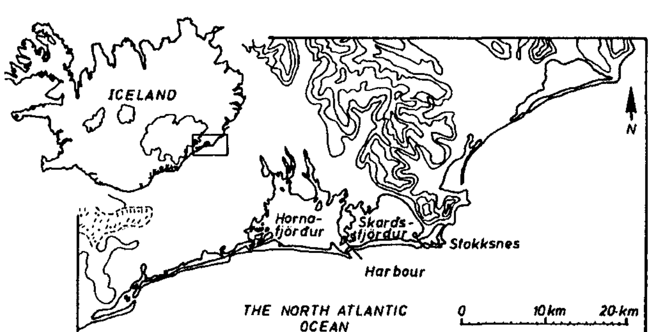

Figure 1. The location of Hornafjordur

[1] Icelandic Maritime Administration, Vesturvor 2, 200 Kopavogur, Iceland

shelter the entrance from southerly waves are located about 2 km south of it. Although the entrance has been stable in its present location for about 100 years, heavy shoaling has occurred in the entrance at 10-15 year intervals. The configuration of the coastline around Hornafjordur is controlled by two headlands, Skinneyjarhofdi 15 km in the west and Stokknes 7 km in the east as shown in Figure 1. The sea off Hornafjordur is very rough. While the total amount of drift along the shore may be of the order of millions of cubic meters of material, the net drift appears to be relatively small due to the headland configuration of the shore which made an entrance possible (Viggosson et al, 1994).

Southeast storms move material into the entrance to the shoal at the tip of the East Barrier and at high tides some material is flushed over the East Barrier into the navigation channel. At an interval of 10 to 15 years, shoaling has occurred in the navigation channel at the tip of the East Barrier as the ebb current is not able to flush material out to the shoal. Southwest and southeast storms move material into the entrance to the shoal at the tip of East Barrier and at high tides during storms some of the material may be flushed over the barrier into the navigation channel. During the summer when the wave activity is low the ebb current flushes material from the tip of East Barrier out to the shoal, so the tip may recede up to 100 meters. Usually in late August the entrance is considerably wider than in January when the wave activity is normally the highest. The diversion of currents due to shoaling and material being carried over the East Barrier cause the shore on the inlet side of the South Barrier to recede and the southeasterly waves cause the shore on the ocean side to receded due to the westward littoral drift. This leads to weakening of the barrier.

Figure 2. About 700,000 m^3 of material flushed into the navigation channel and a breakthrough through the South Barrier

On January 9, 1990, a severe south-west storm struck, with offshore wave heights exceeding 16.7 m. The storm was followed by unusually high wave activity and a breakthrough through the South Barrier occurred in March as shown on Figure 2. It is estimated that over 200,000 m^3 of material was flushed over the East Barrier into the navigation channel during this period and about 500,000 m^3 were flushed over South Barrier. As a result, the inlet was closed for coasters for several weeks.

Environmental parameters

An extensive field measurement program commenced during the summer of 1990 and continued to 1994. Included were several bathymetric surveys, water level measurements, bottom sampling, seismic refraction surveys, hydraulic measurements in the inlet and the channels into the west and east bays, aerial photography, offshore wave measurements, collection of weather data, geological assessments and research for quarry selection for the construction of rubble mounds.

Wave Climate

A Waverider buoy has since 1988 been located offshore off the south coast of Iceland close to Surtsey at 130 m water depth. The results of a statistical ana-lysis fitting the data with a three parameter Weibull distribution is shown in table 1.

Table 1

The long term wave statistics at Surtsey offshore and Hornafjordur buoy

% of time	Return period,year	Hs (m)	Tp (s)	Hs (m)	Tp (s)
60		2.3	10	3.4	10
90		4.1	11	5.8	12
99		6.5	15	9.2	15
	1	9.3	16	12.9	16
	10	10.7	18	15.2	18
	100	11.8	20	17.4	20

Waverider buoy has been located 4 km offshore the Hornafjordur entrance since February 1990, at a water depth of 32 m. The buoy is located just south of a cluster of reefs offshore from the entrance.

Tide Levels

Harmonic analysis of measured water levels in the entrance and harbour are shown in table 2.

Table 2

Tide levels in the entrance and the harbour of Hornafjordur

	Hornafjordur entrance (m)	Hornafjordur harbour (m)
Mean High Water Spring	2.11	1.81
Mean High Water Neap	1.53	1.54
Mean Sea Level	1.12	1.27
Mean Low Water Neap	0.71	1.01
Mean Low Water Spring	0.13	0.75

Measurements of currents and estimation of discharge in the Inlet

At the time when the measurements were performed, the inlet was still recovering form the shoaling that occurred the preceding winter and there were still two channels in the entrance (Snorrason et al, 1994). Current velocity measurements were made in cross section at the tidal entrance during August, 1990, twice during ebb tide and once during flood tide. The maximum recorded current velocity was 2.7 m/s and the maximum discharge during ebb tide was 3,125 m^3/s and 4,239 m^3/s during flood tide. Flow measurements were made in cross section to the Hornafjordur bay (west bay) , once during ebb tide and once during flood tide, Figure 3. The maximum recorded current velocity was 2.1 m/s, ebb tide discharge of 2,008 m^3/s and flood tide discharge of 1,920 m^3/s. The same measurements were repeated for cross section to Skardsfjordur bay (east bay), with the maximum recorded current velocity of 1.2 m/s, ebb tide discharge of 1,080 m^3/s and flood tide discharge of 1,130 m^3/s.

The fresh water influx to the inlet was estimated to be less than 5% of the tidal prism and is therefore of minor importance to the total water budget of the inlet.

For each set of measurements, the current velocity is measured in several locations distributed across the width of the cross section at different depths. The discharge is calculated from the current velocities and area of the cross section. The water level measurements at the three cross sections were made by pressure gauges from May, 1990 to January, 1991 with few brief interruptions.

The current measurements were made from July until September 1990 in cross sections to the west and east bay and outside the entrance to the harbour. Measured parameters were the current velocity, its direction, temperature of the water, the all-around pressure and the sea conductibility.

Seismic Refraction Measurements

In May 1993, a seismic profiling survey was carried out in the inlet area and the navigation channel up to the harbour. The survey indicated a basalt, generally occuring at a depth of 25 m or more, and is overlain by sediments of a variable provenance. In the channel outside the harbour entrance, for example, a thick sequence of cross-bedded sediments, dipping towards the north, may be precursors of the East Barrier.

Bottom Sampling

The material in the inlet is coarse. The main trends that can be noted are that it is primarily 3-5 mm material in the shoal off the East Barrier. The material in the channel is very coarse, d_{50} of 3-20 mm. Outside of Hvanney and west of Einholtsklettar, the material is generally fine, 0.2 mm, except in an isolated spot between Einholtsklettar and Hvanney with d_{50} of 10 mm. These samples were taken shortly after the closing of the gap between Hvanney and the South Barrier.

Hybrid Model

The goal of the study was to improve the stability and the navigational conditions in the tidal inlet of Hornafjordur. The plan for construction of shore protection and stabilisation of the inlet consists of three elements. Firstly, a rubble mound shore protection on top of the South Barrier was built to prevent overflowing of material. Secondly, a curved jetty was laid out from the tip of the East Barrier to stabilise the tip and the shoal in the entrance. The aim of the curved jetty was also to improve the current conditions, by deepening and channelling the entrance and thus improving the navigational conditions. And thirdly, a groin has to be built at Thinganessker which is about 1.2 km east of the entrance to minimise sediment transport from the east.

For a very complex situation like the one at Hornafjordur only a hybrid model was possible. It composed of field data, properly interpreted, including sediment budget, numerical and hydraulic models, both calibrated to the field data. In 1994 and 1995 both numerical and physical models of the inlet hydraulics were run to obtain further data to improve the design of solutions for the navigation through the entrance and stabilisation of it.

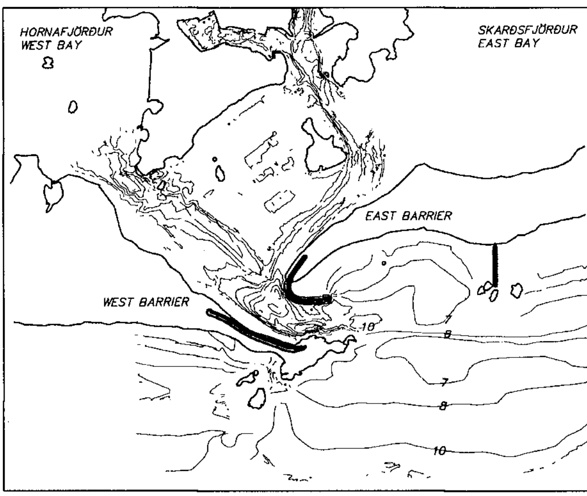

Figure 3. Stabilisation of the Hornafjordur tidal inlet. Layout of the shore protection on the South Barrier, the curved jetty on the East Barrier and the groin 1.2 km east of the entrance which will be constructed in 1999

Numerical Modelling

The AQUASEA mathematical modelling system, developed by Vatnaskil Consulting Engineers, was used to set up a numerical model of the hydrodynamics of the tidal inlet of Hornafjordur (Tomasson et al, 1994). The goal of this model applications was:

- to describe currents and discharge and to establish the relationship between the different types of field measurements carried out at different times and locations
- to describe currents and discharge in the tidal inlet for winter and summer conditions.
- to describe currents around the tip of East Barrier for different proposals for stabilising the barrier.
- to evaluate the influence of dredging the navigation channel and adding a landfill south of the harbour.
- to evaluate the influence of a proposed harbour inside the entrance.

- to evaluate the siltation of suspended load from the Hornafjordur rivers into the existing harbour and the proposed harbour.

The basis for calibration of the model was the field measurements conducted in August 1990 and a bathymetric survey from the same time. As the inlet was still recovering from the shoaling the preceding winter, bathymetric surveys performed in January 1991 and in June 1992 were chosen the basis in the model to describe typical winter and summer conditions respectively. The effects of drying of the two bays, Hornafjordur and Skardsfjordur are included in the model.

Evaluation of Data

Discharge and Tidal Prism

Table 3 summarises the discharges and tidal prism through the different cross sections at mean spring tide (Tomasson et al, 1994). The water levels are approximately the same at the start and the end of the tidal cycle.

Table 3

Results for flow and tidal prism (summer conditions, spring tide)

Cross section	Max. discharge (m^3/s)		Tidal prism (x m^6 m^3)	
	Ebb tide	Flood tide	Ebb tide	Flood tide
Tidal inlet	3440	4420	63.8	63.2
West bay	1750	2120	31.6	30.4
Harbour entrance	890	1180	18.5	16.5
East bay	730	1060	12.2	14.6

The high water in the harbour is reached over 2 hours later than the maximum discharge and velocity in the entrance. At low water in the harbour, the velocity and discharge in the entrance are very nearly zero. The maximum velocity is 2.7 m/s on the ebb and 2.0 m/s on the flood tides. The change in velocity is very rapid. As an example, it increases from 0.7 to 1.3 m/s over a period of 10 minutes.

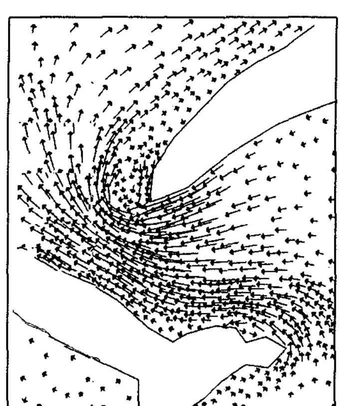

Figure 4. Maximum flood flow at spring tide at summer condition

It is interesting to note the eddy generated on the lee side of the East barrier as shown on Figure 4. A corresponding eddy, but much larger in extent forms on the other side of the east barrier at maximum flood tide.

Hydraulic Model Studies

In April 1994, a physical model study for Hornafjordur was prepared in the 20 x 40 m hydraulic model hall runned by the Icelandic Maritime Administration (Viggosson et al, 1994).

The goal of the model study was to provide design data and information for the improvements to navigation, maintenance and stability of the entrance.

The model was constructed to the scale 1:100, both in plan and vertically, covering the area from a water depth of 10 m up to the harbour area including the entrance and the inlets into the two bays, Hornafjordur and Skardsfjordur. Calibration of the model was done both by field measurements and by results from the numerical model. The model is constructed according to a 1992 bathymetric survey (summer conditions) which corresponds to the topographic information used in the numerical model. The model was built with a fixed bottom but the facility to reproduce a movable bottom in limited areas is featured. Random wave generators simulate south-west and south-east waves. Closed circuit water pump simulates a the currents by creating a steady state flow conditions through the inlets.

Stabilisation of the Entrance

In order to stabilise the entrance it has to be protected against rapid inflow of sediments to the navigation channel. Although the net drift is eastward, stability considerations following Bruun (1978) procedures, reveal that the quantity of westward drift towards the entrance may be of the order of 200,000 to 300,000 m^3 per

Figure 5. Construction of the shore protection on the South Barrier.
The alignment of the shore protection is set according to the maximum known scouring from the entrance

year causing large deposits in the outer entrance area, in the ocean and on the either side of the lava reef, Thinganessker, located about 1.2 km east of the entranee.

Improvements to the South Barrier

After the gap between Hvanney and South Barrier was closed, a southerly swell started to build up the barrier. In 1991, a 665 m long rubble mound shore protection was built along South Barrier. The alignment of the shore protection are shown in Figure 5. To prevent overflowing of material over the South Barrier a decision was made to accelerate this phase as it did not affect the hydraulics in the entrance.

Improvements to the East Barrier

To stop the transfer of material into the navigation channel it was necessary to build a curved jetty at the tip of the East Barrier. It served the dual purpose of putting a strong brake on the drift towards the channel from east at the same time working as a training wall for currents in the entrance. An accumulation capacity exceeding to 80,000 m^3 was sought. It could come during a couple of storms but as the updrift shoreline curves out the accumulation close to the jetty slows down and more material is deposited updrift. We were, therefore, facing a double-sided problem; (1) we wanted less sand from the east towards the entrance, (2) at the same time we wanted material enough to keep a stable shore between the entrance jetty and the Thinganessker tombolo.

During the evaluation of the improvement of the tip of the East Barrier a range of scenarios were developed with respect to bathymetry in the inlet, summer and winter flow conditions and different forms of the proposed jetty and the accumulation of material on the updrift side of the jetty.

Three different forms of a proposed jetty at the tip of the East Barrier, varying in the size, shape and location, were investigated. Considerable differences were found in the current field in the entrance depending on the form of the jetty, especially at flood tide where one form of the jetty was found to channel the current quite efficiently through the entrance and around the tip, while an eddy of considerable size formed on the leeward side of the two other jetties.

Comparison of flow separation at the tip of the East Barrier in the numerical versus hydraulical models reveals high degree of separation in the hydraulic model, especially after the curved jetty has been added. Therefore, careful recalibration of the models were carried out.

For each scenario, the numerical and hydraulic models were calibrated and tested. But the flow conditions at ebb and flood tide were not acceptable regarding eddies with the bathymetry from 1990 and 1992.

The bottom conditions in the spring 1994 had almost stabilised. The depth at the shoal at the tip of the East Barrier was about 4 m along the proposed jetty. By testing various forms of the jetty with material accumulated on the lee side and the bottom topography of 1994, minimum flow separation and turbulence was achieved, both in the hydrodynamic and the physical models when the depth along the jetty was increased from – 4 m by natural slope down to – 8 m. Figure 6 shows the currents at spring tide for the final proposal of the layout of the East Jetty.

To fulfil the criteria of natural slope down to – 8 m along the jetty some 60,000 m^3 of material had to be dredged. Based on these findings, dredging along the proposed jetty was planned a year or two after the construction of the east jetty. Due to the curvature of the east jetty some maintenance dredging along the jetty can be expected in near future.

The methods of (van Rijn, 1993) for bed load transport were used to predict the bottom changes in various time steps from the building of the curved jetty, where both current and wave related transport was included. Coarse sand and gravel are transported primarily as bedload where larger gravel tend to armour the upper most layer.

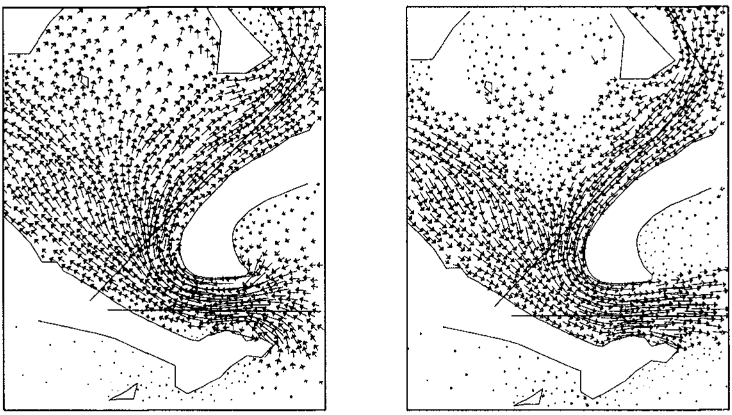

Figure 6. Currents at spring tide for the final proposal of the layout of the east jetty

To investigate the influence of expected increased depth along the proposed jetty, shear stress calculation were carried out. The Chezy bottom friction coefficient in the calibration process of the numerical model was found in the inlet $C_h = 45$ $m^{1/2}/s$. Comparisons were made between the shear stress at maximum flood and ebb tide with and without increased depth to – 8 m along the proposed jetty. At flood tide the maximum shear stress were calculated at the bottom in 1994, to maximum 80 N/m^2 in the curved of the jetty compared to 30 N/m^2 with increased depth. Nearby the entrance the maximum shear stress were 45 N/m^2 compared to 30 N/m^2 with increased depth. At maximum ebb tide, peak shear stress occurs at the tip at the jetty and along the straight leg of the jetty. In both areas the maximum shear stress decreases from 50 N/m^2 to 40 N/m^2 with increased depth.

The results for flow and tidal prism of the final proposal of the layout of the east jetty are shown in table 4.

Table 4

Results for flow and tidal prism (Summer conditions, spring tide)

Cross section		Max. discharge (m^3/s)		Tidal prism (x m^6 m^3)	
Entrance		Ebb tide	Flood tide	Ebb tide	Flood tide
Normal condition		3440	4420	63.8	63.2
Summer	1994	3435	4415	64.1	63.3
Proposed jetty	1994 (- 4 m)	3520	4435	64.6	63.8
Proposed jetty	1994 (- 8 m)	3940	4835	68.7	68.4

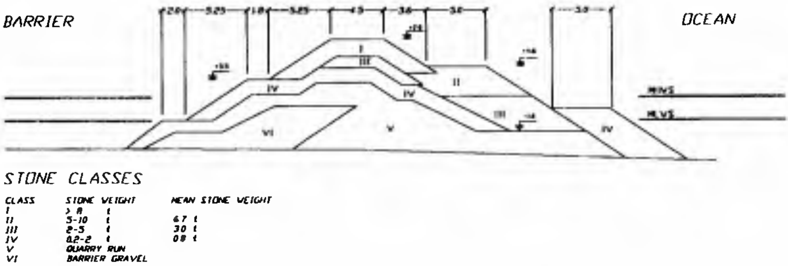

Figure 7. Typical cross section of the shore protection on the East Barrier.

A total of 99,700 m^3 were excavated from the quarry, of which 27,400 m^3 were over 2 tonne. Total project cost was 1.53 million USD, in comparison to the original cost estimate of 1.50 million USD, a difference of only 2%. (All prices include 24.5% VAT).

In the tender documents the contractor was asked to build a construction road between station 262 and 462 during the neap tide from June 3 to 10, but this could be postponed in case of a bad weather. The purpose to minimise the risk of erosion in front of the construction road and at the toe of the breakwater during the construction period. The contractor constructed a 6 m wide low road of quarry run and managed to complete the structure to station 500 within this period, Figure 8. At the same time the road was secured with 0.2 - 2 tonne rocks at the inlet side. Due to the fast progress of the work the erosion in front of the tip was limited, a maximum of 30 cm was observed. The outcome from the quarry was carefully monitored and a 100% utilisation of the quarried material was achieved.

Figure 8. During spring tide after the completion of the construction "road". Note the head difference of about 1 m.

Construction of the Rubble Mounds

The shore protection on the South Barrier

During the summer of 1991, in the early phase of field measurement and modelling, a 665 m long conventional rubble mound shore protection was constructed at the South Barrier (Sigurdarson et al, 1994). The rock in the main quarry for the South Barrier is an 8 m thick basalt lava situated about 30 km from the construction site. A total of 60,000 m^3 of material was needed from the quarry. Total contractor cost was 1.80 million USD, in comparison to the original cost estimate of 1.64 million USD, a difference of only 10 %. (All prices include 24.5% VAT).

The berm breakwater on the East Barrier

The jetty is 330 m long in addition to the 200 m long shore protection along the tip of the East Barrier was constructed during the summer 1995. The design condition of the wave height, current and erosion were evaluated by the numerical and the physical models(Sigurdarson et al, 1997).

Design conditions for the jetty were established as follows:

- The bottom material in the tidal entrance consists of coarse lava sand with particle size of 2-30 mm in diameter with some larger gravel.
- The barrier material is 1-5 mm with some gravel.
- The maximum current velocity was estimated 3.0 m/s at ebb tide and 3.5 m/s at flood tide.
- At spring tide the maximum discharge through the inlet during ebb tide was estimated 3,440 m^3/s and 4,420 m^3/s during flood tide.
- At spring tide the water level is about +2.1 m and the design water level is +3.5 m.
- The offshore significant wave height with 100 year return period is about 17 to 19 m with peak period 18 to 20 s.
- At 30 m water depth outside the entrance the 100 year significant wave height is about 12 m with peak period 18 to 20 s.
- During the design storm the significant wave height just outside the jetty is 3.8 m.

The design of the jetty had to take into account strong currents and moderate storms during the construction time. This led to a berm type breakwater of several stone classes with large toe protection as shown in Figure 7. Stone classes in the range of 2 tonne up to over 10 tonne were used. The berm consists of two classes of mean weight 6.7 and 3.0 tonne, the larger on top of the other, which corresponds to a stability numbers of 1.5 and 2.0. In front of the berm there is a toe protection of 0.2 - 2 tonne stones 20 m^3 per meter length of the structure. The predicted quarry yield over 2 tonne was 25 - 30 % which the design aimed to utilise completely to the advantage of the berm structure.

Monitoring of the tidal inlet and the breakwaters

Depth soundings in the entrance are performed regularly at an interval of 2 - 3 months and the breakwaters are visually inspected every two months. During the construction phase, bottom changes were in accordance with the predictions of the mathematical model (Viggosson et al, 1998). Figure 9 and 10 show an arial photos of the tidal entrance at Hornafjordur on the completion of the stabilisation.

Large volumes of sand and gravel were transported as shown on differential plan in Figure 11 between May 1995 and June 1998 which have not caused any problems for ships navigating the inlet.

Figure 9. Arial photo of the tidal entrance at Hornafjordur

Local accretion of sand of up to 2 meters started already near the strait part of the jetty during its construction. According to Figure 11 this accretion has eroded and the distance has decreased between the two erosion areas at the end of the jetty and near the curved jetty. The volume needed to be dredged along the jetty to fulfil the criteria of natural slope down to – 8 m are about 15,000 m^3 instead of 60,000 m^3 after the completion of the jetty.

Special attention is paid to the toe of the breakwater. The toe protection 0.2-2 tonne has been washed down and protects the eroded slope from further erosion. Figure 11 the differential plan between May 1995 to June 1998 shows an erosion up to – 7 m near the tip and - 11 m along the curved part of the jetty and accretion up to 5 m. The erosion along the curved jetty and at the tip started during the construction. The maximum erosion reaches temporary - 10 m at both places compare with 7–8 m today.

The accumulation of material at the updrift side of the jetty developed fast during the first winter. An accumulation of some 80,000 m^3 was observed after the first winter but since then the accumulation has slowed down. At present some 100,000 m^3 of material has been accumulated and more material is deposited updrift. With 10 to 15 years interval, storms from east and southeast with duration of up to 10 days have cause heavy littoral drift to the entrance. It is, therefore, planned to build a 250 m long groin next year, connecting the Thinganessker reef to the shore, some 1.2 km east of the entrance. The groin is expected to prevent littoral drift to the west and thereby stabilise the entrance.

Figure 10. Arial photo of the tidal entrance at Hornafjordur

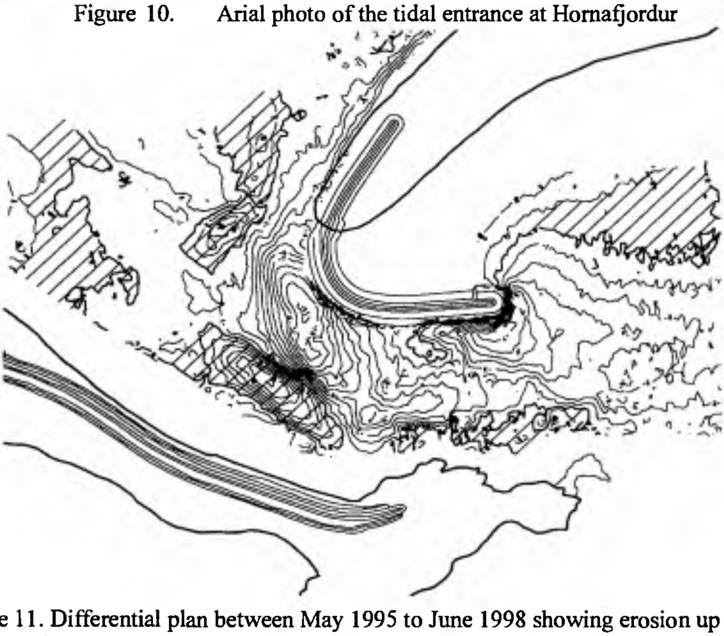

Figure 11. Differential plan between May 1995 to June 1998 showing erosion up to – 7 m near the tip and - 11 m along the curved part of the jetty and accretion up to 5 m. Dashed areas show accretion.

Conclusions

A natural inlet on a shore with heavy littoral drift, large wave forces, coarse material and strong current velocity. It was successfully stabilised with minimum inpact on the surrounding environment and navigational conditions were improved.

In 1991, a 665 m long rubble mound shore protection was built along South Barrier. To prevent overflowing of material over the South Barrier a decision was made to accelerate this phase as it did not affect the hydraulics in the entrance.

Large volumes of sand and gravel have been transported through the tidal inlet since the construction of the east jetty in 1995 and until now it has not caused any problems for ships navigating the inlet or for the berm breakwater.

Instead of a dynamic approach to berm breakwaters, as was the initial philosophy, a more stable approach has been adopted. This has lead to the "Icelandic berm" which is more economical and a more stable design than the original dynamic design approach. The toe protection and the berm breakwater have functioned as expected and the navigation conditions in the entrance are according to expectations.

According to the sounding of June 1998 it is necessary to dredge some 15.000 m^3 to fulfil the criteria of natural slope down to – 8 m along the jetty. Dredging is planned in the autumn of 1998.

There are plans to build a 250 m long groin next year, connecting the Thinganessker reef to the shore, some 1.2 km east of the entrance. The groin is expected to prevent littoral drift to the west and thereby stabilise the entrance. Due to the curvature of the east jetty some maintenance dredging along the jetty can be expected in near future

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