Bakkafjara

Sediment Transport and Morphology
Phase 2

Final Report

Siglingastofnun

Final Report
August 2007
# Bakkafjara

## Sediment Transport and Morphology

### Phase 1 and Phase 2

## Final Report

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### Key words

- Sediment transport
- Bar stability
- Sedimentation

### Classification

- Open
- Internal
- Proprietary

### Distribution

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## CONTENTS

1  INTRODUCTION AND BACKGROUND ................................................................. 1-1

2  SYNOPSIS OF PRELIMINARY ASSESSMENT ................................................. 2-1
   2.1  Near-shore Wave Conditions ................................................................. 2-1
   2.2  Analysis of Historical Storms ................................................................. 2-2
   2.3  Morphological Evolution .......................................................................... 2-3
   2.4  Overall Sediment Budget ........................................................................ 2-5
   2.5  Stability of the Bar and the "Spit" ............................................................... 2-6
   2.6  Shoreline Fluctuations ............................................................................. 2-7
   2.7  Sedimentation Inside the Harbour ............................................................ 2-8
   2.8  Currents in Front of the Harbour ............................................................... 2-8

3  SELECTION OF PERIODS FOR DETAILED MODELLING .............................. 3-1

4  MODEL INPUT .................................................................................................. 4-1
   4.1  Bathymetry and Harbour Layout ............................................................... 4-1
   4.2  Waves ........................................................................................................ 4-3
      4.2.1  Significant Model Parameters ............................................................. 4-5
      4.2.2  Boundary Conditions .......................................................................... 4-5
      4.2.3  Validation of Wave Predictions ............................................................. 4-10
      4.2.4  Modelling Examples ............................................................................ 4-10
   4.3  Flow ............................................................................................................ 4-12
      4.3.1  Modelling of River Flow ....................................................................... 4-13
      4.3.2  Modelling of Tidal Flow ....................................................................... 4-15
      4.3.3  Modelling Results ................................................................................. 4-16
   4.4  Sediment Transport ................................................................................... 4-18
      4.4.1  Grain Size and Relative Density ............................................................ 4-18
   4.5  Calibration and Validation of the Morphological Modelling Complex ........ 4-19

5  ANALYSIS OF SEDIMENT TRANSPORT AT OUTER BAR ............................ 5-1
   5.1  Period-averaged Sediment Transport Field ............................................... 5-1
   5.2  Cross-shore Transport Capacity ................................................................. 5-2
   5.3  Erosion and Deposition Patterns ............................................................... 5-4
   5.4  Sediment Transport Field with Bar Modifications ..................................... 5-10
      5.4.1  Scenario 1: Reduced Water Depth on Outer Bar ................................. 5-10
      5.4.2  Scenario 2: Excavation of the Bar ....................................................... 5-12
      5.4.3  Concluding Remarks on Simulated Scenarios .................................... 5-15
   5.5  Effect of the Harbour on the Bar ............................................................... 5-16
   5.6  Natural Changes in Bar Depression ........................................................... 5-17

6  WAVES, CURRENTS AND BED LEVELS ALONG NAVIGATION LINE .......... 6-1
   6.1  Waves along the Navigation Line and at the Wave Buoy ............................. 6-1
   6.2  Currents along the Navigation Line ............................................................ 6-6
   6.3  Waves along the Navigation Line during Selected Storms .......................... 6-7
   6.4  Currents along the Navigation Line during selected Storms ...................... 6-9

53551.ibh.jhj.be.08.2007  i
DHI Water & Environment
6.5 Morphological Changes along Navigation Line ......................................................... 6-11
6.6 Estimated Annual Transports .................................................................................. 6-15

7 COASTAL IMPACT OF BAKKAJFARA PORT FACILITY .............................................. 7-1

8 SEDIMENTATION INSIDE THE HARBOUR ................................................................. 8-1
8.1 Accumulation of Sand .............................................................................................. 8-1
8.2 Accumulation of Fine Suspended Sediments .......................................................... 8-4

9 EQUILIBRIUM WATER DEPTH IN FRONT OF HARBOUR ENTRANCE .................... 9-1
9.1 Stationary Wave Conditions .................................................................................... 9-1
9.2 Morphological Evolution during Rough South-Westerly Wave Conditions .......... 9-6
9.3 Morphological Evolution during Rough South-Eastern Wave Conditions .......... 9-9
9.4 Equilibrium Depth using Capital Dredging at Entrance ........................................ 9-11
9.5 Check of Equilibrium Depth .................................................................................. 9-13
9.6 Equilibrium Depth for Different Breakwater Configurations ................................ 9-14
9.7 Final Comments on the Equilibrium Water Depths .............................................. 9-18

10 SUMMARY .............................................................................................................. 10-1
10.1 Numerical Model .................................................................................................. 10-1
10.2 Configuration of Harbour ..................................................................................... 10-1
10.3 Sedimentation of Harbour .................................................................................... 10-1
10.4 Outer Bar Morphology ......................................................................................... 10-2

11 REFERENCES ......................................................................................................... 11-1

Appendix A Phase 1 report: Bakkafjara, Sediment Transport and Morphology
Appendix B Comparison between Wave Heights along Navigation Line obtained in MIKE 21 SW and Physical Test Model.
Appendix C Sea State Information Charts for critical waves
Appendix D Sea State Information Charts for 98% waves
FOREWORD

The present report is a revision of the draft final report from March 2007.

The March report covered Phase 2 of the studies only. The present report includes a comprehensive review of Phase 1 of the studies, in which the overall morphodynamics of the site are explained. The revision includes also further refinements, sensitivity testing and validation of the numerical modelling complex with focus on the bar development in front of Bakkafjara.

The refinements of the modelling complex comprise more detailed wave transformation with a fully spectral wave model and more detailed model resolution around Bakkafjara. These refinements have made it possible to compare directly modelled and observed development of the bar and thereby adjust the setting of model parameters. The calibration and validation of the refined model complex are presented in a new Section 4.5 of the present report.

The improvements of the modelling complex and the new numerical simulations undertaken have confirmed the conclusions drawn based on the stage of modelling in March 2007.
1  INTRODUCTION AND BACKGROUND

This report presents the Phase 2 modelling investigations and detailed analysis of waves, currents, sediment transport and morphological conditions in the coastal waters located off Bakkafjöröru. The study is concerned with the morphological impacts of the planned Bakkafjörö Harbour. The purpose of the harbour is mainly to facilitate the ferry connection to the Vestmannaeyjum. The investigation is conducted using 2D modelling techniques and is a continuation of the work reported in Ref. /1/ - Phase 1 of the present study. The report on Phase 1 is included in Appendix E. The main findings obtained during Phase 1 are presented in Section 2.

Bakkafjara is located along the southern coast of Iceland. The area is characterised by high waves that can reach heights of nearly 25 m (100yr return period, offshore region).

The river supplies large amounts of sand every year. The sand supply to the coastline determines how far the delta protrudes. The wave direction then determines whether the sand in the delta is pushed to the east or to the west. If the supply is huge, the delta protrudes far and the spit can grow extensively during events with waves coming from SE. There are two limiting factors for the growth of the river delta: the amount of material in the delta and the persistence of waves from southeast.

A bird view snapshot of the area of investigation is given in Figure 1.1. The picture shows Bakkafjörö and Vestmannaeyjum (to the right) on the 5th of December 2006.

Figure 1.1  December bird view snapshot of Bakkafjörö coastal region with Markarfjot and Vestmannanaeyjum to the right

In the following the English terms for Bakkafjörö and Vestmannaeyjum (i.e. Bakkafjara and Westmann Islands) will be adopted.

The purpose of the study is to investigate the following morphological aspects:

- Overall stability of the outer bar including the depression in the bar at Bakkafjara and the pit-type (deep trough) morphological feature at Bakkafjara
- Sedimentation rates in the harbour
- Equilibrium depth in front of the entrance

These aspects are addressed by the use of coupled numerical 2D models for current, waves and sediment transport. The numerical models are used to simulate the morpho-
logical evolution during two (2) characteristic periods and selected stationary conditions.

The surveyed bathymetry of May 2006 was used as the starting point for most simulations. However, to understand in greater detail the stability of the outer bar, additional morphological simulations are conducted.

The long-term impacts of the harbour on the coastline have been outlined by using a coastline evolution model.

Further, the wave and current conditions along the navigation line have been analysed for critical storms.

Comprehensive analyses of bathymetries (since 2002) and aerial photographs (1954 – 1996) have been performed. These analyses have shown that the location of the river mouth has changed considerably over time and that it needs to be fixed in order to guarantee the stability of the new harbour. In the present study, the river has been assumed fixed by a training wall. The mobility of the river mouth has therefore not been assessed.

The entire nearshore zone was assumed to consist of sand, without non-erodible surfaces or outcrops.

The present report is organised as follows:

Section 2 includes a summary of preliminary findings. Sections 3 and 4 describe the selection of periods which have been modelled in detail as well as set-up, calibration and validation of models. Section 5 discusses the sediment transport and morphological changes of the 2006 bathymetry during the selected storm periods. Section 6 presents the conditions along the navigation line. Sections 7, 8 and 9 present findings on coastal impact of the harbour, estimates of sedimentation in the harbour and of the equilibrium depth in front of the harbour. Section 10 presents the most important conclusions.
2 SYNOPSIS OF PRELIMINARY ASSESSMENT

In January 2006 a preliminary analysis was performed of the physical conditions along the project site that determine the feasibility of the new port. The analysis included a wave transformation study and coastal morphological – and sediment transport analysis that formed the basis for the detailed modelling study as presented in the present report. The preliminary analysis is included as Appendix A. A brief summary for the findings in Phase 1 is given below. It is emphasized that some of the findings have been further refined in Phase 2 of the studies.

2.1 Near-shore Wave Conditions

Siglingastofnun has provided offshore wave data off the Westmannaeyjar for the period 1979 - 2005. The offshore wave statistics are presented in Figure 2.1.

The wave model MIKE 21 NSW (Near-Shore Waves) was used to transform the offshore waves to a number of nearshore positions, located along the 40m depth contour. The extraction points are shown in Figure 2.1. The reference points have been selected well outside the surf zone but close enough to the coastline to account for the sheltering effect of the islands. The derived near-shore wave roses for the locations of the new Harbour (centre) and the adjacent location west (left) and east (right) from it are shown in Figure 2.2. It is clearly seen how the Westmannaeyjar give shelter for the dominating westerly waves.

Figure 2.1 Left: Offshore wave statistics, Right: nearshore extraction locations

Figure 2.2 Derived Near-shore wave roses at the location of the new harbour (centre) and adjacent locations West (left) and East (right)
### 2.2 Analysis of Historical Storms

An analysis was made of the importance of recent historic storms from the period 1979 – 2004 on the sediment budget. Individual storm events were defined as the time period in which the significant wave height $H_s$ exceeds a certain critical height. Table 2.1 shows the rankings of storms in the offshore region. The critical wave height was taken here as 8m. For each storm period the following parameters were calculated:

- Start of the storm (day, month, year)
- Duration (in hours)
- Mean and max wave height ($H_s$) during the event
- Mean wave period (MWD) and mean wave period ($T_m$).
- Littoral drift during the storm normalised with the average net annual drift for the entire period.

<table>
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<th>Start of the storm</th>
<th>Dur (Hrs)</th>
<th>$H_{\text{mean}}$ (m)</th>
<th>$H_{\text{max}}$ (m)</th>
<th>MWD (° N)</th>
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The table indicates that the most severe storm started on Dec 29th 1980 and lasted 42 hours. The mean significant wave height during the storm was 12.3 m and the maximum significant wave height was 15.0m. The average wave period and wave direction were 14.9s and 250.8° N, respectively.

The analysis indicates that the littoral drift is dominated by the occurrence of single storm events. During one single storm, a transport of sediment corresponding to several times the averaged annual sediment transport can occur.
2.3 Morphological Evolution

The shoreline evolution and the development of the river mouth have been analysed from 7 sets of aerial photos from 1954 to 2000, see Ref. /2/.

It appears that the historical coastline has been very stable around the location of the new harbour whereas variability of up to 300 m is seen to the east of the location and 100 m to the west of the location.

The bathymetries have been measured every summer since 2002. These bathymetries are reproduced in Figure 2.3. Depth contours lower than 6 m are indicated in yellow shades. The bathymetries are in fact only snapshots of the morphodynamics that occur during the year. On the basis of the available bathymetries no clear long term trends in the near-shore morphology can be derived. However, the dynamic behaviour in front of the river mouth is demonstrated clearly. The discharge of sand in the river strongly influences the bathymetry in front of the river. It is clearly seen that a bell shaped delta is formed during the summer months with the high discharges in the river. The situation in the summer with relative low wave-generated transport and high sediment supply from the river is clearly shown in the bathymetries of July 2003 and July 2004. The outer bar has migrated away from the port area and an extensive “spit” has developed from the river mouth towards West.

The bathymetry of May 2005 represents the situation at the end of the winter with strong wave-generated transport and low sediment supply from the river. The outer bar has reached its most eastern location and the “spit” on the western side of the river mouth is virtually absent.

The bathymetry of Oct. 2002 represents an intermediate stage between summer and winter conditions. The outer bar still extends quite far towards East, but the “spit” is already starting to erode by the action of the waves and the decrease in sediment supply as a result of decreasing melting water discharge.

The average river discharge of the sand fraction is known to be about 100,000 m$^3$/year, whereas the discharge of fines is much higher about 1 mill. m$^3$/year. It is expected that only the sand fraction contributes to the near-shore morphology. The fines are spread over a very large area and do not interact with the morphology.
Figure 2.3 Bathymetries 2002 – 2006
2.4 Overall Sediment Budget

The sediment transport calculations, performed with DHI’s sediment transport model LITPACK, indicate total east- and west going littoral transport rates of approximately 0.4 mill m$^3$/year and 0.3 mill m$^3$/year respectively. The supply of sand from the river is estimated at 0.1 mill m$^3$/year. These overall sediment transport rates indicate that the coastline is in a dynamic equilibrium.

The littoral drift varies not only along the coastline but also strongly over the year and from year to year. It was found that most of the time the littoral drift is to the east in all points, but short severe south-easterly wave events lead to short periods of high transport rates to the west.

The yearly eastward and westward transports for the harbour (location 5) are shown in Figure 2.4. The diagram shows east-going net yearly transport rates up to 0.5 mill m$^3$ and west-going net yearly transport rates up to 0.7 mill m$^3$.

![Figure 2.4](image)

Figure 2.4 The average net-, east- and west-going transport rates per year, location 5.

Similarly, the variability over the year is illustrated in Figure 2.5, which shows average transport for every month of the 25 years of wave data. It appears that November, December, February and March are the most severe months with gross transport rates up till 40-60,000 m$^3$/month.

![Figure 2.5](image)

Figure 2.5 Variability over the year of net-, east-, and west-going transport, location 5.
The distribution of the long-shore transport along the coastal profiles is shown for the section at location 5 in Figure 2.6. It is seen that the long-shore transport is concentrated on the bar and the inner part of the profiles. The net long-shore transport is west-going on the bar and east going at the inner part of the profile.

**Figure 2.6** Cross-shore distribution of net- and gross long-shore transport along the profile, location 5

### 2.5 Stability of the Bar and the “Spit”

From the analysis of the available bathymetries it was found that a bar exists between the sections at points 4 and 5. From the river mouth and eastward, the bar does not appear. From the analysed bathymetries, it appears that the bar has not reached and passed the harbour in any of these cases. In some cases the growth of a spit formation from the delta off the river mouth can be observed. This spit is growing towards west. However, the spit is not observed to have reached the location of the harbour. There seem to be two reasons for the change from a barred profile at points 1-5 and no bars at profiles 7-9. Firstly, there is a divergent point for long-shore transport between point 5 and 6. Secondly, the combination of smaller waves and high tidal range in the sheltered area behind Westmannaejjar do not provide optimal conditions for the development of a high bar. Furthermore, the system is affected by the yearly supply of sand from the river.

**Growth of the spit towards west**

The worst case is identified to have taken place during 1986. The maximum amount of accumulation was about 500,000 m$^3$. With a volume in the outer bar of approx. 1250 m$^3$ this corresponds to a lengthening of the spit of 400 m. It is noted that the events with west-going transport and spit growth have typically been followed with periods of east-going transport. During the 25-year time period there has not been a series of events with west-going transport, which could cause the spit to grow and close the gap between the eastern spit and the western bar.

Further, it is noted that the average supply of sand from the river is about 100,000 m$^3$/year. The growth of the spit is not only limited by the transport capacity towards the west but also by the limited source of sand in the delta. The calculations have shown that the net littoral drift is clearly east-going further to the east. This indicates that the source of sand for the spit growth is limited to the amount in the river delta.
Based on the present information it is concluded that the spit will not reach the harbour area. New information regarding extreme sand discharges in the river may raise the need for reconsidering this preliminary conclusion.

**Growth of the bar towards east**
The worst case is identified to have taken place in 1990 and has led to an eastward transport on the bar of about 170,000 m$^3$. With a volume in the outer bar of approx 1250 m$^3$/m this corresponds to a lengthening of the bar of 140 m. It is noted that the net transport on the bar at section 5 is directed to the west. It is therefore possible that a series of events with eastward transport and thereby lengthening of the bar towards east may cause problems with regard to the bar temporarily reaching the harbour area. However, with a net westward transport on the bar at section 5, south-easterly storms will push the bar back towards west.

Clearly the bar/spit developments are very delicate balances between opposing transport processes. Even though the above analysis indicates that the gap between the bar and the spit is a relatively stable feature it is recommended to include the possibility of dredging through the bar after unfortunate combinations of south-easterly storms following extreme sediment discharge in the river. The dredging would amount to about 80,000 m$^3$, considering 200 m across a bar with a 200 m wide dredged channel and an average dredging need of 2 m.

In case of strong sedimentation from either E or W, it is expected that the gap as a whole will migrate somewhat along the coastline. Therefore, if the entrance to the harbour should be (partially) blocked by the spit/bar, then the ferry would still be able to enter the port by navigation around the spit and sail a short distance along the shore between the coastline and the spit/bar.

The time periods in which this partial blocking occurs is expected to be small, in the order of a few weeks to months, depending on the prevailing wave conditions. After the establishment of the new harbour, the water depths in the outer part of the beach profile will vary the same way as they do today and are not expected to be affected by the new harbour.

### 2.6 Shoreline Fluctuations

The breakwaters of the new harbour are long enough to block the littoral drift on the shoreward side of the trough in the initial situation. The fluctuations on either side of the harbour were estimated from the accumulated eastward and westward long-shore transport on the inner part of the coastal profile.

The beach accretion along the western breakwater is after one year is estimated around 100 m. In average the volume in the deposition fillet will grow with approximately 100,000 m$^3$/year. An equilibrium will develop where the sediment by-passes the harbour.
2.7 **Sedimentation Inside the Harbour**

Based on empirical relations, the accumulation of sand inside the harbour basin was estimated to be approximately in the order of 20,000 m³/year.

2.8 **Currents in Front of the Harbour**

Flow velocities across the beach profile at Bakkafjara were calculated for 3 wave heights: 3.0 m, 3.5 m and 4.0 m. The waves were assumed to approach the coast under an angle of 45 degrees, corresponding to max. wave driven currents, see Figure 2.7.

The max. flow speed along the open coast was approx. 1.5 m/s for all events. The flow passed the harbour entrance is expected to be slightly lower than the maximum flow seen on the open coast, in the order of 1.1-1.4 m/s.

![Figure 2.7 Simulated long-shore current velocities for three incident wave heights](image)
3  **SELECTION OF PERIODS FOR DETAILED MODELLING**

Morphological model simulations applied in this analysis are computationally demanding and time-consuming. Detailed modelling of the entire period covered by the wave data is not appropriate. Therefore a number of shorter periods being representative for the wave climate at the project site must be selected.

Two distinct periods covering historical storms were selected for detailed modelling. The objective of the modelling study was to investigate the morphological evolution and potential impacts of constructing a harbour at Bakkafjara.

The modelling periods were selected from the analysis of a calculated 27-year time series of waves and longshore transport rates at the site. The time series cover the period 1979-2006 and have a three-hour data-resolution. This history of longshore transport rates was obtained in Ref. /1/ - Phase 1 of the present investigation. The two periods selected are:

- November/December 1985 (1/11/85 to 12/12/85)
- February 1989 (1/2/89 to 1/3/89)

The selected periods cover approximately 70 days in total.

The selected periods include 3 major storms i.e. storms that are placed in the Top-20 of the most important storms in terms of wave exposure (see Table 1 of Ref. /1/). The periods include two storms in November 1985 where waves exceed 8 m for approximately 66 hours in the offshore region. During one of the storms the mean wave direction was south-easterly. The 1989 February storm had 30 hours of waves above 8 m and the mean wave direction was here south-westerly. The selected periods thus include both severe south-easterly and south-westerly storms.

The wave roses for the offshore waves (63°N 21°W) are presented in Figure 3.1.

![Figure 3.1 Offshore wave conditions for November 1985 (left) and February 1989 (right)](image-url)
The corresponding time-series of offshore waves for the two periods are given in Figure 3.2.

![Time series of offshore wave heights](image)

Figure 3.2  
*Time series of offshore wave heights (wave direction indicated with blue arrows) for November 1985 (upper) and February 1989 (lower)*

The selected storms were found to be very important for the longshore transport at Bakkafjara; in fact the 1989 February and 1985 November storms were ranked the most severe storms in terms of longshore transport rates – ranked 1 and 3, respectively (see Table 3b of Ref. [1]).

In addition to including these significant storms, selected periods are seen to be somewhat similar to the annual wave climate at Bakkafjara which can be appreciated in Figure 3.3.

In the upper part of Figure 3.3 wave roses west (left side) and east (right side) of Bakkafjara for the two periods are presented. The locations of the wave roses correspond to Station 3 and 6 in Figure 4.5. In the lower part of the figure the corresponding wave roses for the annual wave climate (based on a 25-year time series of waves) are given.
Figure 3.3 Upper: wave climate west and east of Bakkafjara for the modelled periods. Lower: annual wave climate west and east of Bakkafjara (based on 25 years of waves). Locations west and east of Bakkafjara correspond to Station 3 and 6 in Figure 4.5.
4 MODEL INPUT

The modelling of waves, flow, sediment transport and morphological evolution is done using the coupled MIKE 21 FM model. The model calculates waves (MIKE 21 SW), flow (MIKE 21 HD), sediment transport and morphological evolution (MIKE 21 ST) on an unstructured mesh and in a sequential and fully integrated manner.

4.1 Bathymetry and Harbour Layout

In Figure 4.1 the unstructured meshes and the model bathymetry with the proposed Bakkafjara Harbour breakwaters are presented. The model bathymetry is derived from water depths obtained during the bathymetrical survey of May 2006.

The wave, flow and sediment transport description is refined towards the area of interest. In the vicinity of the harbour the resolution of the bathymetry is increased to \( \approx 30 \) m\(^2\) (the high resolution area includes part of the trough and bar). The relatively large model area (covers 14 km in the alongshore direction) is crucial for describing the alongshore variation in wave height which the Westmann Islands are responsible for.

Two numerical meshes have been applied. The mesh shown in Figure 4.1A is used in the initial investigations of the morphological development whereas an even more detailed mesh has been used for the final fine tuning of the modelling complex.
The harbour or ferry port is implemented as streamlined breakwaters with the entrance facing the sea. The width of the entrance (from the foot of the breakwater to the foot of the breakwater) is 100 m and the entrance is located at a (undisturbed) water depth of 8.0 m. Recent refined assessments of the channel width suggest that an entrance width of approximately 86 m is sufficient. In the present model set-up the width of the entrance has an impact on the harbour sedimentation only. The impact of different entrance widths will be addressed separately in Section 8 dealing with sedimentation of the harbour.

In Section 8 an investigation of the equilibrium depth in the entrance area is presented. Water depths here are determined by various factors including the configuration of the harbour and in particular the outer part of the breakwaters. The optimal configuration of the outer part of the breakwaters has been investigated and it turns out that the outer part...
of the western and eastern breakwater should be aligned such that the angle between them is approximately 65°.

### 4.2 Waves

A fully spectral wave model, MIKE 21 SW, has been used to simulate the propagation of waves. The model includes all wave transformation mechanisms relevant for the present purpose such as shoaling, breaking, refraction, and local wind generation. The wave model does not include wave diffraction. This mechanism is of importance in relation to coastal structures. On natural sloping beaches, such as in Bakkafjara, the effect of diffraction is usually small compared to the effect of refraction. The presence of the Westmann Islands will cause some of the incident wave energy to be diffracted. If this amount of diffracted wave energy was substantial, then it would cause crossing wave fields along the ferry trajectory. Such crossing wave fields would compromise navigation. In order to confirm the insignificance of diffraction for the wave conditions along the ferry trajectory, a model test was made using DHI’s wave model MIKE 21 PMS (Parabolic Mild Slope). This model includes both refraction and diffraction. It is based on a rectangular, equidistant grid. This type of model requires a very fine spatial resolution and is therefore usually applied in small model areas around ports and coastal structures.

Figure 4.2 shows the simulated wave field caused by the effect of diffraction alone. The effect of diffraction was isolated from the effect of refraction by assuming constant water depth in the submerged part of the model area. At the offshore boundary a typical wave situation was specified (Hs=1.0m, Tp=10s. MWD = 210 deg.).

Figure 4.3 shows the complete solution of the wave field behind the Westmann Islands (e.g. including the combined effects of refraction, diffraction, shoaling and surface breaking). It was found that considerably more energy reached the shadow area behind the islands than for the case with diffraction only. The simulations show that the wave height behind the islands can reach up to 60% of the incident wave height. The effect of diffraction accounts for less than 10% of this).
Diffraction only

Figure 4.2 Example of simulated wave field due to diffraction. Offshore wave data: $H_s = 1.0 \text{m}$, $T_p=10\text{s}$, $MWD=210^\circ \text{N}$

Complete solution

Figure 4.3 Example of simulated wave field including all wave transformation mechanisms. Offshore wave data: $H_s = 1.0 \text{m}$, $T_p=10\text{s}$, $MWD=210^\circ \text{N}$

The simulated wave heights were extracted along a line close to the future navigation route of the ferry. The results are shown in Figure 4.4. The simulations show that for this particular example maximal wave heights of approximately 60% of the incident offshore wave height can be observed along the route. It is emphasised, however, that the
simulations presented here were made with the objective to analyse the effect of diffraction only. In order to isolate the effect of diffraction, the wave conditions at the offshore model boundary were specified as unidirectional waves, without any directional spreading. It is clear that this is not a realistic assumption for the actual wave conditions along the ferry route. The simulated wave heights along the ferry route presented here must be regarded as overestimated.

![Figure 4.4 Simulated wave field including all wave transformation mechanisms. Offshore wave data: Hs = 1.0m, Tp=10s, MWD=210° N](image)

From the example simulations it was concluded that wave diffraction is not a dominant transformation mechanism for the wave conditions in the shadow area of the Westmann Islands and along the ferry navigation route. A more detailed analysis of wave conditions along the ferry route is presented in Chapter 6.

### 4.2.1 Significant Model Parameters
The wave transformation model uses the wave breaking method of Ruessink, Ref /4/, which is an elaboration of the model by Battjes and Janssen. Applied wave-breaking parameters in the model of Ruessink are: $\gamma_2=0.6$ and $\alpha=0.55$. The model parameters are determined after calibration against field measurements as well as physical model test including detailed surf zone information.

The waves in the local model area are calculated on the flexible computational meshes shown in Figure 4.1. The model calculates the distribution of wave height, wave periods, wave direction and spreading of waves and calculates radiation stresses which drive the longshore current.

### 4.2.2 Boundary Conditions
Two sets of wave transformations have been made. Initially the transformation of waves from the offshore location (63°N, 21°W) to the boundary of the local model (see Figure 4.1) is done by running the regional wave model described in Ref. /1/ and extracting wave parameters at several positions along a line corresponding to the boundary of the local model. The wave parameters vary significantly along the boundary of the local model area due to shadow effects of Westmann Islands. Wave conditions at 6 locations along the line corresponding to the boundary of the local model are extracted - see
Figure 4.5 where wave conditions at locations 3 to 8 are used as boundary conditions in the local model.

![Wave conditions diagram](image)

**Figure 4.5 Extraction points for near-shore wave time series**

In Figure 4.7 and Figure 4.8 wave roses at positions 2, 3, 4, 5, 6 and 7 (see Figure 4.5) are given for the modelling periods.

In connection with the fine tuning of the morphological modelling of the development of the bar in front of Bakkafjara it is realised that the linear interpolation of wave parameters in the sheltered zone behind Westman Islands is slightly too coarse to reflect the details of the bar migration. The wave transformation has therefore been repeated and wave parameters are extracted for each mesh cell along the outer boundary of the local model. The bathymetry and computational grid of the wave transformation model are shown in Figure 4.6. It is noted that the transformation to the individual positions 1 – 9 is identical in the two models. The new wave transformation adds the variation between the points.
Figure 4.6  Model bathymetry and computational mesh for the refined wave transformation
Figure 4.7 Wave roses for November 1985 along the boundary of the local model. Upper wave roses: position 2 and 3. Wave roses in middle: position 4 and 5. Lower wave roses: position 6 and 7. Positions are shown in Figure 4.5.
Figure 4.8 Wave roses for February 1989 along the boundary of the local model. Upper wave roses: position 2 and 3. Wave roses in middle: position 4 and 5. Lower wave roses: position 6 and 7. Positions are shown in Figure 4.5.
4.2.3 Validation of Wave Predictions
The combined regional wave transformation model and local wave model are validated against measured waves from the Bakkafjara wave buoy. The Bakkafjara wave buoy was deployed 18 November 2003 and has measured wave heights and wave periods continuously ever since. The wave buoy measurements do not provide information about the wave direction. The wave buoy is located on approximately 28 m water west of the proposed Bakkafjara Harbour (see Figure 6.1). The selected validation period is March 2004. During March 2004 the wave heights were among the largest recorded.

In Figure 4.9 the comparison between modelled and measured wave heights at Bakkafjara wave buoy is presented. It can be seen that the considerable wave height reduction taking place from offshore (south of Westmann Islands) to the location of the wave buoy is captured correctly by the model and, more essentially, all important spikes and lows found in the measured waves are well captured by the model.

Figure 4.9 Comparison between modelled (red dots) and measured (blue line) wave heights at Bakkafjara wave buoy, March 2004. The offshore wave heights are shown for comparison (black dots)

4.2.4 Modelling Examples
The effect on the distribution of wave heights (significant) of Westmann Islands is pronounced for south-westerly waves. Examples of the pronounced sheltering effect of the Westmann Islands are shown in Figure 4.10.
Figure 4.10  Examples of simulated wave fields in regional wave model. Waves from SW (top), S (middle), and SE (bottom). Notice the extensive sheltering provided by the Westmann Islands
Details of the wave transformation in the local model for cases where waves approach from south-west (upper) and south-east (lower) are shown in Figure 4.11.

![Figure 4.11: Snapshots of wave fields (significant wave heights) during the February 1989 storm. Upper: waves from south-west. Lower: waves from south-east.](image)

### 4.3 Flow

DHI’s hydrodynamic model MIKE 21 HD has been used to simulate the flow. MIKE 21 HD is a general numerical modelling system for simulating water levels and velocities in estuaries, bays and coastal areas. It simulates unsteady flows using depth integrated formulation (2D flow equations). The model includes all driving forces and phenomena that are important for flow in the nearshore zone such as Coriolis force, tides, storm surge, wave forcing (radiation stress), wind forcing, momentum dispersion and bed friction.

The solution is obtained with the finite volume approach on the triangular meshes presented in Figure 4.1.
4.3.1 Modelling of River Flow

The discharge from the river Markarfljot located approximately 2.5 km east of Bakkafljot is included in the hydrodynamical set-up. The river carries a significant amount of sediment which is sourced into the coastal zone during periods of heavy run-off and thus plays an important role in determining the morphological development. Markarfljot is very dynamic. Historic observations show that the location of the river mouth is shifting continuously. A bird view of the Markarfljot river outlet and the lower part of its river plain is shown in the upper part of Figure 4.12. The pronounced meandering and braiding of the river is a clear indication of the dynamic nature of Markarfljot river. In the lower part of Figure 4.12 the Markarfljot is viewed from the coastal plain itself and up river. Several active water falls (providing the river with water) are seen along the adjacent cliff (east of the river).

Figure 4.12  Markarfljot 5th December 2006. Note scattered water falls along the cliff in the picture below

The annual discharge at the river mouth from 1961 to 2001 has been calculated and evaluated in Ref. /3/. The average discharge was found to be 96 m$^3$/s but with significant seasonal variations. The peak discharge during the large flood of January 11$^{th}$, 2002 was estimated to 1500 m$^3$/s.

The discharge during simulated the two selected periods is presented in Figure 4.13 and Figure 4.14 (from Ref. /3/).

The amount of sand sourced to the coastal zone from Markarfljot river has a significant impact on the coastal sediment budget. The influence on the coastal morphology can be seen from the morphological calculation presented in Section 5. The amount of sediment carried by the river is calculated by MIKE 21. The sediment carried into the coastal zone during the two simulated periods is shown in Figure 4.13 and Figure 4.14. The accumulated sediment load is shown as well. It can be seen that nearly 20,000 m$^3$ sand was discharged from the river during November/December 1985. During February 1989 approximately 10,000 m$^3$ sand was discharged.
Figure 4.13 River discharge and sediment load during November 1985. Upper: river mouth discharge (Markarfljót) – From Ref. [3]. Lower: sediment load and accumulated sediment load.

Figure 4.14 River discharge and sediment load during February 1989. Upper: river mouth discharge (Markarfljót) – From Ref. [3]. Lower: sediment load and accumulated sediment load.
The calculated sediment load depends among other things on the river discharge, the grain size and the profile of the river cross-section. Both grain sizes and river discharges are well-documented. The profile of the river cross-section (i.e. the depth) is, however, unknown. During the 70 days of simulation the discharge of sand from the river was 30,000 m$^3$. The annual discharge of sand based on the simulated periods is therefore estimated to be 150,000 m$^3$ which is close to the well-known value of 100,000 m$^3$ (see Ref. /1/). The profile used in the model bathymetry is thus considered to be suitable.

The fine tuning of the morphological modelling complex around the bar was done for the periods between the bathymetrical measurements: 2004-2005 and 2006 -2007. No detailed information was available on discharges so the yearly variation averaged over the period 1962-2002 was used in these simulations. The yearly variation is illustrated in Figure 4.15.

![Figure 4.15 Average variation over the year of river discharge](image)

**4.3.2 Modelling of Tidal Flow**
Forecasted tidal elevations and velocities from March 2004 were made available at 8 positions just off Bakkafjara. The forecasted tidal elevation and velocity are extracted from a tidal model which is run on an operational basis at the Icelandic Maritime Administration to predict sea level and tidal currents in Icelandic coastal waters using a weather forecast from the European Centre for Medium Range Weather Forecasts. The
model is refereed to as the IMA model. The flow model, MIKE 21 HD, was calibrated to reproduce current speeds and surface elevations from IMA’s model.

In Figure 4.16 surface elevations and current speeds at a water depth of 10 m from MIKE 21 and from IMA’s model are compared for a full spring-neap period, March 2004.

![Figure 4.16 MIKE 21 model results (black line) and forecasted results from the Icelandic Maritime Administration (blue dots) surface elevation [m] (upper) and current speed [m/s] (lower) at water depth 10 m. Period: March 2004](image)

### 4.3.3 Modelling Results

In Figure 4.17 snapshots of the current fields (dominating currents are driven by radiation stresses) for cases where waves approach from south (upper) and south-west (lower) in the area near Bakkafljara are presented. The colours indicate the speed of the depth-averaged velocity whereas the vectors show the direction of the current (length of vector is proportional to the speed).

The snapshot situation with waves from south gives strong currents that are directed away from Bakkafljara and currents are strongest on the open coast west of Bakkafljara. The change in current direction is due to the distinctive change in coastline orientation at Bakkafljara.

The snapshot in the lower part of the figure demonstrates the shadow effect of Westmann Islands on the longshore current. The current attenuates along the bar and is weak east of Bakkafljara.
Figure 4.17  Bathymetry (May 2006) and snapshots of typical current fields
4.4 Sediment Transport

The wave/current induced sediment transport and the associated morphological evolution in the study area were obtained by DHI’s MIKE 21 ST. The transport rates and the morphological evolution are calculated on the flexible mesh presented in Figure 4.1.

The sediment transport is calculated every time step using the information from MIKE 21 SW (wave height, periods, directions, spreading) and MIKE 21 HD (currents, water levels).

The model requires information on mean grain size, the standard deviation and relative density of the sand. The sand at Bakkafjara is known to have a high density which is typical for basalt sand – see Figure 4.18.

![Example of the dark basalt sand which is characteristic at Bakkafjara](image)

For a few model parameters such as bed roughness and eddy viscosity, no measured data was available. These parameters were as a starting point given default values based on DHI’s experience in other, similar applications. Calibration and validation of the modelling complex have been undertaken by comparing simulated and observed morphological evolution on the bar in the periods 2004-2005 and 2006 -2007. Results from these comparisons are presented in Section 4.5.

4.4.1 Grain Size and Relative Density

An extensive number of sediment samples were collected in the vicinity of the proposed harbour along lines perpendicular to the coastline. The positions of the samples are shown on top of the bathymetry. Figure 4.19 also displays the measured mean grain size (the mean grain size is indicated in a palette of light to dark gray).
The samples show a tendency for sand to be finer offshore of the outer sand bar and in the trough. On the bar itself and at the coastline coarser sand is found. The mean grain size typically varies between 0.15 mm to 0.5 mm. The average size is 0.25 mm. The average value for the standard deviation, $\sigma$, for the area is 1.5, indicating relatively well-sorted material. The relative density of sand, $s$, is found to be close to 2.85, which is slightly above the value for quartz sand (=2.65).

4.5 **Calibration and Validation of the Morphological Modelling Complex**

In the period 18 July 2004 to 7 May 2005 the end of the western bar lengthened considerably towards the east and the “spit” formation on the eastern side of Bakkafjara was eroded away. The two measured bathymetries are shown in Figure 4.20. Clearly, this period was dominated by east-going littoral transport. The period has been selected for fine tuning of model set-up and parameters due to the very significant morphological changes in this period.
The following model set-up conditions and parameters have been tested and improved since the first model-set:

- Non-linear variation of wave parameters along the offshore boundary
- Mesh resolution on and around the bar
- Settling velocity of the bed material (grain size)
- Hydraulic roughness of the hydrodynamic model

The final computational mesh is shown in Figure 4.1B. The computational mesh is a trade off between length of computational time and accuracy of the results. In the present case satisfactory results are found with a resolution around 50 m over a large area around the very dynamic depression in the bar. Initially a median grain size of 0.25 mm was applied all over the model area. This value is close to the observed grain sizes. However, maybe due to the sharp edges of the grains, better results are obtained with slightly finer grains corresponding to 0.20 mm. Further, the optimisation of the modelling results has lead to a hydraulic roughness expressed by the Manning number of 40 m$^{1/3}$/s.
The morphological modelling complex has been initiated with the observed bathymetry of June 2004. In the first trial runs the entire period was modelled. However, for the final simulations the calm summer months were cut out to reduce computational time. The final simulation covers November 2004 to March 2005. Sediment transport is highly non-linear with wave heights and the reduction of the time series therefore does not influence the results significantly. Results from the final calibration are shown in Figure 4.21. Here the initial model bathymetry and the final model bathymetry are shown. The two figures correspond to the similar figures of observed bathymetries, see Figure 4.20.

**Figure 4.21**  Upper panel: observed bed level, July 2004 (initial model bathymetry)  
Lower panel: calculated bed level May 2005
It appears from Figure 4.21 that the model successfully reproduces the about 500 m lengthening towards the east of the western bar and the erosion of the eastern “spit”. The observed and modelled bed level changes are shown in Figure 4.22. The development of the main features, the lengthening of the bar and the erosion of the spit and the deposition seaward of the spit are clearly recognised. Further, the observed erosion along the seaward side of the western bar is well reproduced by the numerical model.

The calibrated model set-up has been validated against the morphological development from 2006 to 2007. The bathymetrical measurements from 2007 cover only a very small area in front of the planned harbour. However, these observations are very important as this period represents the critical evolution from a situation where the western bar has developed all the way to the river delta towards re-establishment of the depression. The bathymetry measurements are shown in Figure 4.23.
Figure 4.23 Measured bathymetries, May 2006 and January 2007

The initial model bathymetry (May 2006) and the modelled bathymetry (January 2007) are shown in Figure 4.24. The corresponding bed level changes, observed and modelled, are compared in Figure 4.25.

Figure 4.24 and Figure 4.25 clearly show that the western bar was developed far to the east after the winter storms in 2005/06. In the period from 2006 to the measurements in January 2007 the bar was eroded in front of the location of the harbour (544000 m E). The observed development is reproduced in the numerical modelling complex. The bar is eroded more than 1 m across the bar in the navigation line.
Figure 4.24  Upper panel: initial model bathymetry, May 2006
Lower panel: calculated bed level, January 2007
Figure 4.25  
Upper panel: modelled bed level changes, May 2006 to January 2007  
Lower panel: observed bed level changes, May 2006 to January 2007 (in a small area only)
5 ANALYSIS OF SEDIMENT TRANSPORT AT OUTER BAR

In the following the sediment transport field and the morphological evolution during the selected periods are presented. The following items are covered:

- Period-average transport fields for the selected periods
- Role of cross-shore transport during the selected periods
- Morphological evolution (erosion/deposition patterns) with focus on the navigation line and the bar depression

The main attention of this section is focused on the understanding of the dynamics of the outer bar. It is well-known that the water depth along the outer bar varies along the coast with a maximum in water depth at a location near Bakkafjara. This depression in the outer bar is a persistent feature and confirmed by historical records as well as more detailed surveys conducted during the past 6 years. Recent surveys during the last 6 years indicate, however, that the water depth of this depression is decreasing. In this section, the morphological developments on the outer bar will be investigated in detail through mathematical modelling supported by comprehensive field measurements available at this site.

5.1 Period-averaged Sediment Transport Field

In Figure 5.1 and Figure 5.2 the period-averaged sediment transport fields for the November 1985 and February 1989 storm are shown.

Both periods include waves from south-westerly and south-easterly directions; however, the main storm in November 1985 is south-easterly dominated. The overall characteristic of the November 1985 storm is that sediment is transported to the west, west of Bakkafjara and to the east, east of Bakkafjara. The important aspect of this is that the point of inflexion (divergence in sediment transport) is located over the bar depression which is found just east of the proposed ferry port. Thus, during this period, or more generally, periods of south-easterly storms, the depression in the bar is maintained.

The overall characteristic of the February 1989 storm is eastward directed sediment transport. Sediment is thus transported by the longshore current on the outer bar towards the bar depression. Normally, this would lead to filling up the gap in the bar, however, looking more closely at the transport field it is realised that a small offshore deflection in the transport direction is induced. The physical explanation of the deflection is discussed in more detail later; however, the impact of such deflection in the transport direction is that the bar-depression is preserved.

Thus, simulation of the two periods clearly reveal mechanisms that maintain the bar depression.
5.2 Cross-shore Transport Capacity

The sediment transport results presented above were obtained with a sediment transport description which is based on depth integrated flow description, where the direction of the sediment transport was assumed to be in the direction of the depth-averaged flow. In this section sediment transport associated with complex 3D flow patterns are evaluated over a fixed bed. This is done as 3D flows in the surf zone mobilise cross-shore transports, examples of mechanisms that induce such transport are:

- Streaming in the wave boundary layer and wave drift
- Undertow
- Net bed shear stress from asymmetrical waves
• Wave-current motion (produce net cross-shore transports due to wave-current non-linearity)
• Helical motion from centrifugal forces

Cross shore sediment transport determines the shape and the dynamics of the cross-shore profile, including the bar. The net cross-shore transport and the associated cross-shore profile evolution are the result of two opposing components (e.g. onshore and offshore directed transport) with magnitudes that often are considerably larger than the net transport. Small errors or inaccuracies in the description of the balance between the onshore - and offshore directed forces thus lead to relatively large errors in the calculated net cross-shore transport rates. In morphological models, these errors accumulate and sometimes lead to increasingly unrealistic cross-shore profiles. Therefore, in many applications where the morphodynamics are dominated by the longshore transport, rather than cross-shore transport, the complex 3D effects on the sediment transport are simply turned off. This makes the models more robust and reliable with regard to morphological evolution dominated by longshore sediment transport. In the present case the bathymetrical surveys indicate that the distance from the bar to the coastline is fairly constant, suggesting that cross-shore sediment transport is not a dominant phenomenon for the present case. Therefore, neglecting the 3D effects on sediment transport seems justified for the morphological modelling.

In this section, the effects of the 3D flow mechanisms on the sediment transport are evaluated. Figure 5.3 shows the simulated sediment transport component normal to the depth-averaged flow, which is associated to the 3D mechanisms mentioned above. The transport rates are averaged over the modelling period i.e. February 1989. The transport rates shown here must thus be seen in addition to the transport rates shown in Figure 5.2 which were calculated according to a depth-averaged flow description.

The model simulations indicate onshore directed sediment transport on the seaward side of the bar. Closer to the bar crest, wave breaking is more intensive and the sediment transport is directed offshore due to the wave generated undertow. In the trough the transport is directed onshore. The period-averaged cross-shore transport pattern will tend to widen the outer bar of May 2006.

The period-averaged cross-shore transport along the bar is seen to reduce towards the gap. This decrease in longshore sediment transport is associated to a decrease in wave height along the shore, caused by the sheltering effect of the Westmann Islands. Close to the gap the offshore directed transport suddenly increases somewhat and reaches a local maximum. On the east side of the gap the period-averaged cross-shore transport was found to be onshore directed.
5.3 Erosion and Deposition Patterns

In Figure 5.4 the modelled development from 1. November 1985 to 12. December 1985 is shown. The following important morphological activity has taken place during November 1985:

- Accretion of sediment near the river outlet (between 0 m and -2 m)
- Deposition of sediment near the breakwaters (between 0 m and -6 m) especially on the east side. The coastline has advanced approximately 60 m on the east side
- No significant changes in the deep trough located between the harbour mouth and the bar are seen
- Changes at water depths up to 14 m, i.e. along the outer side of the bar
- Only small changes in navigation depth in the mouth of the breakwaters (see also Figure 6.13).
- Lowering of the bar in front of the harbour.
- Simulation indicates the formation of a bar in front of the harbour entrance (i.e. a reduction in depth compared to the undisturbed initial water depth)

The latter morphological development is investigated in-depth in Section 8.
Figure 5.4  Morphological changes during November 1985 (initial bathymetry May 2006). Upper plot: initial bathymetry, Middle plot: end bathymetry, Lower Plot: Morphological changes.
The following important morphological activity has taken place during February 1989:

- Accretion of sediment near the river outlet (between 0 m and -2 m) with a clear displacement of the sand towards east
- Deposition of sediment near the breakwaters (between 0 m and -6 m). The coastline has advanced approximately 50 m and 35 m on the east and west side of the breakwater, respectively
- Small changes in the deep trough located between the harbour mouth and the bar are seen
- Deposition on the bar to the west of the harbour
- General erosion to the east of the harbour out to the -6 m contour
- Lowering of the bar in front of the breakwaters
- Very small changes in the navigation depth in the mouth of the breakwaters (see also Figure 6.13).
- Simulation indicates the formation of a bar in front of the harbour entrance (i.e. a reduction in depth compared to the undisturbed depth)

The latter morphological development will be investigated in more detail in Section 8.
Figure 5.5  Morphological changes during February 1989 (initial bathymetry May 2006)
Upper and middle plots: isolines: initial bathymetry and end bathymetry
Lower plot: evolution during February 1989
The tendency of the outer bar to break up while being eastwardly displaced under conditions with eastward transport is worth noting. These changes in the morphological pattern are due to the seaward deflection in the transport briefly mentioned in the previous section. The deflection of sediment on the outer bar is caused by a cross-shore transport mechanism pronounced at the location of the bar depression (i.e. at Bakkafjara). In Section 5.2 period-averaged transports perpendicular to the depth-averaged flow originating from 3D flow phenomena are presented; however, these do not show particularly pronounced cross-shore transports at the bar depression. In Figure 5.6 instantaneous sediment transport patterns are shown for conditions with strong eastward transport. The figure shows that while sediment is moving parallel with the bed contours both east and west of Bakkafjara, a considerable deflection of the transport (compared to the contours of the bar) at Bakkafjara is seen. The deflection of sediment can be caused by several mechanisms; however, most likely is the formation of a rip current generated by the alongshore variations in the set-up (from wave breaking). On an open coastline rip currents are normally caused by variations in the height of the bar and are very mobile. In the present case the mechanism can be caused by the variation in the wave height. A variation in the wave height is seen due to the shadow effect of Westmann Islands. Behind the Westmann Islands waves are smaller and consequently the set-up here is reduced. This gives an alongshore gradient in the surface elevation driving a current. This current is in addition to the longshore current. The current turns seaward at the point of the smallest set-up. The location of the smallest set-up varies with the direction of the wave such that the location where sediment is being pushed seaward by the rip changes.

Figure 5.6  Instantaneous transport profiles along the Bakkafjara coastline (storm, February 1989)

The magnitude of the rip current is controlled by the gradient in the surface elevation. In Figure 5.7 the surface elevation for conditions with waves from south-west and south is depicted. The figure clearly shows the variation in water level along the coastline with south-westerly waves changing from 1.4 m at approximately 4 km west of Bakkafjara to around 1.15 m in the sheltered area near Bakkafjara. The difference in surface elevation is approximately 25 cm which is of a magnitude fully capable of driving a relatively strong current. For the southerly waves the gradient increases even further with a change in surface elevation 30 cm over 800 m.
The rip-current will maintain a depression in the outer bar; however, the dimensions of the depression are a function of the wave height and wave direction. The dimensions are determined by the continuous “battle” between the scouring effect of the rip current and the infill of sediment transported by the longshore current along the outer bar on the up-drift side and erosion on the down-drift side. The longshore current is strongest when the wave angle is 45 degrees to the coastline and weakest when waves attack perpendicularly, thus dominant rip scouring may be expected when the waves approach from SSW (or SSE).

The mechanism is not limited to south-westerly wave conditions. It can be initiated during southerly storms as well – see Figure 5.7.
5.4 Sediment Transport Field with Bar Modifications

To further understand the morphological features and governing processes at Bakkafjara two scenarios were tested where the bathymetry was artificially modified. The objective of these tests was to study the morphological response of the system and its capacity to recover after an extreme event.

The two scenarios included:

1. a pile of sand is placed on the bar and in front of the breakwaters,
2. sand has been excavated from the bar in front of the breakwaters

Both scenarios were tested using the morphological model and simulating the February 1989 event. (Note that these scenarios are tested prior to the final fine tune of the morphological complex. The fine tuning has mainly led to a change in time scale, whereas the morphological features and development of features have not changed). The results are presented below.

5.4.1 Scenario 1: Reduced Water Depth on Outer Bar

The modifications of the outer bar are made in the vicinity of the navigation line and such that the water depth over the bar is reduced to 2 m over approximately 300 m in the alongshore direction in the first mentioned case.

In Figure 5.8 the model bathymetry with the described modifications in the morphology is shown. The white rectangle indicates the model output area as shown in Figure 5.10.

![Figure 5.8 Model bathymetry for morphology with deposition on bar](image)

The simulated sediment transport field for February 1989 is shown in Figure 5.9. The upper part of the figure shows the period-averaged transport, the lower part shows a snap-shot of the transport field.
The period-averaged transport for February 1989 is seen to be affected significantly by the crest elevation of the bar near the navigation line. The morphology is very active. The combined action of currents and waves causes the pile of sand to erode. The eroded sand is seen to be transported landward. The landward transport is due to the flow induced by the more intense wave breaking over the elevated part of the bar.

The morphological changes in the area indicated in Figure 5.8 are presented in Figure 5.10. The water depth along the original bar is nearly re-established at approximately 5 m. Eroded sand has not been supplied to the bar east of the pile. The pile has simply moved landward reducing the volume of the deep pit-type trough located in front of the harbour entrance.
Figure 5.10  Morphological changes during February 1989: top: initial situation, bottom: final situation

5.4.2 Scenario 2: Excavation of the Bar

The water depth over the bar in the case where the bar has been excavated is increased to 7 m over a distance of 150 m.

In Figure 5.11 the model bathymetry with the described modifications in the morphology is shown. The white rectangle indicates the model output area as shown in Figure 5.13. In Figure 5.12 the period-averaged sediment transport over the 1989 storm is presented.
In the upper part of the figure the period-averaged transport is shown and in the lower part a snap-shot of the transport field is shown. During periods of moderate longshore transports the processes are easily identified.

For the present case the bar morphology is less active compared to the situation where sand has been deposited. However, the level of the excavation is seen to be maintained with a slight tendency for further deepening. Sand is seen to be transported seaward. The reason for the seaward transport is due to the rip current induced by the less intense wave breaking over the lower part of the bar.
The morphological changes are presented in Figure 5.13. It is seen that the excavation is displaced in the eastward direction by approximately 70 m (the 6 m contour line). The water depth in the central part of the excavation is increased by 10-20 cm over the period. This case shows exactly the two mechanisms at play. The resulting level of the bar is a continuous battle between the rip current induced erosion and the deposition of sand caused by the longshore current. The present storm is seen to displace the depression eastward and although the water depth over the depression is increased the depth at the navigation line is decreased. The simulated bed levels were extracted along a line across the excavation, shown as a black dashed line in Figure 5.13. Figure 5.14 shows the simulated bed levels before and after the storm.
5.4.3 Concluding Remarks on Simulated Scenarios

The simulations with the modified bathymetries clearly demonstrate that the cross-shore transport related to depth integrated currents over the outer bar at Bakkafjara is responsible for the existence of the depression. In the case of sand being piled up on the outer bar the shoreward transport mechanism is seen to be very dominating. The pile is eroded and the material is disposed behind the bar. In the case of excavating part of the outer bar the bar is more stable implying that the excavated area with water depth of 7m is closer to being in equilibrium with the forcing conditions of the selected period.

This investigation shows that the depression in the outer bar is a persisting feature in the morphology and that the reason for the depression is the transport induced by along-shore variation in the wave set-up.
5.5 **Effect of the Harbour on the Bar**

The harbour is planned to be built landward of a large natural trough between the outer bar and the inner bar. The depth of the trough is about 12 m; it extends almost constantly about 1 km along the coast and holds a width of about 2-300m, see Figure 2.3.

The influence of the harbour on this large scale phenomenon has been studied by simulation of the February 1989 storm with and without the harbour. The average sediment transport patterns are compared in Figure 5.15. The net bed level changes are compared in Figure 5.16. It appears that the harbour blocks the sediment transport on the inner bar, which is directed around the harbour, see the discussion in Section 9. The sediment transport and thereby the morphological evolution on the bar is only marginally influenced by the harbour. However, a slight tendency to seaward displacement of the bar and tendency for lowering of the crest due to the harbour can be observed.

![Figure 5.15 Net sediment transport pattern with and without harbour. Initial bathymetry May 2006. Hydrographic scenario: February 1989](image)
5.6 Natural Changes in Bar Depression

In above sections the relationship between the sheltering effect of the Westmann Islands (inducing a rip current) and the presence of a depression in the outer bar at Bakka fjara has been made evident through numerical modelling. In this section the historical data is revisited and the rip-current hypothesis is tested. Figure 5.17 shows the wave energy averaged over the period September – April for each year between 1959 and 2005. The data originates from the offshore location (63°N, 21°W). Similarly, the wave direction is shown in Figure 5.18.
The average wave conditions for the period where the bathymetry was measured at regular intervals are shown in greater detail in Figure 5.19 and Figure 5.20.
Figure 5.20  Monthly-averaged offshore wave direction from Jan 2000 to Oct. 2006

To supplement the time-averaged wave-energy and wave directions presented above, the observed minimum water depth over the outer bar (at the position of the navigation line) is shown in Figure 5.21 as well as the measured depths along the navigation line for the years 2004, 2005, 2006 and 2007. (Note, the profile measurements are slightly smoothed as these were used as model bathymetries).

The results from morphological modelling of the periods 2004 to 2005 and 2006 to 2007 are shown for comparison, see also Figure 4.21 and Figure 4.23.
Figure 5.21  Upper panel: Minimum measured water depth at the outer bar at the position of the navigation line. Middle panel: Blue and black lines are measured depths along navigation line in 2004 and 2005. Modelled depths are shown in red for comparison. Lower panel: Blue and black lines are measured depths along the navigation line in 2006 and 2007. Modelled depths are shown in red for comparison.
These depths are based on the bathymetrical surveys conducted annually since 2002. The water depth over the outer bar at the navigation line decreased in the period from 2002 to 2006, see Figure 2.3 and Figure 5.21. However, the most recent bathymetric survey from January 2007 shows a clear increase in the minimum water depth, see also Figure 4.23.

A variation in water depths of approximately 2 m over the past 5 years is not surprising when taking into consideration the large variability in the offshore wave direction and wave energy. Shallow water depths (over the outer bar depression) were seen in the May 2006 bathymetry whereas large water depth was seen in the October 2002 bathymetry. If the October 2002 bar morphology (where the outer bar water depths were the largest surveyed so far) is studied closely (see Figure 2.3) a very distinct difference compared to later bathymetries can be recognized namely that the eastern side of the deep trough is seen to scour the outer bar. In later surveys the trough is seen to be located behind the outer bar. The question is what makes the October 2002 bathymetry different? If the wave and sediment transport climate a few months prior to the 2002 survey is studied the following interesting conditions can be identified:

- Two (2) very significant storms were observed west of Bakkafjara. Both these storms were among the 6 most severe storms in terms of wave exposure (in the period from 1979 to 2004). This is documented in Table 2a, Ref. /1/

- These storms did not give rise to longshore transport rates that match the severity in terms of wave exposure. In fact both storms did not produce longshore transport rates in the Top 20 – see Table 2b, Ref. /1/

- These storms did not produce significant wave exposure or longshore transport rates at Bakkafjara – see Table 3a and 3b, Ref. /1/

Storms that have the combination of great wave exposure west of Bakkafjara, small wave exposure at Bakkafjara and relatively low rates of longshore transport rates west and at Bakkafjara, are a prerequisite for strong rips and thus for the bar depression to deepen. Due to the relatively small longshore transport rates the rip current induced scouring is not counteracted by the infill provided by the longshore transport.

By studying Table 2a through Table 3b in Ref. /1/ it is realised that conditions similar to those itemized above are not unusual. In fact, such conditions can be identified in the tables in the years 1983, 1992, 1993 and 2001. In other words, the return period may be approximately 10 years which is the same return period derived from the analysis of the offshore waves.

Water depths along the bar and over the depression in particular are adapting continuously to changing wave and current conditions. Depending on the characteristics of the storm the bar depression can be either maintained at a level close to 6.0-5.5 m or under certain rough conditions deepen to the level observed in the October 2002 survey. Few Top 20 wave exposure storms have been observed at Bakkafjara since 2002 (except for the March storm 2003). At the same time, two Top 6 longshore sediment transport storms at Bakkafjara were documented – see Ref. /1/. In other words, the infill provided by the longshore current has been the dominating mechanism in the period 2002 to 2006.
The period May 2006 to Jan 2007 has been modelled in detail as part of the validation of the morphological modelling complex. The offshore wave conditions for the rough winter months of this period are shown in Figure 5.22. It appears that this period is dominated by storms from south-south westerly directions, which as explained lead to lowering of the bar in front of the harbour.

Figure 5.22  Offshore wave conditions in the rough winter months of the validation period
6 WAVES, CURRENTS AND BED LEVELS ALONG NAVIGATION LINE

In this section waves and flow along the navigation line are presented. Both the wave and current conditions are important for navigation. In addition, the simultaneous wave conditions at the Bakkafjara wave buoy and at two locations along the navigation line are investigated. The latter is important for the establishment of a transformation rule between waves measured at the buoy and waves along the navigation line.

6.1 Waves along the Navigation Line and at the Wave Buoy

The relation between the waves at the Bakkafjara wave buoy and the navigation line located in front of Bakkafjara Harbour is investigated by MIKE 21 SW. This was done to see if the transformation rule applied in the physical test facilities at Siglingastofnun could be verified by MIKE 21 SW. The location of the wave buoy (t3) and the navigation line (t2 to t1) is shown in Figure 6.1.

![Figure 6.1](image)

*Figure 6.1 Upper: location of wave buoy (t3) and navigation line (t1 to t2). Lower: depth along navigation line [m]*
MIKE 21 SW was applied to simulate a number of selected wave events (referred to by Siglingastofnun to “critical waves” and “98% waves”). The model was set up such that the simulated wave height at the location of the wave buoy corresponds to the wave conditions listed in Table 6.1 and Table 6.2. Wave heights in Table 6.1 and Table 6.2 are significant wave heights and tabulated as function of wave direction and surface elevation.

Table 6.1  Critical waves: significant wave heights at Bakkafjara wave buoy for different surface elevations and wave directions

<table>
<thead>
<tr>
<th>Surface elevation</th>
<th>Occurrence of direction [%]</th>
<th>+2.3m</th>
<th>+1.4m</th>
<th>+0.5m</th>
<th>0.0m</th>
</tr>
</thead>
<tbody>
<tr>
<td>+2.3m</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S</td>
<td>22.5</td>
<td>3.8m/11s</td>
<td>3.7m/11s</td>
<td>3.5m/11s</td>
<td></td>
</tr>
<tr>
<td>SW</td>
<td>51.7</td>
<td>3.9m/11s</td>
<td>3.8m/11s</td>
<td>3.5m/11s</td>
<td>3.4m/11s</td>
</tr>
<tr>
<td>SE</td>
<td>25.8</td>
<td>3.8m/11s</td>
<td>3.6m/11s</td>
<td>3.5m/11s</td>
<td>3.4m/11s</td>
</tr>
<tr>
<td>Weighted</td>
<td>100</td>
<td>3.85m/11s</td>
<td>3.70m/11s</td>
<td>3.50m/11s</td>
<td>3.40m/11s</td>
</tr>
</tbody>
</table>

Table 6.2  98% waves: significant wave heights and wave periods at Bakkafjara wave buoy for different surface elevations and wave directions

<table>
<thead>
<tr>
<th>Surface elevation</th>
<th>+2.3m</th>
<th>+1.4m</th>
<th>+0.5m</th>
<th>0.0m</th>
</tr>
</thead>
<tbody>
<tr>
<td>+2.3m</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>South</td>
<td>2.8m/7.6s</td>
<td>2.8m/7.6s</td>
<td>2.8m/7.6s</td>
<td></td>
</tr>
<tr>
<td>South-west</td>
<td>2.5m/8.8s</td>
<td>2.5m/8.8s</td>
<td>2.5m/8.8s</td>
<td>2.5m/8.8s</td>
</tr>
<tr>
<td>South-east</td>
<td>2.8m/6.8s</td>
<td>2.8m/6.8s</td>
<td>2.8m/6.8s</td>
<td>2.8m/6.8s</td>
</tr>
</tbody>
</table>

In Table 6.1 “weighted wave heights” are given as well. Weighted waves are found in the following way:

\[
H_{s,\text{weighted}} = \%(\text{South}) \times H_{s,\text{south}} + \%\text{(Southwest)} \times H_{s,\text{southwest}} + \%\text{(Southeast)} \times H_{s,\text{southeast}}
\]

where %() = occurrence of a wave from a given direction.

The waves are seen to vary nearly linearly with the surface elevation. Particularly, the variation of the weighted wave follows the following approximation:

\[
H_{s,\text{weighted}} = 3.4 + 0.2 \times WL
\]

\[1/\]
where \(WL\) = surface elevation. The variation of the critical wave heights is shown in Figure 6.2 as function of the surface elevation for waves from south, south-west, south-east and the weighted waves (in grey). The approximation given in Eq /1/ is shown as well (in black).

![Figure 6.2 Wave height (critical waves) with surface elevation for waves from south, south-west and south-east](image)

As mentioned previously, MIKE 21 SW was set-up to calculate wave fields that match wave conditions at the Bakkafjara wave buoy (i.e. the waves in Table 6.1 and Table 6.2). This was done iteratively by first selecting typical wave events with wave directions from south, south-west and south-east and typical wave height distributions along the boundary of the model. The distribution of the wave height along the boundary of the model was scaled to give the wave heights in Table 6.1.

The wave heights calculated by MIKE 21 SW at the location of the Bakkafjara wave buoy were obtained with less than 3% discrepancy compared to those given in Table 6.1 and Table 6.2.

The wave model was run for 3 different spreading indexes, \(n\), for all the cases given in Table 6.1. Directional spreading factors of \(n=5, 10\) and 20 were investigated (corresponding to the directional standard deviation of 23°, 17° and 12°, respectively). The modelled wave heights at 10 m and 6 m depth contour (along the navigation line) did not vary significantly with \(n\); in fact results for \(n=10\) and 20 differed by less than 3% when compared to \(n=5\). Thus, the spreading factor is of much less importance than other factors in the wave modelling.

In Figure 6.3 is an example of the simulated waves for the case where waves are approaching from south and the surface elevation is +2.3m. The plot includes information...
on the position of the Bakkafjara wave buoy and the navigation line and contour lines in
black show the water depth (water depth of May 2006). The figure is referred to as a
Sea State Information Chart. The wave heights are seen to decrease in the alongshore
direction. In Appendix C Sea State Information Charts for all cases given in Table 6.1
i.e. critical waves are presented. In Appendix D Sea State Information Charts for cases
given in Table 6.1 i.e. 98% waves are presented.

Figure 6.3 Sea State Information Chart. Example of wave field for critical waves near the navigation
line for water surface of +2.3 m and southern waves with indication of depth contours
(shown with black lines). Indication of navigation line, extraction points and position of wave
buoy (T3) is given

In Figure 6.4 the extracted wave heights along the navigation line (indication of extraction line is given in Figure 6.1 and Figure 6.3) for scenarios given in Table 6.1 (critical waves) are presented.

The variation in wave heights is seen to be influenced by shoaling and eventually wave
breaking. The variation of the wave height through the surf zone is important for any
costal study; however, usually site-specific measurements or other information on how
waves behave inside the surf zone are very limited. In this project, however, measurements are in fact available. The wave heights in the surf zone have been measured in a
physical model established at the test facilities at Siglingastofnun. The model set-up and
scale are described thoroughly in Ref. /2/. The measurements have been carried out for a
range of surf elevations and directions at certain positions along the profile of the physical
model bathymetry. Measurements are included in Figure 6.4 (red, green and black dots)
and form an excellent opportunity for fine-tuning the wave-breaking model, see wave pa-
rameters section.
Comparison between wave heights obtained from the numerical model (MIKE 21 SW) and the physical model (developed at physical test facilities at Siglingastofnun) along the navigation line is given in Appendix B for all wave directions (south, south-west and south east) and surface elevations. The numerical wave model is seen to capture the measured variation in wave height well taking into consideration that the physical model is using uni-directional waves and that the wave model does not consider wave reflection.

The wave heights from MIKE 21 SW at 10 m and 6 m water depth on the seaward facing side of the bar (see Figure 6.1) are given in Table 6.3 for the wave conditions lined up in Table 6.1. The values are average values for different locations of the navigation line (average of 3 parallel navigation lines spaced approximately 25 m). This is done due to the significant variation in wave heights along the depth contour. Values presented in Table 6.3 are close to those obtained in the test facility of Siglingastofnun, see Ref. /2/, Table 1.

Table 6.3  Significant wave heights [m] at 10 m/6 m depth contour (CD) along the navigation line for different surface elevations and wave directions. Water depth at crest of the bar is 6m CD

<table>
<thead>
<tr>
<th>Surface elevation</th>
<th>Wave direction</th>
<th>+2.3m</th>
<th>+1.4m</th>
<th>+0.5m</th>
<th>0.0m</th>
</tr>
</thead>
<tbody>
<tr>
<td>South</td>
<td></td>
<td>4.3m/4.1m</td>
<td>4.2m/3.9m</td>
<td>3.8m/3.5m</td>
<td></td>
</tr>
<tr>
<td>South-west</td>
<td></td>
<td>4.5m/4.1m</td>
<td>4.2m/3.8m</td>
<td>4.1m/3.4m</td>
<td>3.6m/3.3m</td>
</tr>
<tr>
<td>South-east</td>
<td></td>
<td>4.6m/4.3m</td>
<td>4.5m/4.0m</td>
<td>4.2m/3.6m</td>
<td>3.9m/3.0m</td>
</tr>
</tbody>
</table>
6.2 **Currents along the Navigation Line**

Figure 6.5 presents examples of Sea State Information Charts for wave driven currents. The Bakkafjara wave buoy and the navigation line are indicated and contour lines in black show the water depth (water depths of May 2006). Examples correspond to cases defined in Table 6.1 (for water level of 1.4 m CD).
6.3 Waves along the Navigation Line during Selected Storms

The wave conditions along the navigation line during the storms of November 1985 and February 1989 at the positions shown in Figure 6.6 are shown in Figure 6.7 and Figure 6.8. The positions indicated in Figure 6.6 are defined in terms of water depths and geographical coordinates in Table 6.4.

Table 6.4 Location and depth of critical points along the navigation line – see also inset figure

<table>
<thead>
<tr>
<th>Coordinates in [UTM-27]</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Entrance</td>
<td>Centre of</td>
<td>Top of bar</td>
<td>Seaward</td>
</tr>
<tr>
<td></td>
<td>of harbour</td>
<td>deep trough</td>
<td></td>
<td>facing side</td>
</tr>
<tr>
<td>Depth [m CD]</td>
<td>6.8m</td>
<td>11.6m</td>
<td>6.0m</td>
<td>9.1m</td>
</tr>
</tbody>
</table>

In Figure 6.7 the significant wave heights are presented whereas Figure 6.8 presents the mean direction of the waves. The wave heights are seen to vary during the simulations with several peaks reaching nearly 6 m at position 4 (9.1 m water depth). The waves decrease further inshore although the significant wave heights are seen as large as 4 m close to the entrance of the harbour in both simulations.
Figure 6.7 Variation in significant wave heights [m] at critical locations (see Figure 6.6) along the navigation line. Upper: November 1985, Lower: February 1989
6.4 **Currents along the Navigation Line during selected Storms**

In Figure 6.9 and Figure 6.10 the maximum and mean current speeds across the navigation line are presented for the two periods simulated, respectively. The mean speed during November 1985 (see Figure 6.9) is seen to be around 0.2 m/s on and inshore of the bar. Offshore of the bar the mean speed is determined by the tidal flow. During February 1989 the mean speed is larger averaging 0.4 m/s on and offshore of the bar. The maximum velocities modelled during these periods are found just offshore of the bar crest and reached 0.85 m/s and 1.70 m/s west-going in November 1985 and east-going February 1989, respectively.

In front of the harbour (300 m on the x-axis) the velocity is seen to have a maximum velocity of 0.8 m/s and 1.2 m/s for the 1985 and 1989 period, respectively. The velocities at this position are higher than on an open coast due to the contraction of streamlines around the harbour concentrating the flow in front of the entrance.
The current speeds during November/December 1985 and February 1989 at the positions indicated in Figure 6.6 are shown in Figure 6.11.

The calculated maximum velocities in the surf zone are sensitive to various set-up parameters including the bed roughness and eddy viscosity. The bed roughness is introduced as a Manning Number and the eddy viscosity through a constant factor on the turbulence model. Applied values are central estimates based on experience from previous type projects.
6.5 **Morphological Changes along Navigation Line**

In Figure 6.12 the modelled bed level changes at the 4 locations shown in Figure 6.6 are shown. Not surprisingly, the morphological activity is seen to be pronounced during periods of significant wave action. During the period of November 1985 the storm managed to modify the bathymetry especially on the bar. In the deep trough behind the bar (point 2) no changes were found; however, on the top of the bar (point 3) deposition of approximately 20 cm was found and slight erosion was seen on the sea facing side of the bar (point 4). Deposition in front of the harbour mouth was also seen (approximately 10 cm).

Generally, the changes in the navigation line are small in the period and are connected to the storms from 16-19 November. During the period of February 1989 no significant morphological activity was seen in the navigation line (fluctuation is less than 5 cm) except on the bar crest. The bar crest, however, was very mobile with an erosion of nearly 40 cm and a subsequent deposition of 50 cm. (It is noted that these results are obtained before the final fine tuning of the modelling complex. The results obtained on the bar development are discussed in Section 5.6. It appears that both storm periods, November 1985 and February 1989 led to erosion of the outer bar, when the May 2006 bathymetry was used as the initial bathymetry).
Figure 6.12  Changes in bed level [m] at critical locations (see Figure 6.6) along the navigation line, November 1985
In Figure 6.14 the longshore sediment transport profiles west and east of Bakkafjara and at Bakkafjara are depicted. The location of the profiles or cross-sections is shown in the upper panel. The transport shows peaks in the transport rates near the crest of the bar; however, the results for the two periods are very different. During November 1985 (red line in the figures) the net transport west of Bakkafjara is to the west (negative values) whereas east of Bakkafjara the net transport was found to be eastward. At Bakkafjara, the transport changes direction over the profile and is close to zero. This pattern is not seen for February 1989. Here the net transport was to the east in all cross-sections, although significantly smaller at Bakkafjara.
Figure 6.14  Net sediment transport distribution at three cross sections around the Bakkafjara Harbour (see upper panel for location) for the two storm periods, November/December 1985 and February 1989. Blue line: November 1985, Red line: February 1989. Negative values: west-going transport
In Table 6.5 the net integrated transports west, at and east of Bakkafjara are quantified for the two simulated periods.

<table>
<thead>
<tr>
<th>Period</th>
<th>West of Bakkafjara (m³)</th>
<th>At Bakkafjara (m³)</th>
<th>East of Bakkafjara (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nov/Dec 1985</td>
<td>-210000</td>
<td>-97000</td>
<td>34000</td>
</tr>
<tr>
<td>Feb/Mar 1989</td>
<td>310000</td>
<td>140000</td>
<td>190000</td>
</tr>
</tbody>
</table>

### 6.6 Estimated Annual Transports

The selected storm periods contain both periods with westward- and periods with eastward transport. The annual longshore transport rates can be estimated from the transports during the simulated storm events by assigning weight factors to each storm event and multiplying these weights with the accumulated transport rates for each event.

In order to derive appropriate weighting factors, not only the net transport but also the gross transport (e.g. the sum of the eastward and westward transport) must be taken into account. This is especially important for a case like Bakkafjara, where the net annual transport is rather small compared to the total west- and east going transport rates.

In order to estimate the average annual transport rates, the transport rates corresponding to the simulated storm events must be related to the average annual wave statistics.

In this analysis the littoral transport is assumed to be a function of wave height and angle of the incoming waves:

\[
Q_{\text{net}} = K \int_{T_1}^{T_2} H^3 \sin(2\alpha) \, dt \tag{6.1}
\]

Similarly, the gross transport yields:

\[
Q_{\text{gross}} = K \int_{T_1}^{T_2} |H^3\sin(2\alpha)| \, dt \tag{6.2}
\]

where \(Q\) = longshore sediment transport, \(T_1\) and \(T_2\) are start and end time of the storm, \(H\) = wave height and \(\alpha\) = wave angle relative to shore normal, \(K\) is a proportionality factor. This formula corresponds to the CERC formula for longshore sediment transport.

The wave height \(H\) and wave angle \(\alpha\) were taken from location 4 from the wave transformation study as presented in Ref /1/. In order to relate the transports during the storms to the average annual transport the derived transport rates were normalised with transport rates corresponding to the entire period where wave data was available. In this
way, the contributions to the annual net- and gross transport were calculated for each storm. The results are shown in Table 6.6.

Table 6.6 Derived relative littoral drift for the two selected storm periods

<table>
<thead>
<tr>
<th></th>
<th>Relative net littoral transport (-)</th>
<th>Relative gross littoral transport (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storm 1 (Nov/Dec 1985)</td>
<td>-0.10</td>
<td>0.12</td>
</tr>
<tr>
<td>Storm 2 (Feb/Mar 1989)</td>
<td>0.41</td>
<td>0.32</td>
</tr>
</tbody>
</table>

The derived weighting factors indicate that the net transport during storm 2 corresponds to 0.41 times the average annual net transport. Similarly, the gross transport during storm 2 corresponds to 0.32 times the average annual gross transport. The negative value for the net transport for storm 1 indicates that the net transport during this storm was directed towards west, whereas the average net annual transport is directed towards east.

The weighting factors for the two storms can now be derived from the following 2 equations:

\[ \text{Fac}_1 \cdot f_{\text{net},1} + \text{Fac}_2 \cdot f_{\text{net},2} = 1.0 \]  
\[ \text{Fac}_1 \cdot f_{\text{gross},1} + \text{Fac}_2 \cdot f_{\text{gross},2} = 1.0 \]

Where \( \text{Fac}_1 \) and \( \text{Fac}_2 \) are the weighting factors for storm 1 and storm 2 respectively and \( f_{\text{x},x} \) are the relative transport rates listed in Table 6.6.

The weighting factors for storm 1 and storm 2 were calculated as 1.07 and 2.70 respectively. Using these weighting factors, the annual transport rates can be estimated from the two storm periods, both with regard to net transport as well as gross transport.

By combining the derived weighting factors with the calculated accumulated transport rates listed in Table 6.5 the annual transport rates were calculated. The results are listed in Table 6.7.

Table 6.7 Integrated net and gross annual transports at Bakkafjara

<table>
<thead>
<tr>
<th></th>
<th>Central</th>
</tr>
</thead>
<tbody>
<tr>
<td>Annual net transport (m³/yr)</td>
<td>270,000</td>
</tr>
<tr>
<td>Annual gross transport (m³/yr)</td>
<td>950,000</td>
</tr>
</tbody>
</table>

The net integrated transports indicate that during Nov/Dec 1985 the morphology in the area around Bakkafjara suffered from net erosion. In the period Feb/Mar 1989 an accretion was simulated. However, local changes within the area (bounded by the three cross-sections) are seen with deposition to the west of the harbour and erosion to the
east. This indicates that the bar and the depression in the bar moved to the east. Further, the river discharge pushes sand from the river mouth into the active littoral zone. This supply is expected to balance the losses when a longer time span is considered.

It is noted that the weighting is chosen to reflect the average sediment transport pattern in the vicinity of the harbour. The weighting cannot be expected to be valid for a large area as the dominating storms vary along the coast. The weighting factors are used to estimate the gross transport passed the harbour and thereby the sedimentation in the harbour.

The uncertainty about sediment transport calculations is in general large and even in cases where all parameters are known sediment transport rates cannot be calculated more accurately than within a factor of 2. In the present case, calibration and validation have been undertaken by comparing natural morphological evolution in nature and in the model. In cases where man-made structures such as groynes are built the morphological response is very distinct and can be used for calibration and validation purposes. In such cases the certainty can be narrowed down. In the present case only the natural changes of a very dynamic system can give indications of the actual orders of magnitude of changes. The uncertainty of the results is therefore assessed to be in the order of a factor 2. On top of this uncertainty is a huge variability from year to year and the morphological evolution is determined by the storm conditions.
7 COASTAL IMPACT OF BAKKAFJARA PORT FACILITY

The coastline adjacent to the proposed Bakka fjara harbour will be affected by the harbour due to its blockage of the littoral drift on the inner part of the profile. The long-term response of the harbour on the coastline is investigated in this section to supplement the shorter periods of morphological calculations presented previously and to provide the following section dealing with sedimentation inside the harbour with input on the time scale of the coastline response.

The time required for the coastline to adapt fully to the new harbour from its original undisturbed state can be estimated roughly. To calculate the time it takes the accretion to reach the harbour entrance the following is assumed:

1 - Representative wave angle during storm from SW: 245°
2 - Present shoreline orientation 180°
3 - Extension of port from present shoreline: 600 m
4 - Active depth of the beach: 10 m
5 - Cross shore Extension of the port, W: 600 m

The total volume of sand required for the fillet is estimated as:

\[ Q = A \times D \]

where \( A \) = plan area \((= 0.5 \times W \times W / \tan[245°-180°] \approx 840000 \text{ m}^2) \) and \( D \) = active depth. Thus, the amount of sand necessary for the fillet is \( Q \approx 840,000 \text{ m}^3 \). The annual drift landward of the outer bar and towards east is in the order of 120,000 \text{ m}^3/year. Thus a rough estimate of the time for the fillet to be established is approximately 7 years.

This is, however, an underestimation of the time scale as the coastline orientation will change continuously (rather than being constant as assumed in the rough estimation). To get a better estimate of the time scale the coastline evolution is taken into account by using LITLINE which is part of LITPACK DHI’s software package for littoral drift and coastline kinetics. The LITLINE module simulates coastline evolution along a quasi-uniform coastline and calculates the coastline evolution by solving a continuity equation for the sediment in the littoral zone. The influence of structures, sources and sinks are included. LITLINE requires information on the sediment discharge from rivers, wave climates along the coastline stretch considered, the coastal profile and sediment properties. This information is all available.

The model is set-up to calculate the coastline evolution starting from January 1989. The model takes the sediment load from Markarfljót into account by using the correlation between the river discharge from Ref. /3/ and the sediment load calculated with MIKE 21 FM (see Figure 4.13 and Figure 4.14). If this correlation is used the average annual sediment discharge is 150,000 \text{ m}^3/yr and the LITLINE model will maintain the delta at the 1989 level.

In the following, LITLINE results are presented.
In Figure 7.1 the coastline after 2 years (i.e. in January 1991) is shown both for the situation with (in white) and without (in dark military green) the proposed harbour. The total discharge of sand during this period is the same in both situations and amounts to 150,000 m$^3$/yr. Therefore, differences in the 1991-coastlines are related entirely to the presence of the harbour. It is seen that the proposed harbour will influence the coast near the breakwaters where sand is seen to accumulate on both sides of the breakwaters. The 1991-coastlines (with and without the harbour) are identical elsewhere also near the river delta implying that the river delta is almost not influenced by the harbour.

![Figure 7.1](image)

In Figure 7.2 the coastline after 2 years (i.e. in January 1991) is presented both for a Markarfljót sediment discharge of 150,000 m$^3$/yr (average annual discharge) and a discharge of 300,000 m$^3$/yr (i.e. twice the average annual discharge). When the sediment discharge is doubled it is seen that the Markarfljót river delta expands and the additional sand being sourced into the coastal zone is seen to feed the coastline both east and west of the outlet. The delta expansion implies that the sediment transport capacity in the littoral zone in the vicinity of the Markarfljót river outlet is too small to remove the additional sediment feed by the river. Still, the additional sediment discharge is not seen to affect the coastline near the harbour during the modelling period.
The optimal position of the harbour is investigated in the following for the simulation period: January 1989 to January 1991.

Figure 7.3 shows the coastline after 2 years (January 1991) in a situation where the position of the harbour is located 500 m west of the proposed harbour and the discharge from Markarfljot is 150,000 m$^3$/yr. The accretion in this case is seen to be larger than the accretion associated with the proposed position of the harbour (see Figure 7.1).
Figure 7.3  Initial coastline and coastline after 2 years for harbour located west of the proposed harbour

Figure 7.4 shows the coastline after 2 years (January 1991) in a situation where the position of the harbour is moved 500 m to the east of the proposed harbour and the discharge from Markarfljot is 150,000 m³/yr. Areas of accretion are shown with yellow.

The accretion of sand along the breakwaters is seen to be smaller than for the harbour located west of the proposed harbour. This is due to the more pronounced sheltering effect of the Westmann Islands and the associated reduction in transport capacity. The accretion along the breakwaters of the harbour located east of the proposed harbour is, however, similar to the accretion found for the proposed harbour itself (see Figure 7.1). It can thus be concluded that the location of the proposed harbour is close to an optimum as a certain distance from the Markarfljot outlet is required. The mobility of the river outlet is great as demonstrated in Ref. /2/. 
In Figure 7.5 the coastline after 2, 5 and 10 years are presented. The simulations are carried out with the harbour located at the proposed location and with a fixed river mouth. The river sediment discharge is 150,000 m$^3$/yr. The coastline at the river mouth is only modified slightly during the 10-year simulation period whereas the coastline in the vicinity of the breakwaters changes significantly and such that the coastline reaches the toe of the breakwaters after 10 years.
In the above, the final shape of the coastline near the harbour has been investigated with LITLINE. 2D effects around the harbour are taken into account through parameterization of certain processes. The importance of 2D effects can, however, be evaluated more directly by adopting the final coastline calculated with LITLINE in the MIKE 21 bathymetry i.e the bathymetry used by the 2D morphological model. The 2D model has been executed with this modified bathymetry and the coastline is seen to be adjusted. The applied modelling period is combined by the two selected simulation periods (February 1989 and November 1985) using a speed-up factor of 10. The combined period resembled the annual period – see Figure 3.3. In Figure 7.6 the equilibrium coastline obtained with LITLINE and MIKE 21 FM is shown. It is seen that LITLINE predicts larger accumulation of sand near the breakwater. The acceleration of water along the breakwater, however, gives a smaller accretion here which entails a steepening of the profile. The process of profile steepening is not included in LITPACK.
Figure 7.6  Equilibrium coastline (after 10 years) at harbour with LITLINE (upper) and then modified in MIKE21FM (lower)
8 SEDIMENTATION INSIDE THE HARBOUR

In this section an assessment of the anticipated annual variability (or more technically: standard deviation) of the sedimentation and the mean annual sedimentation of the harbour is carried out using the modelled sediment transport fields for the two periods selected and the coastline evolution presented in the previous section. A distinction is made between accumulation of sand and accumulation of fines.

8.1 Accumulation of Sand

Figure 8.1 shows a snapshot of the sediment transport field around the entrance of the proposed Bakkafjara breakwaters just after its construction.

![Figure 8.1 Calculated snap-shot sediment transport field around Bakkafjara breakwaters at the peak of the February 1989 storm](image)

Sediment is transported into the harbour basin through the effect of vortices. These vortices create a constant exchange of water (and sediment) between the sea and the harbour basin. Detailed modelling of these vortices and the associated sediment flux through the harbour entrance would require mathematical models that operate on very small spatial- and temporal scales, and is beyond the scope of the present work. Instead, empirical relations are used to estimate the annual sedimentation into the harbour basin. As a rough estimate, the following rule of thumb for exchange of suspended sediment into a harbour basin can be applied (see Ref. /5/):

\[
Q = 0.07 \times D \times V \times C \times W
\]

where \(Q\) is the annual sedimentation (\(\text{m}^3/\text{year}\)), \(D\) = water depth, \(V\) = current speed, \(C\) = mean concentration and \(W\) = width of the harbour entrance. The formula accounts for exchange of sediment due to the turbulence in the harbour entrance caused by the velo-
ity gradient. The velocity gradient is large as the velocity changes from the accelerated velocity just outside the harbour entrance to a value close to zero just inside the harbour.

To estimate annual sedimentation rates and to account for the impact of waves the formula has been interpreted in the following way:

\[ Q = 0.07 \times W_0.07 \times Q_l \] (8.1)

where \( Q_l \) = longshore sediment transport (m\(^3\)/m/year) passed the entrance.

In Ref. /1/ the sedimentation of the harbour was estimated to be in the order of 20,000 m\(^3\)/year for a harbour with a 70 m wide entrance. This sedimentation rate was obtained with LITPACK by using the maximum gross transport rate on the inner part of the profile on the open coast. The sedimentation rate obtained by using the maximum gross sediment transport rate (in the inner part of the profile) corresponds to a value valid in the case where the morphology and coastline have attained its equilibrium profile around the breakwaters. The sedimentation rate is a central estimate based on 25 years of wave data. The variation in annual sedimentation (from year to year) will be evaluated as well.

The sedimentation rate valid just after the construction of the breakwaters can be estimated as well. According to LITDRIFT the initial sedimentation rates (from average wave conditions) can be estimated to be 1000-1500 m\(^3\)/year. This value is obtained by considering \( Q_l \) at a water depth of 5.5 m (from Figure 3.13 in Ref. /1/) which equals the smallest allowable water depth in the entrance of the harbour. This value can be compared to the sedimentation rates derived in the MIKE 21 FM model. The results from MIKE 21 FM are different from LITPACK results in the sense that they include 2D effects. Thus, a better estimate of the sedimentation value is possible as both the effect of flow contraction and the effect of the 2D bathymetry on the wave conditions in front of the entrance are included.

Figure 8.2 presents the annual gross sediment transport profile in front of the harbour entrance estimated from the MIKE 21 FM results. The annual transport rates are obtained by utilizing the weights of modelled periods worked out previously – see section 6.6. The gross annual transport profile, \( q_{\text{gross,annual}} \), is estimated as:

\[ q_{\text{gross,annual}} = 1.07 \times q_{\text{gross, November 1985}} + 2.70 \times q_{\text{gross, February 1989}} \]

Based on the results presented in Figure 8.2, the sedimentation rate is found to be approximately 3000 m\(^3\)/year for the initial stage with a 90 m wide entrance and using the maximum gross sediment transport rate in the profile just outside the entrance. This value is a little larger than that obtained with LITPACK (1000-1500 m\(^3\)/year) which can be attributed mainly to the contraction of streamlines around the breakwaters; a condition which is disregarded in LITPACK. The 2D effects are thus responsible for an increase in sedimentation.
The gross sediment transport passed the harbour in the equilibrium case where the depth will be approximately 5-6 m has been roughly assessed based on the assumption that the gross transport close to the mouth of the harbour in the equilibrium situation is of the same order of magnitude as the present gross transport on the outer bar. This is assessed to be a conservative assumption. The sedimentation in the equilibrium case is thereby approximately 15,000 m$^3$/yr.

The estimates of sedimentation shall be ascribed a safety factor of 2 due to the large uncertainty of the sediment transport calculations.

The estimated sedimentation rates are presented in Table 8.1.

The infill of sediment into the harbour from the 3D helical motion and sediment transports induced by second order wave phenomena (wave asymmetry, streaming, wave drift) is not accounted for in the above estimate; however, the impact of these can be assessed by considering the period-averaged transport field in Figure 6.13 showing transports normal to the depth-averaged flow direction. The order of magnitude of the transports in Figure 6.13 is seen to be approximately 2-3 m$^3$/m/year in the mouth of the breakwaters. The transports induced by these mechanisms thus contribute with approximately 200-300 m$^3$/year. The contribution is two orders of magnitude smaller than that caused by the turbulence of the main flow and can therefore be disregarded.
### Table 8.1 Estimated sedimentation rates for three entrance widths

<table>
<thead>
<tr>
<th></th>
<th>MIKE 21 FM (W=70m) Weighted</th>
<th>MIKE 21 FM (W=90m) Weighted</th>
<th>MIKE 21 FM (W=110m) Weighted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial rates after construction [m³/year]</td>
<td>4,400</td>
<td>5,600</td>
<td>7,000</td>
</tr>
<tr>
<td>Equilibrium rates after construction [m³/year]</td>
<td>25,000</td>
<td>30,000</td>
<td>39,000</td>
</tr>
</tbody>
</table>

#### 8.2 Accumulation of Fine Suspended Sediments

Fine suspended sediment is released by the river during periods with high discharges. Aerial photographs of the plumes in front of the river mouth suggest that a large part of the fine sediment is transported far into open sea. Here it settles in deep water where it cannot easily be re-suspended by the action of waves. During events with waves from easterly directions sediment-laden water will pass the harbour entrance. A part of the sediment will enter the harbour basin due to the daily exchange of water generated by the tide. At present no information about suspended sediment concentrations are available. Therefore, the sediment volumes presented here must be regarded as indicative.

The volume of water that enters the harbour each day (Tidal prism) is approximately 600m x 800m x 2m = 960000 m³. This means that each month a volume of approximately 960000m³ x 30 x 2 = 58 mill m³ enters the harbour basin.

Assuming a significant concentration for 2 months per year and assuming that this concentration is 100 mg/l the sedimentation inside the harbour is 4400 m³ per year. The sediment will accumulate evenly across the entire harbour basin and will lead to an annual sedimentation of about 1 cm.

More accurate estimates of sedimentation of fines can be made if measurements of fine suspended matter at the location are carried out.
9 EQUILIBRIUM WATER DEPTH IN FRONT OF HARBOUR ENTRANCE

In this section the water depths in front of the harbour entrance are considered.

A critical water depth of 5.5 m has been defined for the entrance area of the proposed Bakkafjara harbour. A requirement on entrance area water depths is defined to ensure that the ferry (connection Westmann Islands and the main land) will be able to navigate safely through the entrance. In the following the natural water depths in front of the entrance will be investigated using the 2D morphological model complex and an optimization of the configuration will be made to minimize sedimentation.

The natural or equilibrium water depths in front of the entrance are determined by the sediment transport capacity here. If the capacity is relatively large then sediment will be able to bypass at relatively large depths, whereas low transport capacities will result in relatively shallow depths.

The sediment transport capacity is determined by the level of turbulence near the seabed and in the water column. The turbulence is generated by the current and waves and acts as a stirring mechanism for keeping sediment in suspension. The transportation of the stirred sediment is caused by the current. In the present case the current plays an important role in the morphodynamics as the breakwater will concentrate the flow. The contraction of the flow affects not only the sediment transport capacity but also how fast the sand is being removed. The interplay between the sediment transport capacity and the morphology is a highly non-linear process and the equilibrium morphology is a balance between complex processes. To determine such equilibrium the MIKE 21 FM model is applied using stationary offshore wave conditions to the point in time where equilibrium water depths are attained.

Most of the estimates of equilibrium depths are made before the final fine tuning of the morphological modelling complex. The main effect of the fine tuning was a speed up of all processes. The development towards equilibrium has therefore been checked for the most critical case with the final model set-up. These results are presented in Section 9.5.

9.1 Stationary Wave Conditions

To this point, the morphological model has been used to run historical periods. These periods include waves ranging from extreme to moderate. Integrating historical events with its shifting wave directions and alternating wave heights are important for the understanding of the dynamics of the morphology during storms.

The calculated changes in water depths in front of the harbour for the 1989 modelling period (with time factor 25) is shown in Figure 9.1. The location of the extraction points are shown in Figure 9.1 as well.
The water depth in front of the harbour (point 2) changes from 9 m to 6.5 m during the period, with significant changes taking place during the peak of the storm. The adjustment of the water depth would, however, have continued if the duration of the storm was prolonged. In other words the duration of the storm is a governing factor for estimating the equilibrium depth. To properly determine the potential water depths in front of the harbour the morphological model is executed by using conditions that are stationary in terms of the wave forcing. Modelling conditions where the wave forcing is stationary is attractive from another point of view as the use of speed factors in the morphological model is valid. The use of a speed-up factor can enhance the simulation period by an order of magnitude and thus allow the model to predict longer-term changes in the morphology.

The starting point of the morphological simulations is the bathymetry measured May 2006. The model is executed to the point in time where the bar building up along the up-drift side of the breakwaters is fully developed and where water depths in the area in front of the entrance have attained the value of equilibrium. To reach this stage a speed-up factor of 10 on the morphological development has been applied. As a result of the enhancement in simulation period, the potential displacement of the bar depression is obtained as well.
The equilibrium water depth depends on the wave condition. Therefore, the equilibrium water depth is investigated by considering a range of different events which are characteristic of the wave climate. The following conditions have been selected from the modelling periods:

- Extreme south-westerly storm condition
- Moderate-to-rough south-westerly storm conditions
- Rough south-easterly storm condition
- Rough south-westerly storm conditions

The initial wave fields for the different conditions are shown in Figure 9.4 to Figure 9.5 (applied bathymetry is May 2006).
Further, the wave cases presented in Figure 9.4 to Figure 9.5 are listed in Table 9.1 in terms of the significant wave heights at the Bakkafjara wave buoy and at the offshore station located at (63°N, 21°W).
Table 9.1 Wave heights at wave buoy for wave conditions shown in Figure 9.4 to Figure 9.5.

<table>
<thead>
<tr>
<th>Characterization of wave condition</th>
<th>Wave heights at wave buoy at (63°N, 21°W) [m]</th>
<th>Wave heights at (63°N, 21°W) [m]</th>
<th>Wave direction at (63°N, 21°W) [°N]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extreme south-west</td>
<td>5.0</td>
<td>11.5</td>
<td>251</td>
</tr>
<tr>
<td>Moderate-to-rough south-west</td>
<td>2.7</td>
<td>6.2</td>
<td>237</td>
</tr>
<tr>
<td>Rough south-east</td>
<td>6.9</td>
<td>8.8</td>
<td>145</td>
</tr>
<tr>
<td>Rough south-west</td>
<td>3.7</td>
<td>8.9</td>
<td>244</td>
</tr>
</tbody>
</table>

The exceedence probability for the offshore wave height (63°N, 21°W) has been derived on the basis of the available data (25 years). The exceedence probability curves are presented in Figure 9.6. From the figure the exceedence probability can be worked out for the selected wave events. The extreme condition is exceeded 6 hours/year whereas the mild-to-rough conditions are exceeded 12 days/year. Approximately 70% of the time significant wave heights fall below 3.0m.

In addition to these 4 different stationary events, the rough south-westerly storm condition has been used to investigate the positive impacts of capital dredging on the water depths. The water depths in front of the entrance are simulated using a bathymetry dredged to 7 metres inside the harbour and in a confined area outside the entrance. The use of capital dredging could, however, pose a breakwater stability problem. If this is the case then capital dredging is obviously not a viable option in which case maintenance dredging in the entrance area will be required in the initial stage after the construction of the harbour.
In addition to the investigations described above an optimisation of the configuration of the outer part of the breakwaters has been investigated. The optimal configuration aims at minimizing sedimentation. The configuration is optimised by changing the angle between the orientation of the outer part of the breakwaters.

9.2 **Morphological Evolution during Rough South-Westerly Wave Conditions**

In Figure 9.7 the initial and final stage of the morphological evolution for the rough south-westerly conditions are presented. The presence of the harbour is seen to have an immediate impact on the morphology in the vicinity of the harbour. The most prominent development in the morphology is found on the up-drift side of the harbour along the breakwater. Here a bar is building up to accommodate the bypass of sediment. The bar is seen to migrate towards the entrance, however only to a certain point. The build-up of the bar comes to an end as the equilibrium between the sediment transport capacity and the water depth is attained. This happens when the sediment transport capacity becomes constant along the streamlines. As the flow is concentrated in the area in front of the entrance the depth must increase until the sediment transport capacity becomes constant.

In the final stages of the morphological simulation the bathymetry starts to resemble the bathymetry measured on the 7th of May 2005 with the outer bar being attached to the near-shore morphology east of the harbour. It is worth mentioning that prior to May 2005 significant storms from south-west were in fact observed.
The changes in water depths in front of the harbour for the rough south-westerly condition are shown in Figure 9.8. The simulations show that the water depth in front of the harbour (black line) exceeds 6 m at the end of the simulation. The water depth of the outer bar near Bakkafjara does not decrease during the simulation (for the stationary wave forcing).

The water depths in front of the entrance are calculated for other characteristic south-westerly conditions as well. Upper part of Figure 9.9 presents water depths in front of the harbour for extreme conditions whereas lower part of Figure 9.9 presents water depths for mild-to-rough conditions. Undulations in the water depth as function of time are pronounced for these cases and are due to the tidal flow.

The water depth for the extreme condition reaches equilibrium quickly and at the end of the simulation water depths are approximately 7 m in front of the entrance. In the mild-to-rough case water depths in front of the entrance becomes less than 5 m. The mild-to-
rough wave conditions thus define the most critical waves in terms of natural water depths in front of the entrance as the equilibrium value is found to be smaller than the accepted value of 5.5m.

Figure 9.9 Water depth at extraction points (see Figure 9.1) for extreme (upper) and mild-to-rough (lower) south-westerly conditions (Hs=5.0 and 2.7m at Bakkafjara wave buoy)

The sudden decrease in water depths is a typical pattern in the plots showing water depths at extraction points as function of time. The sudden decrease is due to the bypass bar reaching the extraction points. Therefore, the decrease is monitored first at the up-drift extraction point, then at the entrance extraction point and finally at the down-drift extraction point.

For the extreme south-westerly wave conditions, the bar is seen to reach the extraction point west of the harbour after only 14 days – see Figure 9.9. The bar reaches the entrance extraction point one (1) month later. The development of the bypass bar is slower for the mild-to-rough south-westerly conditions. Under these conditions the bar shows up at the extraction point west of the harbour 3 months into the simulation and at the extraction point located in front of the entrance the bar shows up after 6 months. For the mild-to-rough conditions it takes nearly one year for the bar to reach the extraction point east of the harbour.
9.3 Morphological Evolution during Rough South-Easterly Wave Conditions

In Figure 9.10 the initial and final stage of the morphological evolution for the rough south-easterly wave condition is presented.

For the rough south-easterly conditions the littoral sediment is transported to the west and near the harbour the sediment is seen to bypass along two main sediment routes; one close to the eastern breakwater and one along the existing outer bar. Along the eastern breakwater sediment is seen to form a bar in a way similar to the bypass bar formed during rough south-westerly conditions (i.e. along the western breakwater). The bar
formed along the eastern breakwater does not develop, however, to the same degree as that seen for the rough south-westerly conditions. Also, sediment is seen to bypass along a second route as well being the existing outer bar. The existing outer bar develops into a bypass route as it attaches to the near-shore morphology just east of the harbour. The existing outer bar develops faster than the bar building up along the eastern breakwater. The existing outer bar is seen to migrate quickly towards the navigation line. In conclusion, the bypass of sediment mainly takes place along two routes during south-easterly wave impact; the favoured route being the existing outer bar. Again, the build up of the bar comes to an end as an equilibrium between the sediment transport capacity and the water depth is attained for the rough south-easterly conditions. In the final stages of the morphological simulation the bathymetry has striking resemblance to the bathymetry measured on July 2004. Note that prior to the July 2004 sounding a very significant storm from south-east was observed.

The calculated changes in water depths in front of the harbour for the rough south-easterly condition are shown in Figure 9.11. The water depth in front of the harbour (black line) is more than 8 m at the end of the calculation which is well below the critical navigation depth requirement of 5.5 m. For the rough south-easterly waves, the sudden decrease in water depths is seen to take place from the beginning of the simulation at the extraction point east of the harbour. The bar reaches the entrance extraction point one (1) month later and three (3) months later it reaches the extraction point west of the harbour.

The larger water depths found for south-easterly wave conditions compared to south-westerly wave conditions are due to the way sand is bypassing the harbour. As explained above, south-easterly waves will move sand to the west along two main sediment routes; one is along the existing outer bar and one is along the eastern breakwater, the latter route being less dynamic. As a consequence, the bar formation takes place more seaward for south-easterly waves and water depths are consequently larger.

![Figure 9.11 Water depth at extraction points (see Figure 9.1) for rough south-easterly condition. (Hs=6.9m at Bakkafjara wave buoy)](image)

The water depth along the outer bar thus decreases east of the harbour. The minimum water depth at the navigation line and outer bar intersection, however, is not seen to change significantly compared to the water depth from the initial bathymetry. West of
the navigation line the outer bar is seen to deteriorate and the water depth increases with approximately 1 m.

9.4 Equilibrium Depth using Capital Dredging at Entrance

From the simulations of the morphological evolution presented above it was found that the bar developing on the up-drift side of the breakwater contains less sand than what is available in the existing bathymetry. The excess amount of sand in the existing bathymetry bypasses the entrance in the initial stage just after the construction of the harbour. Once this excess amount of sand has bypassed or settled within the harbour entrance the morphology starts to adjust continuously towards equilibrium. During the passage of the excess sand water depths just outside the entrance area are seen to be close to or even smaller than the critical water depth of 5.5 m. Furthermore, sand is seen to be captured in the entrance itself. As a consequence, water depths between the breakwaters remain below the critical threshold water depth of 5.5 m as the captured sand will not escape the sheltered entrance. Outside the entrance the morphology will reach the same equilibrium as in the case of no capital dredging.

To reduce the amount of sand which is captured in the entrance a test is made where the bathymetry is dredged down to -7 m along the breakwaters and in the entrance area. This capital dredging situation is modelled for rough south-westerly conditions. In Figure 9.12 the initial and final stage (No.1 and 5) of bathymetrical evolution is shown along with intermediate stages (No. 2 to 4) of the morphological evolution.

Apart from the morphological changes taking place near the entrance of the harbour, the evolutional stages presented in Figure 9.12 shows that the water depth at the outer bar does not change significantly with time. In fact, the water depths at the outer bar are seen to increase slightly at the position of the navigation line and further downdrift (to the east).
Figure 9.12 Close-up of initial bathymetry (1) with capital dredging and intermediate stages of the bathymetrical evolution (2 to 5). $H_s=3.7m$ at Bakkafjara wave buoy
The changes in water depths at the extraction points defined in Figure 9.1 for the rough south-westerly condition are presented in Figure 9.13. The water depth in front of the harbour (black line) is nearly 7 m at the end of the simulation and therefore well above the navigational depth requirement of 5.5m.

![Figure 9.13 Water depth at extraction points (see Figure 9.1) for rough south-westerly condition and capital dredging (Hs=3.7m at Bakkafjara wave buoy)](image)

**9.5 Check of Equilibrium Depth**

Following the fine tuning of the transport field during the calibration/validation phase, a check of the equilibrium depth was carried out. It should be noted that differences in equilibrium depths are not expected; however, the speed at which the equilibrium is achieved may vary with the fine tuned model. The present check is based on the forcing condition that yields the worst case scenario with regard to minimum water depth at the harbour entrance i.e. the morphological evolution for mild-to-rough south-westerly conditions (see above section).

In Figure 9.14 the morphological evolution for the mild-to-rough south-westerly condition is presented. As expected, the calibration/validation efforts do no result in differences in the equilibrium morphology: depths in front of the entrance are still observed to be around 5 m.
9.6 **Equilibrium Depth for Different Breakwater Configurations**

Sediment transport conditions in front of the entrance area are crucial for bypass and sedimentation. The transport of sediment around the harbour is determined by the stirring capacity of the flow and the ability of the current to transport the stirred material. Both factors can be controlled by the shape of the breakwaters. The shape of the breakwaters should be streamlined such that the current is forced along the upstream breakwater smoothly to concentrate the current in front of the entrance area. Depending on the environment, however, the streamlined shape may alter.

One of the important parameters in the configuration of the breakwater is the shape of the outer part of the breakwater. Therefore, to meet the required navigation depth of 5.5 m the configuration of the outer part of the breakwaters has been investigated by altering the orientation of the outer part of the breakwaters. The orientation is measured as the angle between the orientation of the eastern and western breakwater heads. In Figure 9.15 the definition of the angle between the breakwaters is sketched.

So far, the applied orientation has been 40° which is commonly accepted as an optimal angle with respect to minimizing sedimentation. However, the optimal angle may change under different conditions and environments. In the following, an angle of 65° is investigated.
In Figure 9.16 the final simulated morphology in the vicinity of the harbour is shown for breakwater orientations of 40° and 65° for the mild-to-rough south-westerly conditions.

The mild-to-rough south-westerly conditions were the critical conditions in terms of fulfilling the required navigation depth at entrance. The final water depths in the case where the breakwaters have a 65° angle between them are seen to be larger than with the 40°-case. The reason for this is that the more pointed harbour configuration entail bypass of sediment in front of the harbour along a more narrow route. The more pointed
configuration also increases slightly the depth of the Bakkafjara pit and the area of large
depths within the pit are seen to move northward and therefore closer to the structure.
The modifications to the configuration of the breakwaters show that an alteration of the
angle has a positive impact on the water depths in front of the entrance.

The change in water depth in front of the harbour is shown in Figure 9.17. The initial
water depths are larger in front of the entrance compared to the initial depths found with
the 40° breakwaters as the breakwaters with the 65° angle extend a bit further seaward.
At the end of the simulation, the water depth in front of the harbour (black line) is more
than 7 m and close to an equilibrium value. The water depths are thus larger than the
equilibrium water depths obtained with the 40° breakwaters. The increase in angle thus
further improves the water depths in front of the entrance area.

![Figure 9.17 Water depth at extraction points (see Figure 9.1) for rough south-westerly condition and an angle between the breakwaters of 65°. (Hs=3.7m at Bakkafjara wave buoy)](image)

In Figure 9.18 the water depth at extraction points are shown for the rough south-easterly conditions using an orientation of the breakwaters of 65°. The water depths are
quite similar to the water depths found for an angle of 40°. The similar results are due to
the fact that the sand is bypassing along the existing outer bar rather than building a
strong secondary bar along the eastern breakwater. Therefore, the details of the break-
water configuration are of less importance under south-easterly conditions.
Figure 9.18 Water depth at extraction points (see Figure 9.1) for rough south-easterly condition and an angle between the breakwaters of 65°. (Hs=6.9m at Bakkafjara wave buoy)

Finally, in Figure 9.19 the water depth at extraction points are shown for the mild-to-rough south-westerly conditions using an orientation of the breakwaters of 65° including capital dredging. The water depth in the entrance area is close to 5.5 m and only just fulfils the required minimum navigation depth set to 5.5 m for the entrance area. The time required to obtain equilibrium depths in front of the entrance for the mild-to-rough wave conditions is nearly twice that found for the rough wave conditions.

Figure 9.19 Water depth at extraction points (see Figure 9.1) for mild-to-rough south-westerly condition and an angle between the breakwaters of 65° and with capital dredging. (Hs=2.7m at Bakkafjara wave buoy)

The morphological simulations show that the mild-to-rough wave conditions are the critical in terms of fulfilling the navigational depth requirement. Under these conditions, however, the time-scale for building up the bypass bar is longer compared to the rough and extreme conditions but as the exceedence probability curve for offshore wave heights (63°N, 21°W) show (see Figure 9.6) the mild-to-rough wave heights are more frequent. The mild-to-rough waves are seen to be exceed 12 days/year as opposed to the rough wave climate which is exceeded approximately 2 days/year. As a first approximation, the bar can be estimated to develop 6 times slower during mild-to-rough conditions.
compared to the development during rough wave conditions and still be equally important. The morphological simulations presented above show that the bar develops approximately 2-3 times slower under mild-to-rough conditions, which justifies the importance of considering the mild-to-rough wave case as the critical condition for navigational depths at the entrance.

9.7 Final Comments on the Equilibrium Water Depths

The water depth in front of the entrance has been assessed using the 2D morphological model. The depths have been investigated for different stationary wave conditions and the impact of capital dredging and the role of the breakwater configuration in terms of the angle between the breakwaters have been outlined.

It has been shown that the use of capital dredging gives less sedimentation problems inside the harbour and between the breakwaters. The capital dredging may, however, pose a breakwater stability problem. If capital dredging poses a stability problem and is discarded, then additional maintenance dredging in the entrance area itself should be expected in the initial stage after the construction of the harbour (additional to that estimated in Section 7).

Furthermore, an optimal streamlined harbour configuration for the Bakkafjara environment was determined. An optimal configuration was obtained when the angle between the breakwaters was approximately 65°. This more pointed breakwater configuration was seen to perform better than 40° in term of navigation depth in front of the harbour. With an angle of 65° the harbour was found to perform well and to satisfy the minimum navigation depth set to 5.5 m outside the entrance area.

The simulations of the morphology furthermore show that the pit-type feature and the depth over the outer bar at the navigation line are slightly modified and such that an increase in water depths compared to the initial bathymetry of May 2006 is found for all of the stationary wave events considered. The pit-type feature is seen to migrate insignificantly and even deepen.
10 **SUMMARY**

10.1 **Numerical Model**

A coupled and fully integrated 2D numerical model for waves, currents and sediment transport (MIKE 21 FM) has been established to simulate impacts of a proposed harbour on the morphological evolution of the surveyed May 2006 bathymetry both for stationary forcing conditions and for historical periods. The later includes the two major storm periods: November 1985 and February 1989.

A validation of the wave model has been performed successfully. The wave model is seen to give excellent results capturing the significant wave height reduction from offshore (south of Westmann Islands) to the wave buoy as well as the wave height attenuation in the surf-zone correctly. For the present investigation it is concluded that the near-shore wave climates are reproduced with sufficiently accuracy and further near-shore wave data (including directions) are presently not crucial for the studies.

In addition, the sediment transport model and the morphological model have been calibrated against the observed development from 2004 to 2005 and validated against the observed evolution from 2006 to beginning of 2007.

10.2 **Configuration of Harbour**

The proposed harbour is composed by two breakwaters. The proposed entrance width is 90 m and the initial water depth (May 2006 Bathymetry) here is 8 m. The location of the harbour has been investigated both with LITLINE and a very detailed morphological model and the proposed position is found to be optimal. The water depths in the entrance area were investigated using the MIKE 21 FM and stationary forcing conditions to obtain the potential depths for various storm scenarios. The morphological calculations show that if an angle between the breakwaters of 65° is used then a water depth of 5.5 m in front of the harbour entrance can be maintained. Thus, the alignment of the outer part of the eastern and western breakwater is such that the angle between them should be 65°.

10.3 **Sedimentation of Harbour**

The determination of the equilibrium depths in front of the entrance with MIKE 21 FM revealed that the use of capital dredging would decrease the sedimentation problems in the entrance area. This is because more sand is present in the existing profile than in the equilibrium profile found after the construction of the harbour. Obviously, a fraction of this excess amount of sand is seen to settle between and within the breakwaters. The use of capital dredging would mitigate this initial sedimentation problem; however, capital dredging in the proposed form could pose a breakwater stability problem. If there is a stability problem then capital dredging is not an option. In such case maintenance
dredging in the entrance area is required in the initial stage after the construction of the harbour.

The infill of a fraction of the excess amount of sand in the bathymetry is an initial sedimentation problem; however, a more permanent sedimentation problem is that associated with the bypass of littoral drift (i.e. sand updrift of the harbour area). Harbour sedimentation rates associated with the bypass of sand have been calculated as well. The rates are presented in Table 10.1 for both initial conditions and with the morphology in full equilibrium with the impacts imposed by the harbour. A central estimate of the time scale for the coastline to attain its equilibrium was estimated to be 10 years using LITLINE; however, based on the variability in gross transport the equilibrium stage may be obtained after 5 to 20 years. Sedimentation rates are given for entrance widths of 70 m, 90 m and 110 m. For an entrance width of 90 m which is close to the proposed width the mean annual sedimentation rate in the first year after construction is 5,600 m³. After approximately 10 years (and beyond) the mean sedimentation rate is increased to 32,000 m³. These estimates include a factor of 2 to accommodate the uncertainty on the sediment transport calculations. The annual sedimentation rate will, however, change significantly due the variability in the wave climate. During calmer years the sedimentation rate will be approximately half of the average rate whereas during more rough years the sedimentation rates will be approximately double of the average.

<table>
<thead>
<tr>
<th>Width: (W=70m)</th>
<th>(W=90m)</th>
<th>(W=110m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial rates after construction [m³/year]</td>
<td>4,400</td>
<td>5,600</td>
</tr>
<tr>
<td>Equilibrium rates after construction [m³/year]</td>
<td>25,000</td>
<td>30,000</td>
</tr>
</tbody>
</table>

The sedimentation is not evenly distributed within the harbour. The coarser fractions of the trapped sediment will mainly settle in the area just inside or between the breakwaters. Finer fractions of suspended material will settle along the inner side of the breakwaters i.e. further inside the harbour.

10.4 Outer Bar Morphology

Morphological changes of the outer bar during the two periods (more than 70 days simulated) were modelled with the May 2006 surveyed bathymetry as starting point and the simulations revealed the following:

- During storms with waves from the south-east a general erosion of the May 2006 outer bar level was seen near Bakkafjara
- During periods of south-westerly waves bed features were displaced eastward; however, under these conditions a seaward deflection of the transport on the outer bar
was found near Bakkafjara. A general lowering of the May 2006 bar level was observed

- During periods of south-south-westerly waves the deflection of transport at the depression was seen to be more pronounced

Therefore, if the present bathymetry is exposed to conditions comparable with those of November 1985 and February 1989 the bar depression will not only be maintained but increased slightly. This points to the conclusion that the depression in the outer bar is maintained during storm periods comparable with those applied. This is a conclusion which is supported by the latest survey (January 2007) showing a significant lowering of the bar depression under forcing conditions (i.e. a wave climate) which is quite similar to the forcing conditions applied in the numerical model.

To further understand the dynamics of the outer bar the following morphological modifications in the outer bar were made to the May 2006 bathymetry:

- Placing a pile of sand on the bar in front of the breakwaters
- Excavating sand from the bar in front of the breakwaters

The modifications to the outer bar were made such that the water depth over the bar was reduced to 2 m over approximately 300 m in the alongshore direction and to 7 m over a distance of 150 m. In the former case, the storm was capable of eroding the pile of sand and the original level of the bar was nearly re-established. In the case of the excavated bar water depths in the central part of the excavation were increased by 10-20 cm over the period.

These findings fully support findings of previous simulations i.e. that the level of the bar in front of the deep pit-type trough is reduced under storm conditions. All simulations have demonstrated how two dominating morphodynamical mechanisms are at play during storm conditions. The resulting level of the bar is a continuous battle between the rip current induced scouring and the deposition of sand caused by the longshore current. In contrast to the shifting position of a rip channel on an open coast the position of the rip at Bakkafjara is locked to the zone in which the Westmann Islands provides sheltering. The Bakkafjara rip is thus more or less fixed and the existence of the bar depression is a robust morphological feature.

Having obtained an understanding of the importance of the rip current the extensive wave and bathymetrical data can be analysed with this in mind. The combination of great wave exposure west of Bakkafjara (which is a necessity for rip currents to be generated) and relatively low rates of longshore transport at Bakkafjara are regarded as a prerequisite for the bar to deepen to the level seen in the May 2002 survey. By studying Table 2a through Table 3b in Ref. /1/ it is realised that this particular combination is not unusual. In fact, such combination can be identified in the tables in the years 1983, 1992, 1993 and 2001. On average the level seen in 2002 therefore has a probable return period of 8-10 years which is supported by the running average of the wave-energy from the offshore direction. The present level of the bar depression is more likely to be the level characteristic for periods where the deposition caused by the longshore transport is counter-balanced by rip induced scouring. This level is not an exceptional level.
The level of the bar depression was measured back in 1973 (the only survey of the bar conducted before 2002) to a similar level (5.8 m).

It is concluded that the bar formation at Bakkafjara is in a dynamic equilibrium with the forcing which has been present for the past decades. It may be critical for the development if significant changes in the dominant wave direction or the discharge of sediment due to global climatic changes will take place. Such uncertainties have not been considered.
11 REFERENCES


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/3/  Markarfljót, calculated discharge with WaSiM-ETH watershed model. By ORKUSTOFNUN for Sinlinggastofnun, June 2006.


APPENDIX A

Phase 1 report: Bakkafjara, Sediment Transport and Morphology
APPENDIX B

Comparison between Wave Heights along Navigation Line obtained in MIKE 21 SW and Physical Test Model
Wave-direction: SOUTH

Figure B1 Variation over bathymetry with 5.5m water depth (CD) at top of bar. Full drawn lines are numerical model results.

Figure B2 Variation over bathymetry with 6m water depth (CD) at top of bar. Full drawn lines are numerical model results.
Wave-direction: SOUTH-WEST

Figure B3  Variation over bathymetry with 5.5m water depth (CD) at top of bar. Full drawn lines are numerical model results

Figure B4  Variation over bathymetry with 6m water depth (CD) at top of bar. Full drawn lines are numerical model results
Wave-direction: SOUTH-EAST

Figure B5  Variation over bathymetry with 5.5m water depth (CD) at top of bar. Full drawn lines are numerical model results

Figure B6  Variation over bathymetry with 6m water depth (CD) at top of bar. Full drawn lines are numerical model results
APPENDIX C

Sea State Information Charts for Critical Waves
Wave-direction: SOUTH

Figure C.1 Sea State Information Chart for waves from south for: Upper: water level (wl) =2.3m CD, middle: wl = 1.4m CD, Lower: wl = 0.5m CD
Wave-direction: SOUTH-WEST

Figure C.2  Sea State Information Chart for waves from south-west for: Upper: water level (wl) =2.3m CD, middle: wl =1.4m CD, Lower: wl = 0.5m CD
Wave-direction: SOUTH-WEST cont’d

*Figure C.3*  Sea State Information Chart for waves from south-west for water level = 0.0m CD
Wave-direction: SOUTH-EAST

Figure C.4 Sea State Information Chart for waves from south-east for: Upper: water level (wl) =2.3m CD, middle: wl =1.4m CD, Lower: wl = 0.5m CD
Wave-direction: SOUTH-EAST cont’d

Figure C.5  Sea State Information Chart for waves from south-east for water level = 0.0m CD
APPENDIX D

Sea State Information Charts for 98% Waves
Wave-direction: SOUTH

Figure D.3  Sea State Information Chart for waves from south for: Upper: water level (wl) = 2.3m CD, middle: wl = 1.4m CD, Lower: wl = 0.5m CD
Figure D.4  Sea State Information Chart for waves from south-west for: Upper: water level (wl) =2.3m CD, middle: wl =1.4m CD, Lower: wl = 0.5m CD
Wave-direction: SOUTH-WEST cont’d

Figure D.3  Sea State Information Chart for waves from south-west for water level = 0.0m CD
Wave-direction: SOUTH-EAST

Figure D.4  Sea State Information Chart for waves from south-east for: Upper: water level (wl) =2.3m CD, middle: wl =1.4m CD, Lower: wl = 0.5m CD
Wave-direction: SOUTH-EAST cont’d

Figure D.5  Sea State Information Chart for waves from south-east for water level = 0.0m CD