MIKE 21 BW

Boussinesq Waves Module

User Guide

PLEASE NOTE

1 About This Guide

1.1 Purpose

The main purpose of this User Guide is to enable you to use the Boussinesq Wave modules included in MIKE 21 BW, for determination and assessment of wave dynamics in ports, harbours and in coastal areas. The User Guide is complemented by the Online Help.

Chapter 2 Introduction *(p. 9)*, gives you a general description of the modules included in MIKE 21 BW and type of typical applications.

Chapter 3 Getting Started *(p. 17)*, contains a step-by-step procedure which can be followed when working on an application or when writing a proposal. It is the intention that by following this procedure and by using the Reference Manual you should be able to get good and reliable results from MIKE 21 BW although a formal procedure is no substitute for common sense.

Chapter 4 Examples *(p. 25)* includes a number of simple and more compli-cated MIKE 21 BW applications. These are chosen to cover typical applica-tion areas of MIKE 21 BW. The emphasis in these examples is on how the parameters are selected and how the results are presented.

Chapter 5 Reference Manual *(p. 109)* describes the parameters in the MIKE 21 BW dialogues. It provides more details on specific aspects of the opera-tion of MIKE 21 BW and is what you will normally refer to for assistance if you are an experienced user. The contents of this chapter is the same as found in the Online Help.

In Chapter 6 Scientific Documentation *(p. 191)*, you can find information on where to find the scientific background for MIKE 21 BW.

An INDEX is found at the very end of this MIKE 21 BW User Guide.

1.2 Assumed User Background

Although MIKE 21 BW has been designed carefully with emphasis on a logi-cal and user-friendly interface and although the User Guide contains model-ling procedures and a large amount of reference material, common sense is always needed in any practical application.

In this case, "common sense" means a background in wave mechanics which is sufficient for you to be able to check whether the results from MIKE 21 BW are reasonable or not. This User Guide is not intended as a substitute for - and it cannot replace - a basic knowledge of the area in which you are work-ing: mathematical modelling of complex wave problems.

It is assumed that you are familiar with the basic elements of MIKE Zero: File types and file editors, the Plot Composer, the MIKE Zero Toolbox, the MIKE 21 Toolbox and the Bathymetry Editor. The documentation for these can be found from the MIKE Zero Documentation Index.

A step-by-step training guide on how to set up a MIKE 21 BW for a typical application is also available from the same place.

2 Introduction

2.1 General Description

The two modules included in the MIKE 21 BW are based on the numerical solution of time domain formulations of Boussinesq type equations. The Boussinesq equations include nonlinearity as well as frequency dispersion. Basically, the frequency dispersion is introduced in the momentum equations by taking into account the effect of vertical accelerations on the pressure distribution. Both modules solve the Boussinesq type equations using a flux-formulation with improved linear dispersion characteristics. These enhanced Boussinesq type equations (originally derived by Madsen et al, 1991, and Madsen and Sørensen, 1992)[1] make the modules suitable for simulation of the propagation of directional wave trains travelling from deep to shallow water. The maximum depth to deep-water wave length is $h/L_0 \approx 0.5$. For the classical Boussinesq equations the maximum depth to deep-water wave length is $h/L_0 \approx 0.22$.

The model has been extended into the surf zone by inclusion of wave breaking and moving shoreline as described in Madsen et al (1997a,b)[1], Sørensen and Sørensen (2001)[1] and Sørensen et al (1998, 2004).

Figure 2.1   MIKE 21 BW is a state-of-the-art numerical tool for studies and analysis of short and long period waves in ports and harbours and coastal areas

---

1 The papers are included in the Scientific Documentation

MIKE 21 BW is capable of reproducing the combined effects of all important wave phenomena of interest in port, harbour and coastal engineering. These include:

- Shoaling
- Refraction
- Diffraction
- Wave breaking
- Bottom friction
- Moving shoreline
- Partial reflection and transmission
- Non-linear wave-wave interaction
- Frequency spreading
- Directional spreading

Phenomena, such as wave grouping, surf beats, generation of bound sub-harmonics and super-harmonics and near-resonant triad interactions, can also be modelled using MIKE 21 BW. Thus, details like the generation and release of low-frequency oscillations due to primary wave transformation are well described in the model. This is of significant importance for harbour reso-nance, seiching and coastal processes.
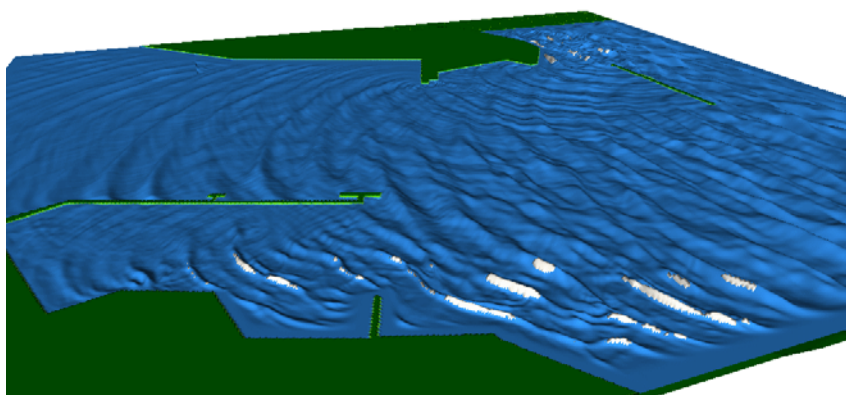
Figure 2.2   Simulation of wave propagation and agitation in a harbour area for an extreme wave event. The breaking waves (surface rollers) are shown in white

The present release of MIKE 21 BW includes two modules:

- 2DH Boussinesq Wave Module
- 1DH Boussinesq Wave Module

The 2DH module (two horizontal space co-ordinates) solves the enhanced Boussinesq equations by an implicit finite difference technique with variables defined on a space-staggered rectangular grid.

The 1DH module (one horizontal space co-ordinates) solves the enhanced Boussinesq equations by a standard Galerkin finite element method with mixed interpolation for variables defined on an unstructured (or a structured) grid. Surf zone dynamics and swash zone oscillations can be simulated for any coastal profile in this module.
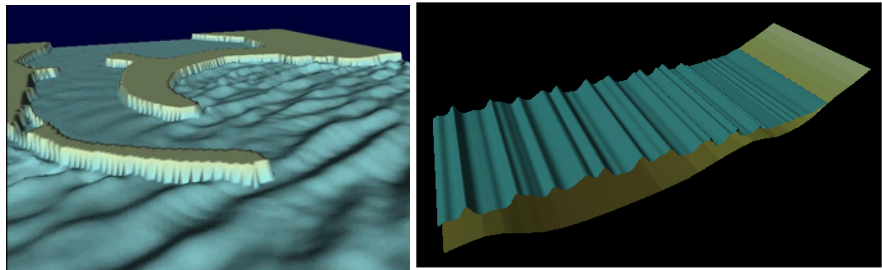
Figure 2.3   The MIKE 21 BW includes two modules. The 2DH module (left panel) is traditionally applied for calculation of wave disturbance in ports and harbours. The 1DH module (right panel) is selected for calculation of wave transformation from offshore to the beach for the study of surf zone and swash zone dynamics

MIKE 21 BW also includes porosity for the simulation of partial reflection from and transmission through piers and breakwaters. Sponge layers are applied when full absorption of wave energy is required. Finally, MIKE 21 BW also includes internal generation of waves.

2.1.1   Application Areas

A major application area of MIKE 21 BW is determination and assessment of wave dynamics in ports and harbours and in coastal areas. The disturbance inside harbour basins is one of the most important factors when engineers are to select construction sites and determine the optimum harbour layout in relation to predefined criteria for acceptable wave disturbance, ship movements, mooring arrangements and handling down-time.

With inclusion of wave breaking and moving shoreline MIKE 21 BW is also an efficient tool for the study of many complicated coastal phenomena, e.g. wave induced-current patterns in areas with complex structures.

2DH Boussinesq wave module

Applications related to the 2DH module include:

- determination of wave disturbance caused by wind-waves and swell

- analysis of low-frequency oscillations
  (seiching and harbour resonance) caused by forcing of e.g. short-wave induced long waves

- wave transformation in coastal areas where reflection and/or diffraction are important phenomena

- Surf zone calculations including wave-induced circulation and run-up/run-down

- simulation of propagation and transformation of transients such as ship-generated waves and tsunamis

Assessment of low-frequency motions in existing as well as new harbours is often performed using a combination of simulations with synthetic white-noise spectra and simulations with natural wave conditions. The purpose of the former type of simulations is to investigate the potential of seiching/resonance and identification of natural frequencies. This is particularly useful for relative comparisons between different layouts.

MIKE 21 BW is also applied for prediction and analysis of the impact of ship-generated waves (also denoted as wake wash). Essential boundary conditions (at open or internal boundaries) for the models can be obtained from 3D computational fluid dynamic (CFD) models, experimental data, full-scale data and/or empirical relationships.
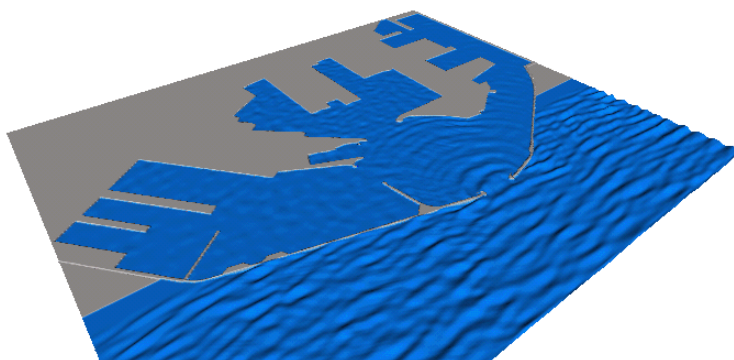
Figure 2.4   Simulation of wave penetration into Frederikshavn harbour, Denmark

01/01/90 12:33:36.000
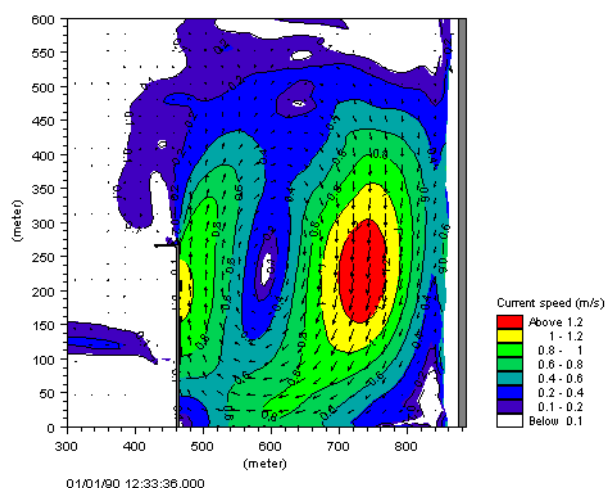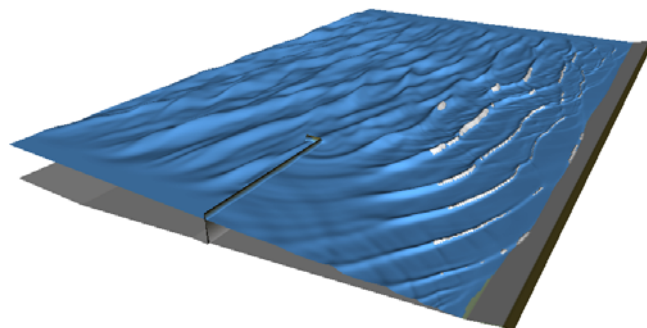
Figure 2.5   Wave transformation, wave breaking and run-up in the vicinity of a detached breakwater parallel to the shoreline. The lower image shows the associated circulation cell behind the breakwater

1DH Boussinesq wave module

Applications related to the 1DH module include:

- computation of wave transformation for nonlinear waves from deep water, through the surf zone and all the way up to the beach.

- analysis of generation and release of low-frequency waves.

- assessment of wave breaking, undertow and run-up on dikes, revetments and beaches.

The 1DH module can be applied for a number of transects (one spatial dimension) where surf zone dynamics and swash zone dynamics are simulated in real-time.

With the inclusion of wave breaking not only the spatial variation of e.g. the significant wave height, maximum wave height and wave setup on the beach can be computed, but also details like the generation and release of low-frequency energy due to primary wave transformation can be computed. This is of significant importance for harbour resonance (seiching) and coastal processes.
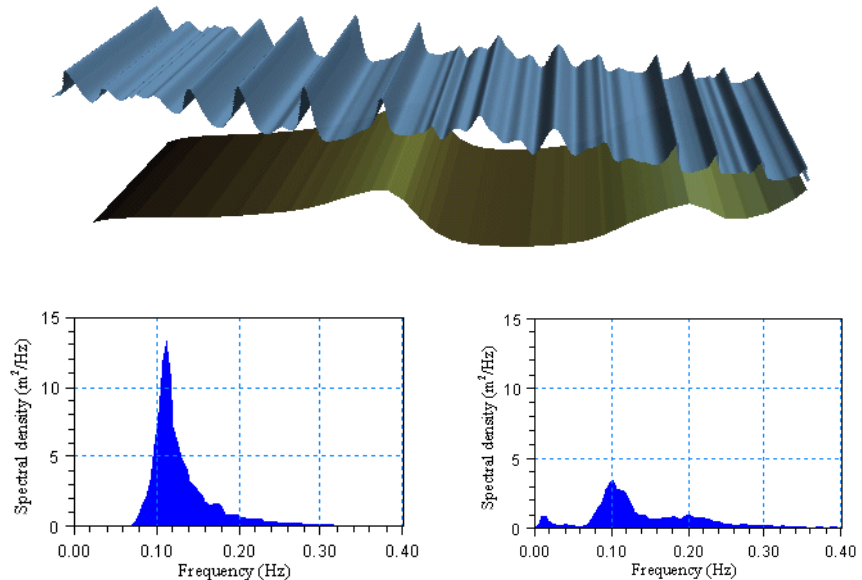

Figure 2.6   Transformation of irregular non-linear waves over a natural barred beach profile (upper panel). Offshore (left) and onshore (right) frequency wave spectra (lower panels). The spectra are computed using the WSWAT Linear Spectral Wave Analysis module included in MIKE Zero

3 Getting Started

3.1 General

The purpose of this chapter is to give you a general check list, which you can use for determination and assessment of wave dynamics in ports, harbours and coastal areas using the MIKE 21 Boussinesq Wave model.

The work will normally consist of the six tasks listed below:

- Defining and limiting the wave problem
- Collecting data
- Setting up the model
- Calibrating and verifying the model
- Running the production simulations
- Presenting the results

Each of these six tasks are described for a "general wave study" in the following sections. For your particular study only some of the tasks might be relevant.

Please note that whenever a word is written in *italics* it is included as an entry in the *Online Help* and in the Reference Manual.

3.2 Defining and Limiting the Wave Problem

3.2.1 Identify the wave problem

When preparing to do a study of e.g. wave disturbance in a harbour, you have to assess the following before you start to set up the model:

- What are the "wave conditions" under consideration in the "area of interest"?

- What are the "important wave phenomena"? The following phenomena should be taken into consideration:
  - Shoaling
  - Refraction
  - Diffraction
  - Partial reflection/transmission
  - Bottom dissipation
  - Wave breaking
  - Run-up
  - Wind-wave generation
  - Frequency spreading

– Directional spreading
– Wave-wave interaction
– Wave-current interaction

The MIKE 21 BW module can handle these phenomena with the exception of wind-wave generation.

3.2.2 Check MIKE 21 BW capabilities

Next, check if the MIKE 21 BW module is able to solve your problem. This you can do by turning to Chapter 2, which gives a short description of MIKE 21 BW and an overview of the type of applications for which MIKE 21 BW can be used, and by consulting the Scientific Documentation, section 6.

3.2.3 Selecting model area spectral and temporal resolution

When selecting the model area (or profile) you must consider the area of interest, the alignment of the model grid relative to the main direction of approach of the incident wave trains and the position and types of model boundaries to be used, see *Selecting the Model Area* under *Bathymetry*.

The choice of the grid spacing and time step depends on the wave conditions for which simulations are to be performed and the water depth in the area of interest.

- The ratio of the maximum water depth to the deep water wave length of waves with the shortest wave period must not become larger than 0.22, if the deep water correction terms are excluded, and 0.5, if these terms are included. See *Linear Dispersion Relation*

- The grid spacing is restricted by the resolution of the shortest wave length or the surface roller, if wave breaking is included. See *Selecting Grid Spacing* under *Bathymetry*

- The time step is restricted by the resolution of the shortest wave period. See *Time Step*

- The Courant number should be kept equal or less than unity (0.5 for the 1DH module) to avoid instability problems. See *Courant Number*

In practice the choice of the grid spacing and time step is often a compromise between low computer costs and high accuracy.

The MIKE 21 BW Model Setup Planner (Figure 5.7) is an efficient tool for the setup of your model.

3.2.4 Check computer resources

Finally, before you start to set up the model, you should check that you are not requesting unrealistic computer resources:

- The *CPU time* required should be estimated.
- The *RAM* requirements should be estimated.
- The *Disk Space* required should be estimated.

An estimate of the required CPU and RAM can easily be obtained using the MIKE 21 BW Model Setup Planner integrated in the Online Help. The disk space is assessed through the MIKE 21 BW Editor.

Particularly when wave breaking and moving shoreline are included in the 2DH module (space resolution is 1-2 m and temporal resolution 0.02-0.2 s as typically values) the required CPU time and memory demand can be high (> 12 hours) for large computational domains.

3.3 Collecting Data

This task may take a long time if, for example, you have to initiate a monitoring program. Alternatively it may be carried out very quickly if you are able to use existing data which are immediately available. In all cases the following data should be collected:

- Bathymetric data such as charts from local surveys or, for example, from the Hydrographic Office, UK, or MIKE C-MAP

- Boundary data, which might be measurements (existing or planned specifically for your model), observations, wave statistics, etc.

- Information on type of structures for assessment of the reflection properties

- Calibration and validation data; these might be measured wave parameters at selected locations, observations, etc.

3.4 Setting up the Model

3.4.1 What does it mean

"Setting up the model" is actually another way of saying transforming real world events and data into a format which can be understood by the numerical model MIKE 21 BW. Thus generally speaking, all the data collected have to be resolved on the spatial grid selected.

3.4.2 Bathymetry

You have to specify the bathymetry as a type 2 or type 1 data file containing the water depth covering the model area. Describing the water depth in your model is one of the most important tasks in the modelling process. A few hours less spent in setting up the model bathymetry may later on mean extra days spent in the calibration process.

3.4.3 Sponge layer

In practical for all MIKE 21 BW applications you have to prepare maps (2DH, dfs2-file) or profile series (1DH, dfs1-file) for efficient absorbtion of short and long period waves, see section 5.3.11 (Sponge layers).

3.4.4 Porosity layer

Modelling of partial reflection/transmission requires preparation of maps (2DH) or files (1DH) including porosity layers. An efficient procedure is outlined in section 5.3.10 (Porosity layers).

3.4.5 Boundary data

In most cases you will force the model by waves generated inside the model domain. The *internal wave generation* of waves allows you to absorb all waves leaving the model domain (radiation type boundaries).

3.5 Calibrating and Verifying the Model

3.5.1 Purpose

Having completed all the tasks listed above you are ready to do the first time-domain wave simulation and to start on the calibration of the model.

The purpose the calibration is to tune the model in order to reproduce known/measured wave conditions. The calibrated/tuned model is then verified by running one or more simulations for which measurements are available without changing any tuning parameters. This should ensure that simulations can be made for any wave conditions similar to the calibration and verification wave conditions with satisfactory results. However, you should never use simulation results, whether verified or not, without checking if they are reasonable or not.

3.5.2 Verification

The situations which you select for calibration and verification of the model should cover the range of situations you wish to investigate in the production runs. However, as you must have some measurements/observations against which to calibrate and, as the measurements are often only available for short periods, you may only have a few situations from which to choose.

When you have finished the calibration you can run one more simulation for which you have measurements (or other data) without changing the calibration parameters. If you then get a satisfactory agreement between the simulation results and the measurements you can consider your calibration to be successful.

3.5.3 Calibration parameters

When you run your calibration run for the first time and compare the simulation results to your measurements (or other information) you will, in many cases, see differences between the two. The purpose of the calibration is then to tune the model so that these differences become negligible. You can change the following model specifications in order to reduce the differences:

- Wave conditions
- Porosity
- Bed resistance
- Bathymetry
- Wave breaking
- Moving shore line

Recommendations on how the specification can be changed are given in the Reference Manual.

3.6 Running the Production Simulations

As you have calibrated and verified the model you can get on to the "real" work, that is doing your actual investigation. This will, in some cases, only include a few runs.

3.7 Presenting the Results

Throughout a modelling study you are working with large amounts of data and the best way of checking them is therefore to look at them graphically. Only in a few cases, such as when you check your bathymetry along a boundary or you want to compare simulation results to measurements in selected locations, should you look at the individual numbers. Much empha-

sis has therefore been placed on the capabilities for graphical presentation in MIKE Zero and it is an area which will be expanded and focused on even further in future versions (e.g. GIS).

Essentially, one plot gives more information than scores of tables and if you can present it in colours, your message will be even more easily understood.

A good way of presenting the model results is using contour plots of e.g. the calculated wave disturbance coefficient by using the Plot Composer tool in MIKE Zero.

Instantaneous pictures/videos of the simulated surface elevations can also be generated in the Plot Composer. For 3D visualisation MIKE Animator Plus is recommended.

3.8 A Quick Guide for MIKE 21 BW Model Simulation Setup

This section includes a chronological list of tasks you should pass through when setting up a MIKE 21 BW model simulation. Emphasis is put on applications using the 2DH module.

In addition a step-by-step training guide on how to set up a MIKE 21 BW model for a typical application can be accessed online via the Documentation index.

Providing MIKE21 BW with a suitable bathymetry (dfs2 data file for 2DH applications and dfs1 data file for 1DH applications) is essential for obtaining reliable results from the model. Based on available xyz data for the area of interest you create the model bathymetry by use of the **MIKE Zero tool Bathymetry Editor**. The above mentioned step-by-step guide shows how to generate a bathymetry from scratch.

1. When setting up the bathymetry it should be kept in mind that the bathymetry determines which wave conditions that can be modelled. The maximum water depths restrict the minimum wave period that can be modelled and the minimum water depth may restrict the wave height if wave breaking is not included. Therefore it can be necessary to modify the bathymetry to reach an acceptable compromise between a correct bathymetry and correct wave conditions. Please use the **MIKE 21 BW Model Setup Planner** Java Script embedded in the Online Help when you are setting up the bathymetry.

   Following ranges of grid spacing are typically used in practical applications

   – Δx = 2-10 m for the 2DH module without wave breaking
   – Δx = 1-2 m for the 2DH module including wave breaking and moving shoreline

    – Δx = 0.1-10 m for the 1DH module (unstructured mesh) with wave breaking and moving shoreline

The next step is to generate a sponge layer map. Sponge (or absorbing) layers are used as wave absorbers. These may be set up along model boundaries to provide radiation boundary conditions, which absorb wave energy propagating out of the model area. Sponge layers may also be used along shorelines (see e.g. the Rønne Harbour example and Torsminde Harbour example in this User Guide). The sponge layer map is easily created using the **MIKE 21 Toolbox Generate Sponge and Porosity Layer Map** tool.

2. For simulation of partial wave reflection and/or wave transmission through various types of structures you would need to create a porosity layer map. The porosity layer map includes a porosity, which is set to unity at open water points (no dissipation) and between 0.2 and 1 along structures where you want to include the dissipation effect of porous flow. If a porosity value is backed up a land value (> 0), partial reflection will take place. Conversely, (partial) transmission will also take place if land points do not back up the porosity values.

   An efficient step-by-step procedure for establishment of a porosity map is described in this User Guide, see Section 5.3.10. The porosity map is easily created using the **MIKE 21 Toolbox Generate Sponge and Porosity Layer Map** tool. The porosity values are calculated using the **MIKE 21 Toolbox Calculation of Reflection Coefficient**.

3. If wave breaking and moving shoreline is included you would most often also have to specify a map (2DH, dfs2 data file) or profile (1DH, dfs1 data file) including filter coefficients. This lowpass filter is introduced to remove high-frequency instabilities during uprush/downrush and to dissipate the wave energy in the area where the surface rollers can not be resolved properly.

4. Next step is to prepare wave boundary conditions for the model. In most applications you will force the model by waves generated inside the model domain, i.e. using internal wave generation. Internal wave generation is performed by adding the discharge of an incident wave field along one or more generation lines. One of the advantages of using internal generation is that sponge layers can be placed behind the generation line, to absorb waves leaving the model domain (radiation type boundary).

5. The format of the internal wave generation data depends on type of waves; regular, irregular or directional. For generation of regular wave data you should use the **MIKE 21 Toolbox Regular Wave Generation** tool and for irregular and directional waves **MIKE 21 Toolbox Random**

Wave Generation.

6. Now you are ready to set up up your Boussinesq model run; open MIKE Zero → New → MIKE 21 → Boussinesq Waves. It is recommend using the default values in the dialogs in your first model run. If you have decided to include the deep water terms during your model setup (i.e. allowance of shorter waves) then the deep water terms should of course be included in the "Type of Equation" dialog. Regarding specification of output parameters it is recommended to save maps (dfs2 files) and time series (dfs0 files) of both deterministic parameters (e.g. the surface elevation) and phase-averaged parameters (e.g. the significant and maximum wave height). For harbour agitation studies also maps of the wave disturbance coefficient should be saved.

7. It is recommended to view the results during the model run to make sure everything is as expected. This can be made directly from the editor by clicking on the "View" button, which automatically opens an editor for viewing the model results.

8. After the simulation you can use various visualization tools included in MIKE Zero for result presentation. Use e.g. the **Plot Composer Grid Plot** for creation of plots and 2D animations of wave disturbance coefficient and instantaneous surface elevation. For 3D visualization **MIKE Animator Plus** is recommended (license required).

9. Additional analysis (of both dfs0 and dfs2 data) including spectral, directional, filtering or crossing analysis of the model output can be made using the **WS Wave Analysis Tool** in MIKE Zero (license required).

4 Examples

4.1 General

One of the best ways of learning how to use a modelling system like MIKE 21 is through practice. Therefore we have included a number of applications for each of the two modules which you can go through yourself and which you can modify, if you like, in order to see what happens if some of the parameters are changed.

The specification data files for the examples are included with the installation of MIKE 21. For each example a directory is provided. The directory names are as follows:

2DH Boussinesq Wave Module

- Numerical flume test:
  .\Examples\MIKE_21\BW\Flume

- Diffraction test:
  .\Examples\MIKE_21\BW\Diffraction

- Rønne Harbour:
  .\Examples\MIKE_21\BW\Ronne

- Hanstholm Harbour:
  .\Examples\MIKE_21\BW\Hanstholm

- Torsminde Harbour:
  .\Examples\MIKE_21\BW\Torsminde_Harbour

- Island:
  .\Examples\MIKE_21\BW\Island

- Rip channel:
  .\Examples\MIKE_21\BW\Rip_Channel

- Detached breakwater:
  .\Examples\MIKE_21\BW\Detached_Breakwater

- Kirkwall Marina
  .\Examples\MIKE_21\BW\Kirkwall_Marina

- Demo-Diffraction
  .\Examples\MIKE_21\BW\Demo-Diffraction

1DH Boussinesq Wave Module

- Partial reflection test:
  .\Examples\MIKE_21\BW\Partial_Reflection

- Sloping beach with wave breaking and moving shoreline:
  .\Examples\MIKE_21\BW\Sloping_Beach

- Barred beach with wave breaking:
  .\Examples\MIKE_21\BW\Torsminde_Barred_Beach

Please note that the layout of some figures and illustrations presented in this release of the User Guide is optimised for the integrated Online Help and digital version of this User Guide.

4.2 2DH Boussinesq Wave Module - Examples

4.2.1 Numerical flume test

Purpose of the example

This fairly simple example is included to illustrate the propagation of irregular waves in a numerical wave flume using the 2DH Boussinesq Wave Module.

Model setup

The model setup is illustrated in Figure 4.1. The length (in the model east/west direction) of the flume is 350 m and the width (in the model north/south direction) is 55 m. The uniform depth is 10 m.
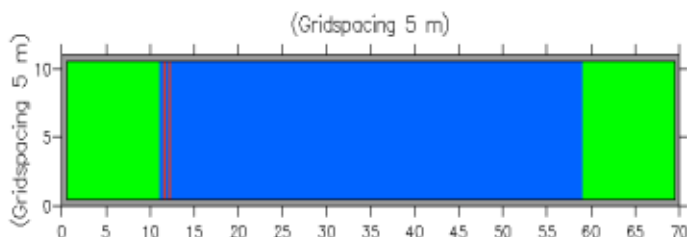
Figure 4.1   Model layout. The 10 point wide areas indicate sponge layers and the line at j=12 indicates the internal wave generation line

The waves are generated internally inside the model domain, which is the most common method of forcing a Boussinesq wave model in practical applications. Thus, all model boundaries are closed, i.e. considered as land points. At the generation line (j= 12) a time series of fluxes is imposed. This time series is generated using the *MIKE 21 Toolbox* (wave part) program *Random Wave Generation*. The significant wave height is H_{m0} = 1 m and the spectral peak period is T_p =10 s. The waves are synthesised based on a mean JONSWAP spectrum. As the minimum wave period is T_{min} = 6 s, the

problem can be solved using the classical Boussinesq equations (i.e. the dispersion coefficient is B= 0).

The grid spacing is selected to be 5 m and the time step is 0.5 s. The simulation duration is 10 minutes (1201 time steps).

Although a 10 point thick sponge layer is used in this example you are in general recommended to apply at least 20 point wide sponge layers in most practical applications of the 2DH Boussinesq Wave Module. Excellent wave absorbing characteristics have been obtained for sponge layers having a thickness of 1-2 times the wave length of the most energetic waves (corresponding to the spectral peak wave period). In this example the thickness of the 10 point wide sponge layer is 0.5-0.6 times the wave length corresponding to the spectral peak wave period. However, from the results shown in Figure 4.2 (panel showing the significant wave height) it is hard to detect any spurious reflections from the sponge layers.

Model results

A time series of the simulated surface elevation extracted at point P(35,5) is shown in Figure 4.2, as well as the spatial variation of the significant wave height along the channel. Note that the significant wave height is less than 1 m as the incident wave spectrum is not re-scaled.
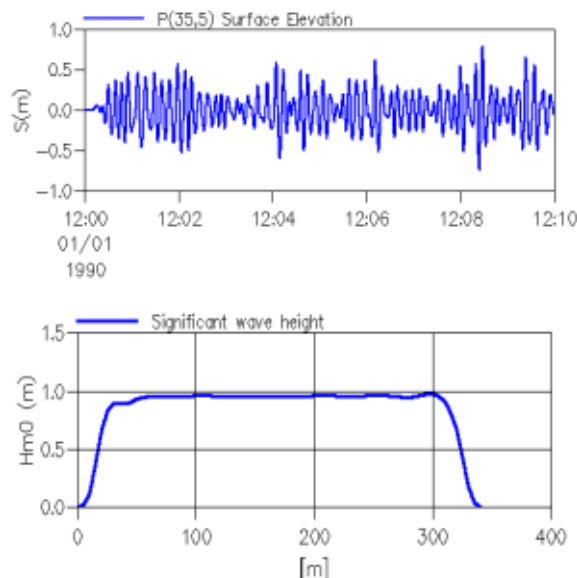

Figure 4.2   Model results. Time series of simulated surface elevation at a given point (upper panel) and line series of the stationary significant wave height along the centre line (lower panel)

List of data and parameter files

All data required for this example is included in the default installation:

Binary data files

Name: Layout.dfs2
Description: Bathymetry and sponge layer coefficients

Name: IBC.dfs0
Description: Boundary conditions – time series of flux and surface elevation slope

Parameter files

Name: Flume.bw
Description: Numerical Flume test

4.2.2   Diffraction test

Purpose of the example

This example is included to illustrate the phenomenon of diffraction. Diffraction is an important process in regions, which are sheltered by barriers such as breakwaters, jetties or inlets.

The example describes what happens when a single fully reflecting breakwater interrupts a regular wave train.

Model setup

The model setup is illustrated in Figure 4.3. The model size is 1200 m by 700 m with a uniform depth of 40 m. Wave absorbing sponge layers are applied at the eastern and northern model boundary, while the western boundary is treated as fully reflective. The area of interest is the shadow zone behind the breakwater. Due to radiation of energy from the point of diffraction the western boundary has to be placed quite far from this area to avoid reflections.
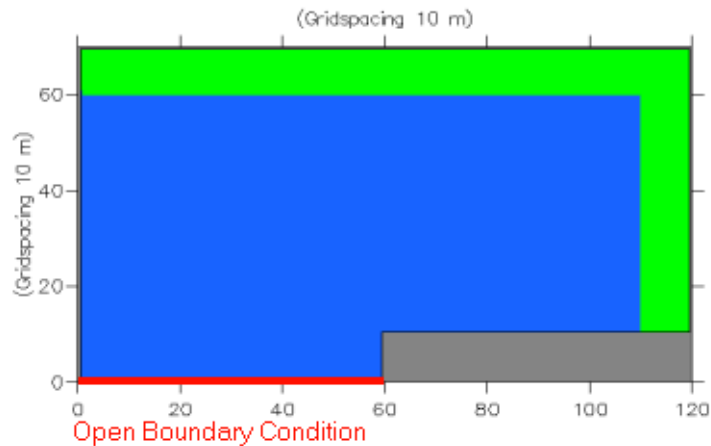
Figure 4.3   Model layout. The 10 point wide areas indicate sponge layers and the line at k=0 the open essential boundary. The thickness of the sponge layer is about one times the wave length

At the open south boundary a flux boundary is applied. Sinusoidal waves with a wave period of 8 s and wave height of 1m is generated. According to linear theory the wave length is approximately 100 m. Thus, the simulation requires the so-called "deep-water terms" to be included, i.e. it is the enhanced Boussinesq type equations which have to be solved in this case.

The grid spacing is chosen to be 10 m representing the waves with approximately 10 points per wave length. The time step is chosen to be 0.25 s representing the waves with 32 points per wave period.

The simulation period is chosen to be 200 s corresponding to 800 time steps.

Model results

The model results are presented in Figure 4.4 showing a contour plot of the instantaneous surface elevation and a map showing the isolines of the wave disturbance coefficient in the entire domain.
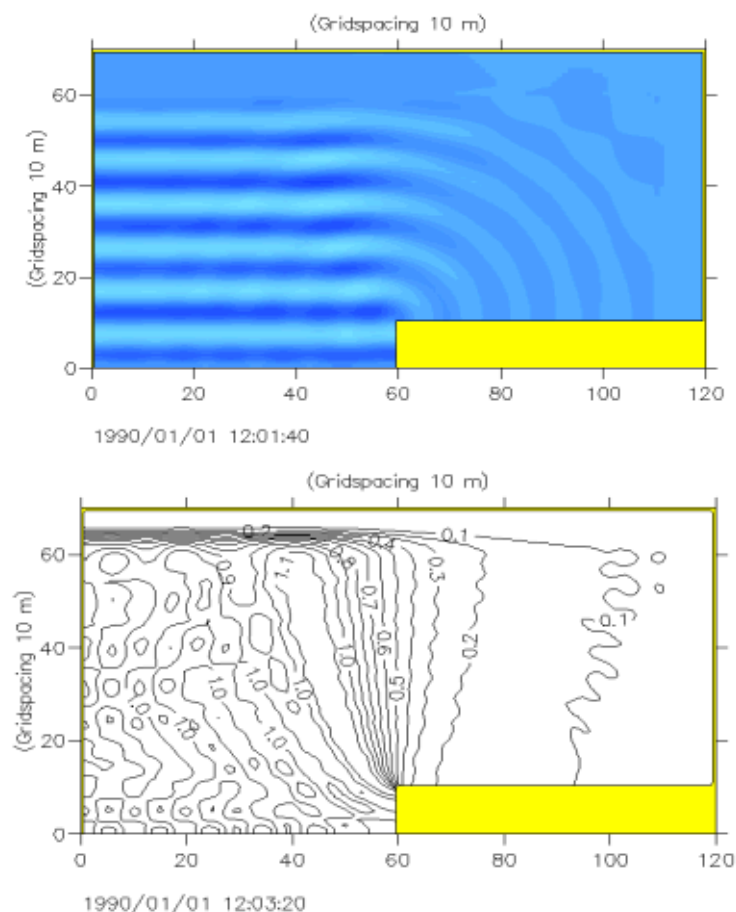
Figure 4.4   Model results. The upper panel shows the instantaneous surface eleva-
tion and the lower panel the wave disturbance coefficient behind a fully
reflective breakwater

The model results are in good agreement with semi-analytical solutions to the
Helmholtz equation, see e.g. Shore Protection Manual (1984), see section
5.5.16.

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files

Name: Layout.dfs2

Description: Bathymetry and sponge layer coefficients

Name: Bc.dfs0

Description: Boundary conditions – time series of surface elevation and its curvature

Parameter files

Name: Diffraction.bw

Description: Diffraction test

4.2.3   Rønne Harbour

Purpose of the example

The purpose of this example is to simulate the wave disturbance in Rønne harbour, Denmark, situated in the Baltic Sea. Of special interests is wave disturbance at the new cruise terminal, see Figure 4.5.

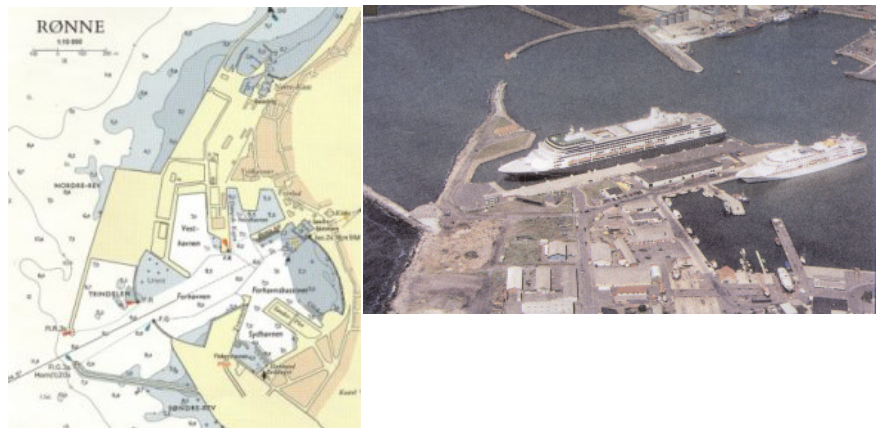Figure 4.5   Right panel shows the new cruise ship terminal in Rønne harbour, Denmark. Left panel shows the harbour layout before the cruise terminal was constructed

Model setup

The model setup is illustrated in Figure 4.6.

The model area is chosen based on following considerations:

The model is turned –22.5° relative to true north to align the model grid relative to the main direction of the approach of incoming waves.

To reduce the number of water points artificial land is introduced outside the main breakwaters of the entrance leaving only a small area outside the harbour from where the waves can enter the harbour. This limits the application to unidirectional incoming waves from direction WSW (247.5° N).

Artificial land is also introduced in the innermost part of the harbour. This area is backup by a 10 point wide sponge layer in order to absorb the waves here. In reality the waves will break at a small beach here, but this will not affect the waves in the central part of the harbour.
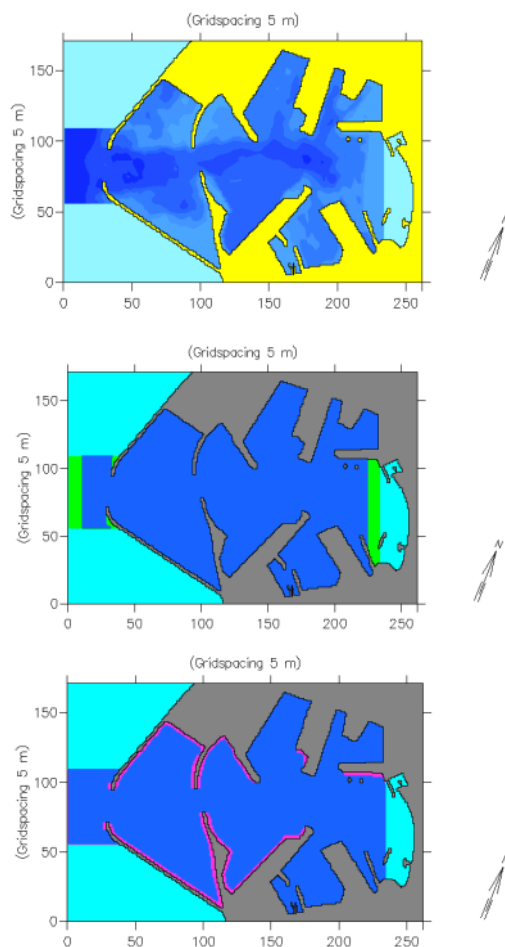
Figure 4.6   Model setup for Rønne Harbour. Upper panel shows the bathymetry, the middle panel the absorbing sponge layers (10 point wide) and the lower panel the areas where partial wave reflection is required (3-5 point wide)

The maximum water depth is approximately 11 m in the model area. The event to be simulated corresponds to a situation occurring 10 hours per year (on average) and is characterised by having a significant wave height of H_{m0}

= 2.65 m with a spectral peak period is T_p = 8.6 s. The waves are synthesised based on a mean JONSWAP spectrum, as the minimum wave period is set to T_{min} = 5.7s. The wave disturbance problem can be solved using the classical Boussinesq equations (i.e. B= 0). Please note that the truncated wave spectrum is not rescaled, i.e. the incoming wave height is less than 2.65 m.

A grid spacing of 5 m is chosen and a time step of 0.4 s. This results in a maximum Courant number of 1 in the deepest part of the model. The length of the simulation is 12 minutes (corresponding to 1801 time steps).

Model results

The model results are presented in Figure 4.7 showing a contour plot of the instantaneous surface elevation and a map showing the isolines of the simulated wave disturbance coefficients (after 12 minutes).

Figure 4.8 shows a 3D plot of the instantaneous surface elevation for entire harbour.

The wave disturbance coefficient along the main quay wall of the cruise terminal is 0.05-0.20 for this event. Other statistics (max., min., mean, std. dev. and number of data) for the two areas defined in the area code map file can be found in the ASCII file named *WaveDisturbanceAlongNewQuay.txt*.
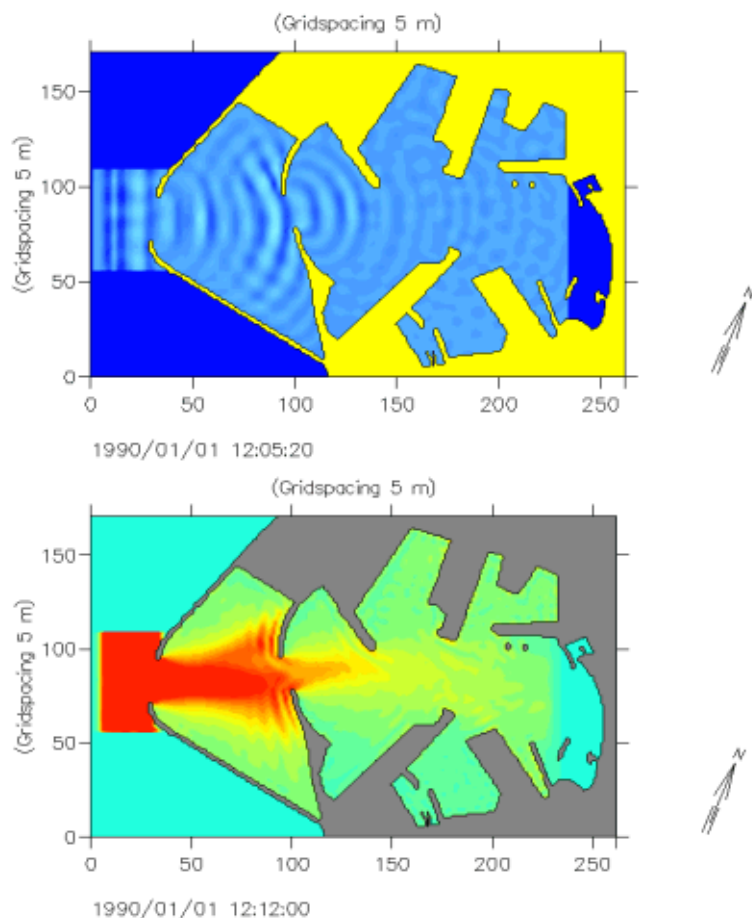
Figure 4.7   Model results. The upper panel shows the instantaneous surface eleva-
              tion and the lower panel the wave disturbance
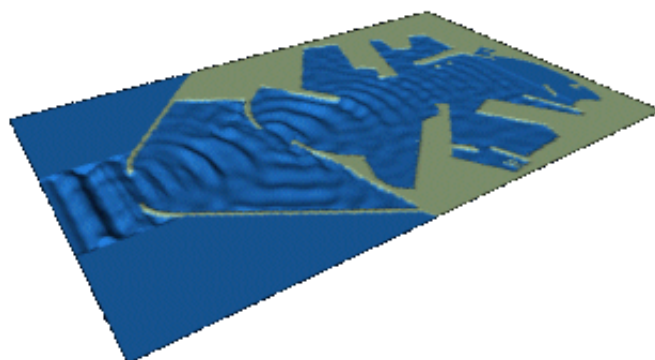
Figure 4.8   3D picture of the simulated instantaneous surface elevation in Rønne
              harbour

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files

Name: Layout1998.dfs2
Description: Bathymetry, sponge layer and porosity coefficients

Name: Ibc.dfs0
Description: Boundary conditions – time series of flux

Name: CodeMap.dfs2
Description: Area code map

Parameter files

Name: Ronne1998.bw
Description: Wave disturbance test

4.2.4   Hanstholm Harbour

Purpose of the example

The purpose of this example is primarily to illustrate how to use two internal generation lines for obliquely incident unidirectional waves. Hanstholm harbour, Denmark, situated at the Danish west coast, is used in this case, see Figure 4.9 and 4.10. In this example the enhanced Boussinesq type equations are solved, i.e. the deep-water terms are included.

Figure 4.9   Hanstholm harbour, Denmark

Figure 4.10   Location of Hanstholm harbour

Model setup

The model setup is illustrated in Figure 4.11.

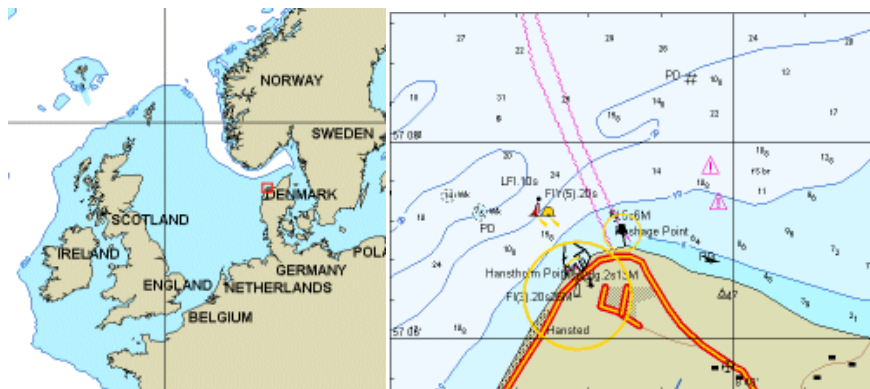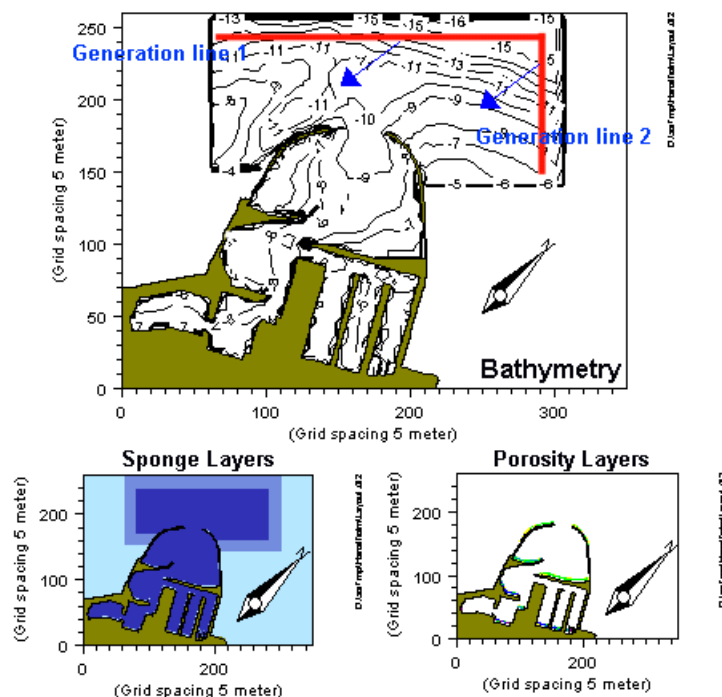Figure 4.11 Model setup for Hanstholm harbour. Upper panel shows the bathymetry, the lower left panel the absorbing sponge layers (20 point wide) and the lower right panel the areas where partial wave reflection is required (3-5 points wide)

In this example we consider a case having H_{m0}= 1.0 m and T_p= 8 s. The waves are coming from N and the minimum period of interest is set to T_{min}= 5 s. As the maximum water depth in the model area is h_{max}= 17.5m it is necessary to include the deep-water terms (h_{max}/L_o= 0.45). The minimum depth is h_{min}= 4 m. The incoming waves are synthesised based on a mean JONSWAP spectrum.

For visualisation purposes oblique incident unidirectional irregular waves are used (you can modify the specification, if you like, in order to see what happens if e.g. directional waves are applied). The input data files are generated using the *Random Wave Generation* tool in the *MIKE 21 Toolbox* (open the file Hanstholm_Harbour_Preprocessing_Data.21T by double clicking).

To reduce the computational time artificial land is introduced outside the main breakwaters of the entrance leaving only a smaller area outside the harbour from where the waves can enter the harbour.

A grid spacing of 5 m is chosen and a time step of 0.2 s. This results in about 6 grid points per minimum wave length, 25 time steps per minimum period and a maximum Courant number of 0.52 in the deepest part of the model area. Please note that the space- and time-resolution in this example is less

than the recommended values (e.g. obtained using the Java Script *MIKE 21 BW Model Planner* included in the Online Help), which are minimum 7 grid points per wave length and about 35 time steps per minimum wave period. The coarser numerical resolution is chosen in order to reduce the overall computational time and would affect the simulated wave disturbance coefficient marginally.

The length of the simulation is 25 minutes (corresponding to 7501 time steps).

Model results

The model results are presented in Figure 4.12 showing a contour plot of the instantaneous surface elevation and a map showing the isolines of the simulated wave disturbance coefficients (after 25 minutes). Figure 4.13 shows time series of the simulated surface elevation inside and outside the harbour.
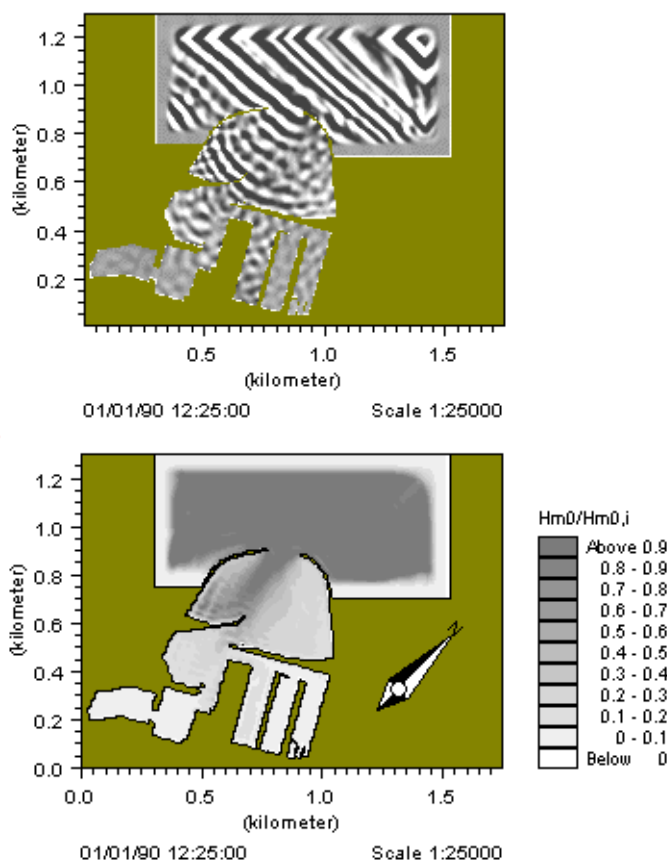
Figure 4.12   Model results. The upper panel shows the instantaneous surface elevation and the lower panel the wave disturbance

Figure 4.14 shows a 3D plot of the instantaneous surface elevation for the entire harbour.
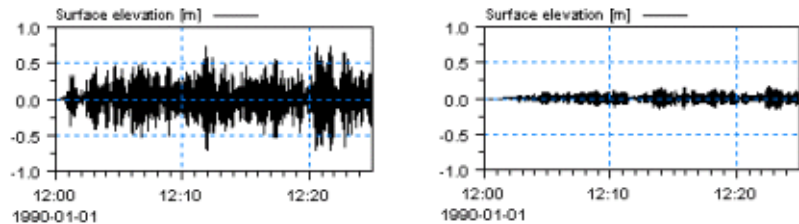
Figure 4.13   Model results. Time series of simulated surface elevation at the main harbour entrance (left panel) and at the ferry terminal quay (right panel), see Figure 4.9
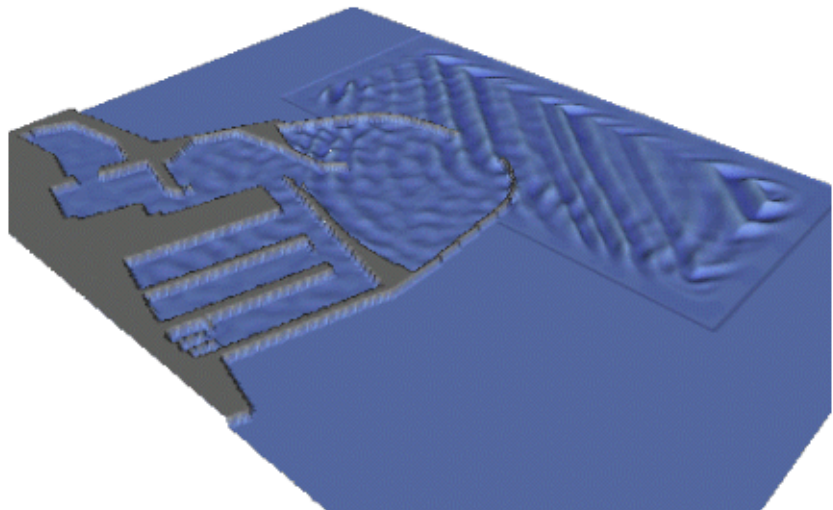
Figure 4.14   3D picture of the simulated instantaneous surface elevation in Hanstholm harbour

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files

Name: Layout.dfs2

Description: Bathymetry, sponge layer and porosity coefficients

Parameter files

Name: Hanstholm.bw

Description: Wave disturbance test

Name: Hanstholm_Harbour_Preprocessing_Data.21T

Description: Generation of wave input along the two generation lines. You have to generate the wave input prior to model execution.

Name: Hanstholm_Harbour_Preprocessing_Plot.plc

Description: Specification for Plot Composer (prior to model run). Please double-click the file after you have executed Hanstholm_Harbour_Preprocessing_Data.21T.

Name: Hanstholm_Harbour_Postprocessing_Plot.plc

Description: Specification for Plot Composer (after model run). Please double-click the file after you have executed Hanstholm.bw.

4.2.5 Torsminde Harbour

Purpose of the example

This example illustrates how MIKE 21 BW can be used for simulation of long-period waves (or seiches) inside a harbour. Torsminde Harbour situated at the Danish west coasts (see Figure 4.15) has suffered of seiches causing moored vessels move to-and-fro from their berthing positions resulting in breaking of mooring lines, damages on fenders and piers and sometimes collisions of vessels with each other. Field measurements show substantial long-period wave energy on wave periods 30-60 s in the harbour.
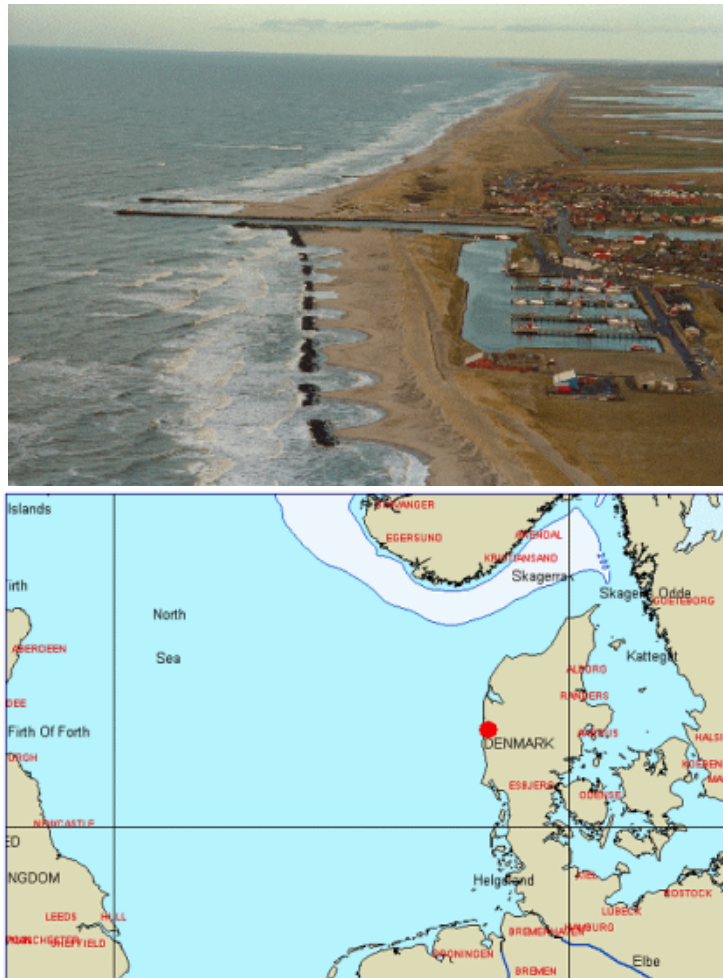
Figure 4.15   Torsminde Harbour, Denmark. The red dot in the lower panel shows the location

In this example the occurrence of the long-period waves is caused by interactions of short-period wind waves. The shoaling and breaking of the short-period wave trains are non-linear processes involving a number of complicated details. Quadratic interactions between harmonics in shallow water lead to substantial cross-spectral energy transfer over relatively short distances, and during shoaling still more energy will be transferred into bound sub-harmonics and super-harmonics. In a number of situations, part of the energy may be released as free harmonics, e.g. during passage over submerged bars or reefs, during diffraction around a breakwater, or during the process of breaking. When wave breaking occurs, the primary waves will start dissipating, and this will allow for a gradual release of the bound sub-harmonics and result in free long waves moving towards the shoreline and harbour entrance.

Wave breaking is included in this 2DH module application and a wave event occurring approximately 30 hours/month is considered (H_{m0}= 4.0 m, T_p= 9 s, MWD= 270 ºN).

Model setup

The model bathymetry is illustrated in Figure 4.16 where the maximum depth is 11.5 m and minimum depth is 1 m. The spatial resolution is 2 m. We will consider a wave situation where substantial wave breaking will occur:

- Significant wave height, H_{m0}= 4.0 m

- Peak wave period, T_p= 9 s

- Standard JONSWAP spectrum

- Minimum wave period T_{min}= 4.1 s

- Mean wave direction, MWD= 270ºN

- Cos^8-directional spreading

The most energetic and breaking waves have a period around 9 s and are resolved by the 20-40 points per wave length at depths 2-10 m, where the primary part of the waves will break. In most practical 2DH applications including wave breaking, a grid spacing of 1-2 m is used for peak wave periods larger than, say, 7s.

The model bathymetry is generated by using the Bathymetry Editor and algebraic manipulations in the Grid Editor.

As shown in Figure 4.17, a 20-point wide sponge layer has been set up at the shoreline. Hence wave run-up is not considered in this example. The minimum model depth of 1m is well inside the sponge layer. A 50 points wide sponge layer is used for efficient wave absorption at the other boundaries. The thickness of the sponge layer is here corresponding to one wave length or more. The sponge layer map is generated using the MIKE 21 Toolbox 'Generate Sponge and Porosity Layer Maps' and algebraic manipulations in the Grid Editor.

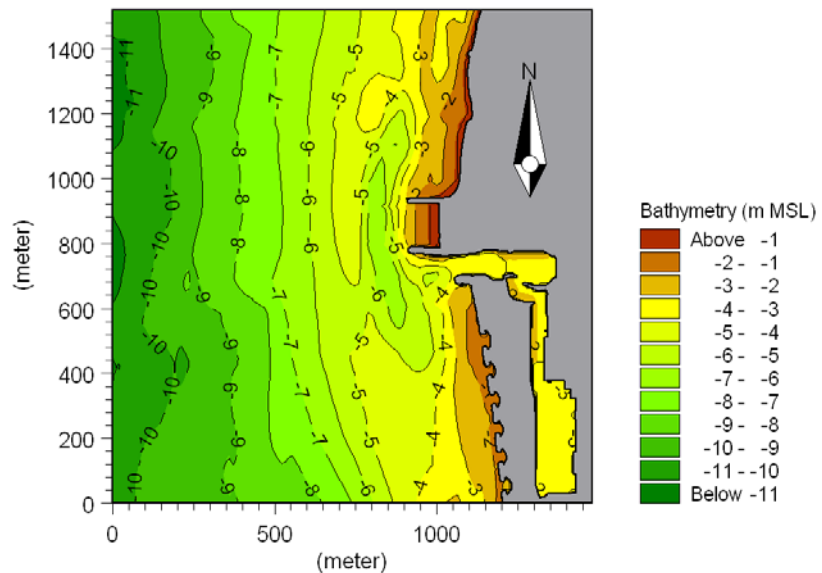Figure 4.16   Model bathymetry (= 2m) of Torsminde Harbour
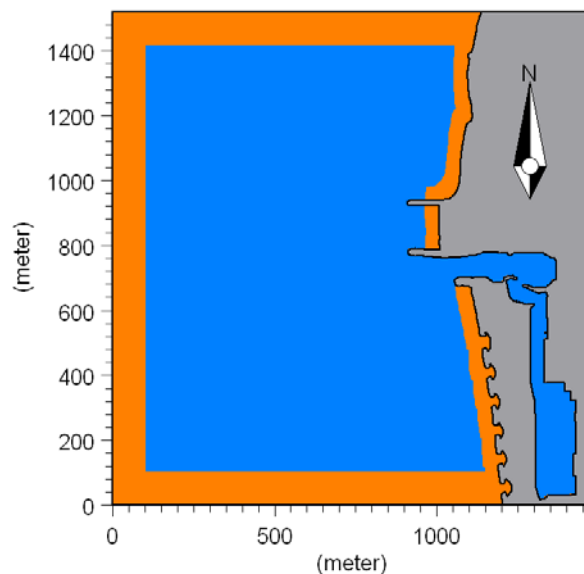

Figure 4.17   Sponge coefficient map

At the harbour entrance, along the entrance channel and at northern groin a porosity map is specified for simulation of wave dissipation in the rubble mound structures (see Figure 4.18). A porosity value of 0.85 is used. Also the porosity layer map is generated using the MIKE 21 Toolbox 'Generate

Sponge and Porosity Layer Maps' and algebraic manipulations in the Grid Editor.
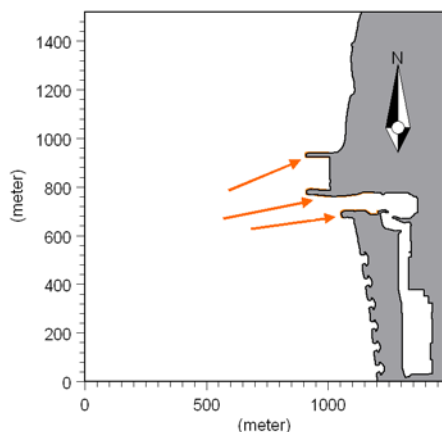

Figure 4.18   Porosity coefficient map

Model parameters

The deep-water terms are included in this example. It is recommended to always include the deep-water terms when wave breaking (and run-up) is modelled.

Figure 4.19 shows the numerical parameters used in this example. A similar set of parameters is recommended for most applications including wave breaking (and run-up). The use of simple upwinding at steep gradients and near land and a time extrapolation factor of slightly less than one (0.9 in this case) has successfully been used in a number of wave breaking applications.

Figure 4.20 shows the wave breaking parameters used in this application. The roller velocity type is set to 3 and the predominant wave direction is set to 270ºN. Hence we assume the breaking waves (rollers) mainly propagate in the positive x-direction, which is a good approximation in this case. The remaining parameters are set as default.

The time step is set to 0.1 s, see Figure 4.21 which shows a snapshot of the Java Script *MIKE 21 BW Model Setup Planner* included in the Online Help.

Please note that the wave breaking option "exclude wave breaking" is ticked in the MIKE 21 BW Model Setup Planner, see Figure 4.21. This requires an explanation. As stated in the Java Script Note 4 the most important issue is to make sure the breaking waves are well resolved at important areas in the model domain. If you use the wave breaking option, the minimum spatial resolution (dx) will be based on hmin and T_{min} ($L/dx > 20$) whereas the non-breaking option is based on $L/dx > 7$. If h_{min} is small in an area which is of less importance the planner provides a very small dx (and dt). Hence, the

most important issue is to resolved the breaking waves - and you can check this in the section "Check/evaluation of selected T_{min}, dx and dt".
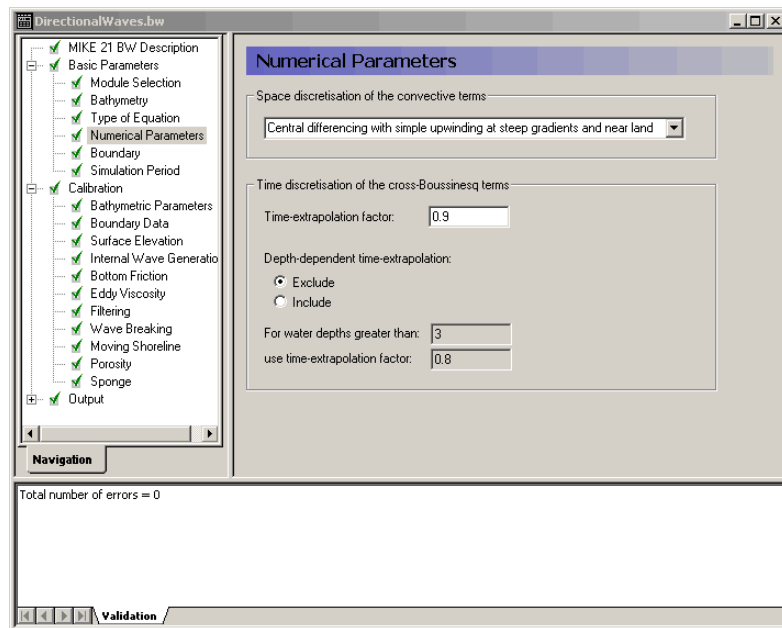
Figure 4.19   Numerical parameters
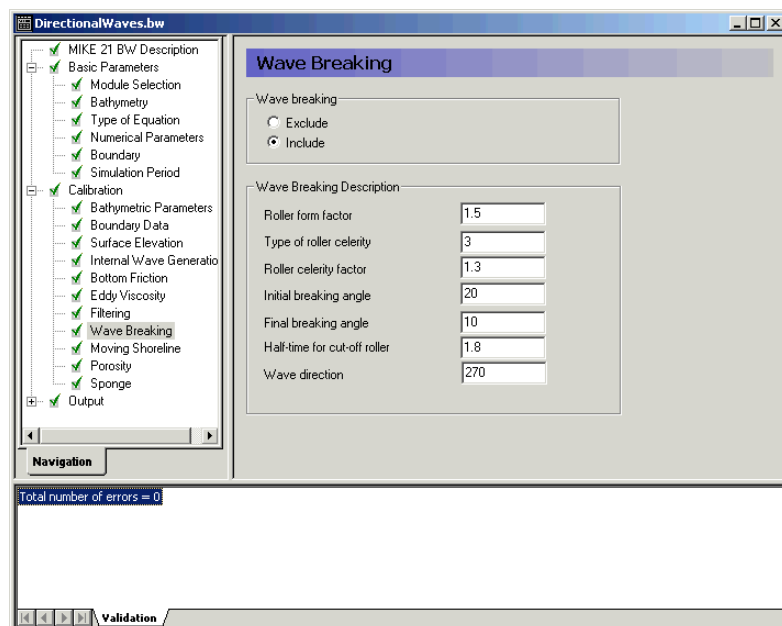
Figure 4.20   Wave breaking parameters

Figure 4.21   MIKE 21 BW Model Setup Planner included in the Online Help

Model run

Before you execute the MIKE 21 BW model simulation you have to provide internal wave boundary data. Simply drag and drop the Torsminde_Harbour.21t-file into your MIKE Zero shell, go to the 'waves' entry and click on run button, see Figure 4.22.
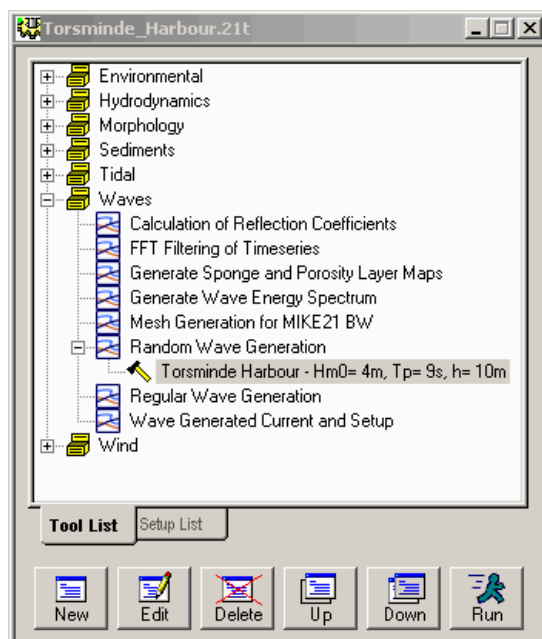
Figure 4.22   Generation of internal wave boundary data using the MIKE 21 Toolbox 'Random Wave Generation' tool

The length of the simulation is 30 minutes (corresponding to 18001 time steps). The computation time depends on the speed of your PC. For a laptop IBM T40 Pentium M (Centrino, 1.6 GHz and 1GB DDR RAM) the required system time is approximately 11 hours (the system CPS is approximately 200,000 points/second).

You may reduce the length of the simulation in order to reduce the overall CPU time.

Model results

Figure 4.23 shows a 2D visualisation of the simulated instantaneous surface elevation. You can make a similar plot by loading (drag and drop) the PFS-file MzPlot.Surfaceelevation.plc into the MIKE Zero shell during or after model execution. From the figure is seen that the intensity of the high-frequency waves is larger in the shallow water parts (say, depths less than 6 m) than at the offshore boundary as the quadratic nonlinear energy transfer is larger in shallow water than in deep water. Some of the generated high-frequency waves (say, wave periods less than 2-3 s) may not be properly resolved. However, they will affect the simulated long waves marginally.

Figure 4.24 shows a 3D visualisation of the simulated instantaneous surface elevation. The surface rollers are shown in white. You can make a similar plot by opening (by double-clicking) the PFS-file 'Anim Directional Waves with

Roller.lyt' in the MIKE Animator Plus program during or after model execution assuming you have a valid licence for MIKE Animator Plus.

From Figure 4.23 and Figure 4.24 is seen that short period wave agitation inside the harbour is very limited.

01/01/90 12:20:00
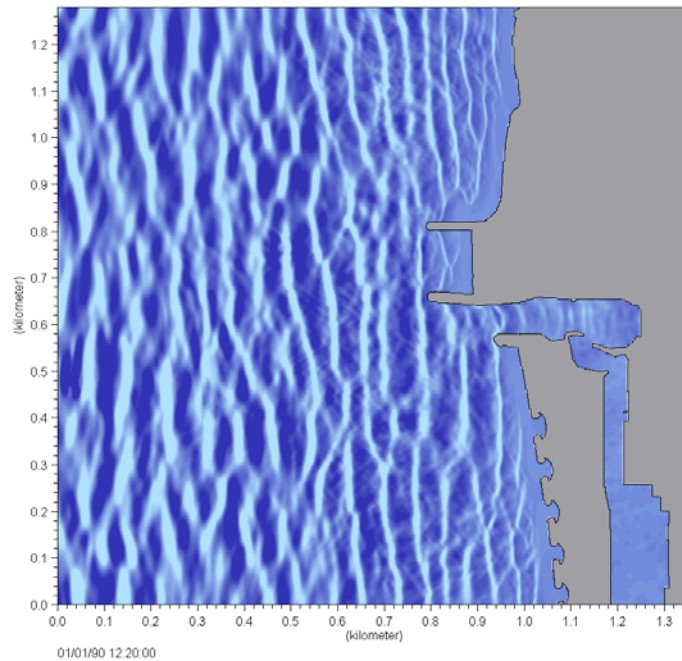
Figure 4.23   2D visualisation of instantaneous surface elevation
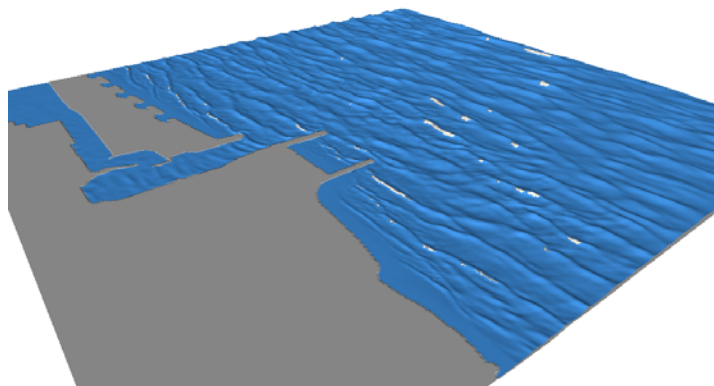
Figure 4.24   3D visualisation of instantaneous surface elevation and the surface rollers. The image (and animation) is made in MIKE Animator Plus

The spatial variation of the significant wave height and wave disturbance coefficient is presented in Figure 4.25. At the harbour entrance the wave height is reduced to approximately 2 m due to the wave breaking process.
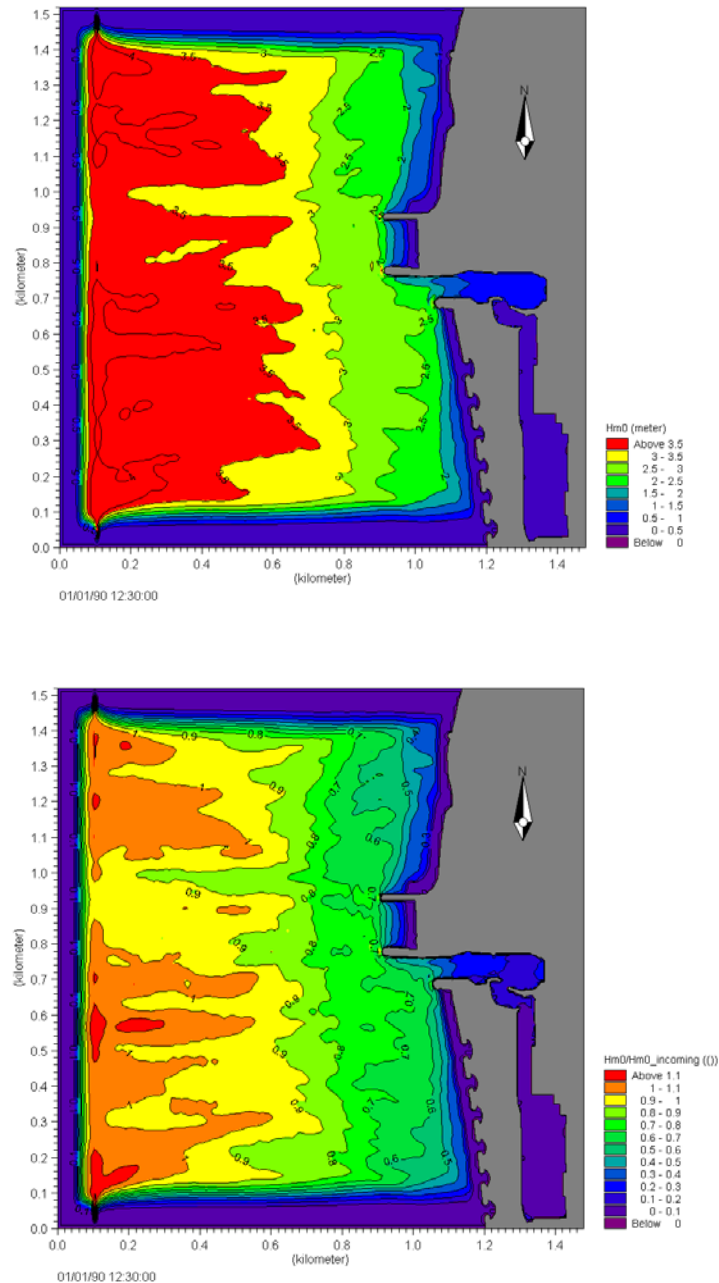
Figure 4.25   Map of wave height (upper) and wave disturbance coefficients (lower) simulated by MIKE 21 BW

The results from the phase-resolving MIKE 21 BW model has been compared to a phase-averaged model MIKE 21 NSW (default model parameters). The upper panel in Figure 4.26 shows a map of significant wave heights and wave directions. The lower panel in Figure 4.26 shows a comparison between the two models prediction of the significant wave height along the horizontal line indicated in Figure 4.27. In general the agreement is good. MIKE 21 BW predicts slightly higher waves along 600-900 m, which is mainly due to the non-linear shoaling. At the harbour entrance (1100 m) the two models predict similar significant wave height.
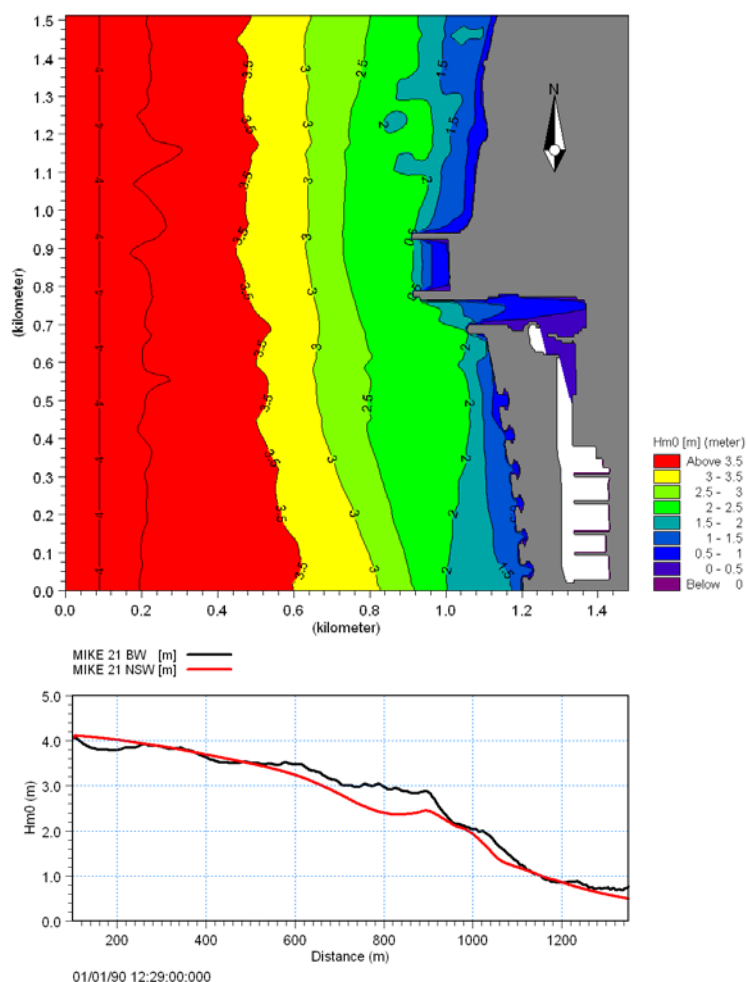
Figure 4.26   Map of significant wave height simulated by MIKE 21 NSW (upper) and a comparison between MIKE 21 BW and MIKE 21 NSW along the horizontal line depicted in Figure 4.27 (lower)

Figure 4.27 presents computed time series of water surface elevations at three different locations. For clarity the vertical scale is change for Point 3

(located in the harbour basin). At Point 3 the dominant wave agitation consists of long-period waves, which is even more clearly seen from the normalised spectra (lower panel). At Point 3 most of the wave energy is on wave periods within the range 30-60 s, which is in excellent agreement with field measurements. Please notice the substantial amount of high-frequency waves at Point 2 caused by nonlinear wave interactions among the primary wind waves (see also Figure 4.28).

.

Time series at Point 1 - offshore

Time series at Point 3 - sluice

Time series at Point 5 - basin
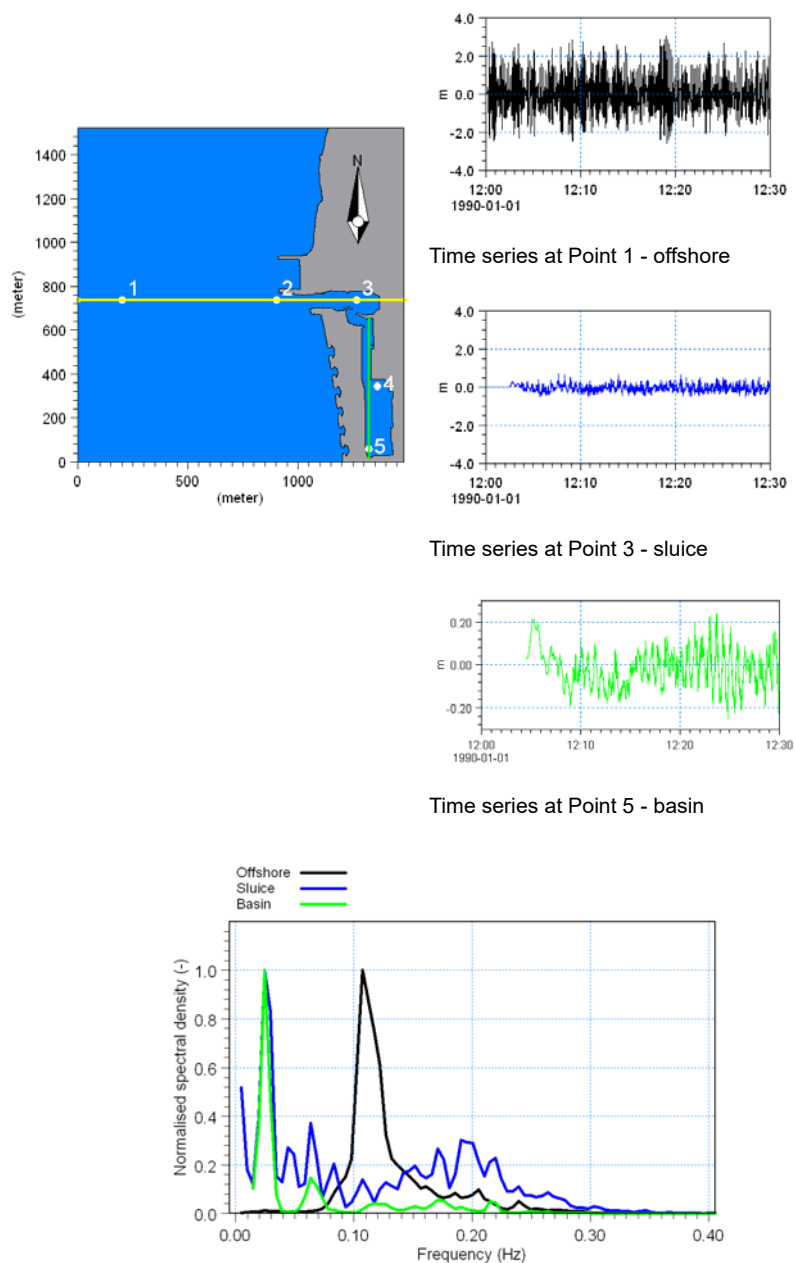
Figure 4.27   Time series of surface elevation at various points (see location map and the output section in DirectionalWaves.BW-file). The lower panel shows the corresponding normalised frequency spectra calculated using the MIKE Zero WSWAT Linear Spectral Analysis Module

Figures 4.28, 4.29 and 4.30 illustrate further spectral and digital analysis of the MIKE 21 BW model results, which requires a valid license for the WS Wave Analysis Tool included in MIKE Zero.
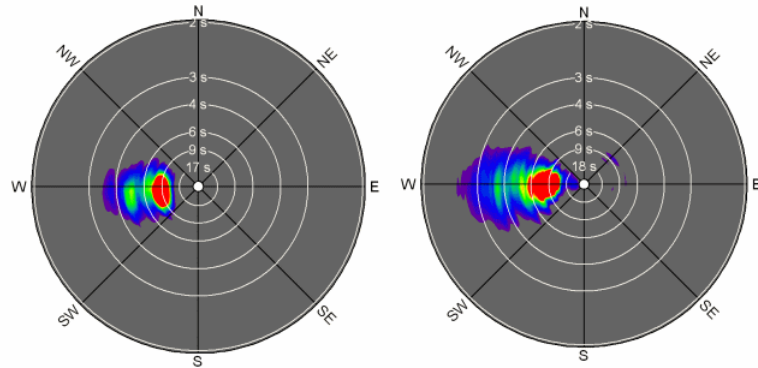
Figure 4.28   Calculated directional spectrum at the Point 1 and Point 2 (see Figure 4.27 for location). The spectrum is calculated using the MIKE Zero WSWAT Directional Wave Analysis Module
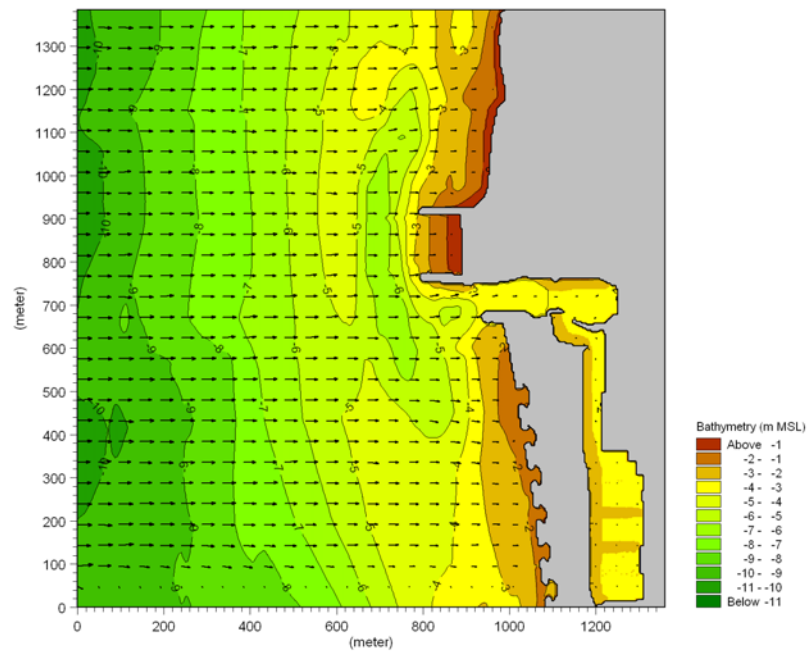
Figure 4.29   Calculated mean wave direction on top of the bathymetry. The spectrum is calculated using the MIKE Zero WSWAT Directional Wave Analysis Module

01/01/90 12:18:16

01/01/90 12:25:00:000
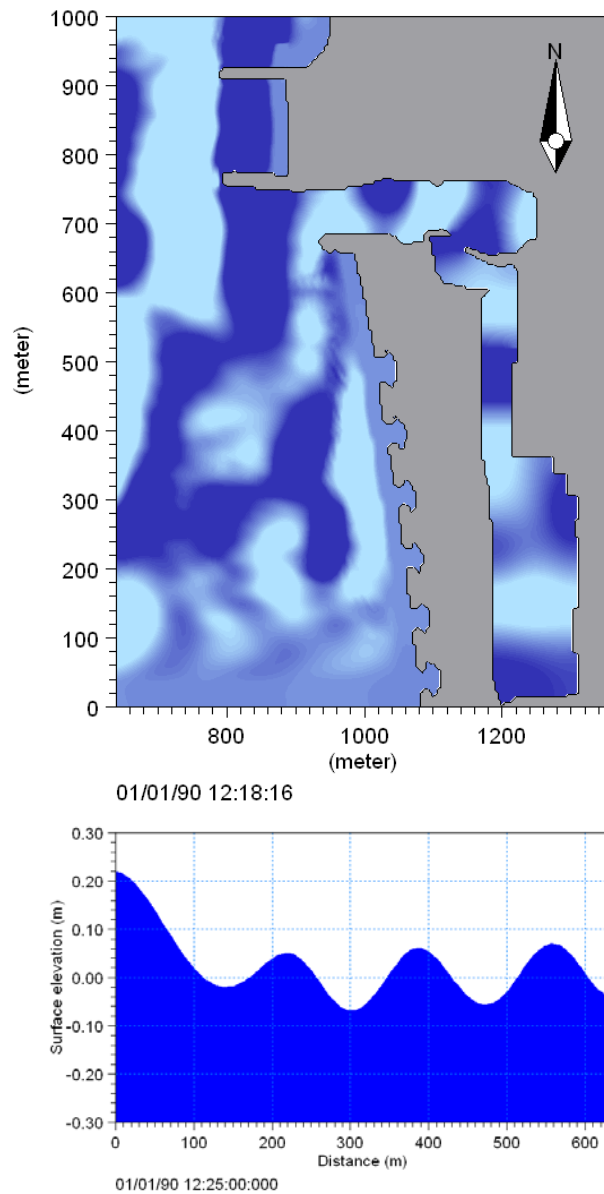
Figure 4.30   Calculated bandpass filtered (0.01-0.05 Hz) surface elevation. The pro-
file plot shows the long waves along the vertical line depicted in
Figure 4.27. The digital filtering is performed using the MIKE Zero
WSWAT Digital Filtering Analysis Module

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files

Name:	Bathymetry.dfs2
Description:	Bathymetry data

Name:	Sponge.dfs2
Description:	Map of sponge coefficients

Name:	Porosity.dfs2
Description:	Map of porosity coefficients

Parameter files

Name:	Tools Waves Generation Line.21t
Description:	PFS file generation of internal wave boundary data

Name:	DirectionalWaves.bw
Description:	PFS file for setup of MIKE 21 BW run

Name:	Plot Surface Elevation.plc
Description:	PFS for 2D visualisation of the simulated surface elevation using the MIKE Zero Plot Composer Grid Plot control

Name:	Plot Wave Disturbance.plc
Description:	PFS for 2D visualisation of the simulated wave disturbance using the MIKE Zero Plot Composer Grid Plot control

Name:	Anim Directional Waves with Roller.lyt
Description:	PFS for 3D visualisation of the simulated surface elevation using MIKE Animator Plus

4.2.6 Island

Purpose of the example

We will consider the transformation of regular unidirectional and irregular directional waves around an island with a moving shoreline, see Figure 4.31. The combined effect of most of the wave phenomena of interest in coastal engineering is included in this example; refraction, shoaling, diffraction, reflection, bottom friction, nonlinear wave-wave interaction, wave breaking and run-up.
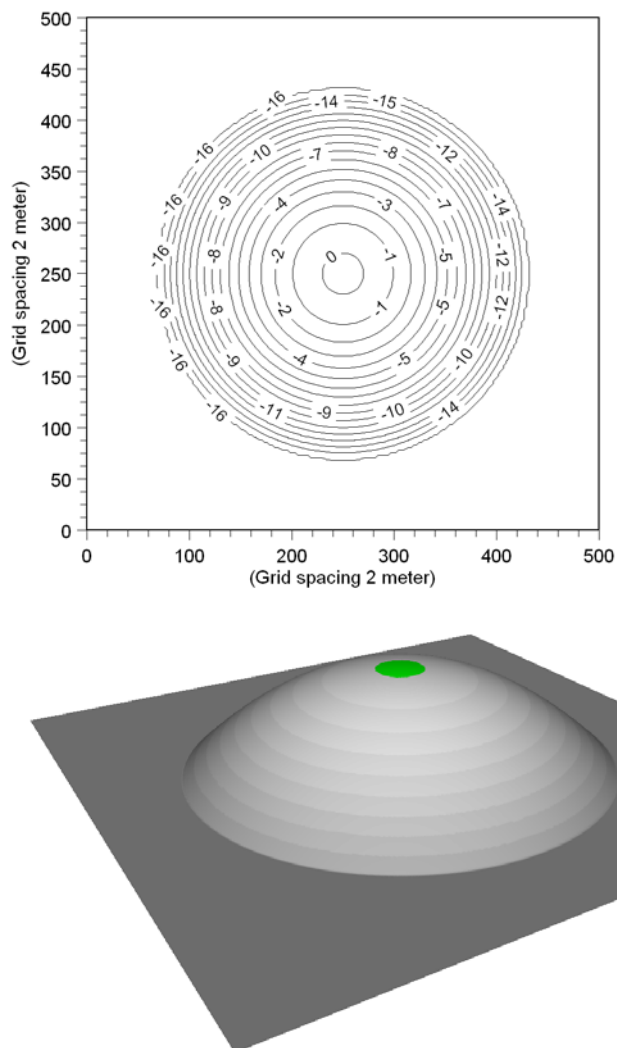
Figure 4.31   Model bathymetry of an island with an elliptic paraboloid form

We consider two different wave situations:

- Regular unidirectional waves (PFS file: RegularWaves.bw)
  H= 2 m, T= 8 s and MWD= 270 ºN

- Irregular directional waves (PFS file: DirectionalWaves.bw)
  H_{m0}= 2 m, T_p= 8 s, T_{min}= 4.5 s, MWD= 270 ºN, standard JONSWAP
  spectrum, Cos^8-directional spreading

Model setup

The size of model area is 1000 m x 1000 m. The bathymetry is an elliptic
paraboloid having an elevation of approximately +0.2 m (above the still water

level) at the top point and decreasing to a constant depth of approximately 16 m. The bathymetry is shown in Figure 4.31.

The spatial resolution is 2 m. The most energetic waves have a period around 8 s and are resolved by the 17-42 points per wave length at depths 2-16 m. As the primary part of the waves is expected to break where $H_{m0} \sim 0.5 \cdot h$ (experience rule), the wave breaking is initiated at approximately 4 m water depth. At this depth the most energetic waves are resolved by 24 grid points per wave length. In most practical 2DH applications including wave breaking and moving shoreline a grid spacing of 1-2 m is used for peak wave periods larger than, say, 7s.

As illustrated in Figure 4.32 a 50 points wide sponge layer is used for efficient wave absorption at the four model boundaries. The thickness of the sponge layer is corresponding to one wave length or more. The sponge layer map is generated by using the MIKE 21 Toolbox 'Generate Sponge and Porosity Layer Maps' and algebraic manipulations in the Grid Editor. Please note the sponge layer map used in the regular wave case is modified compared to the sponge layer map shown in Figure 4.32.
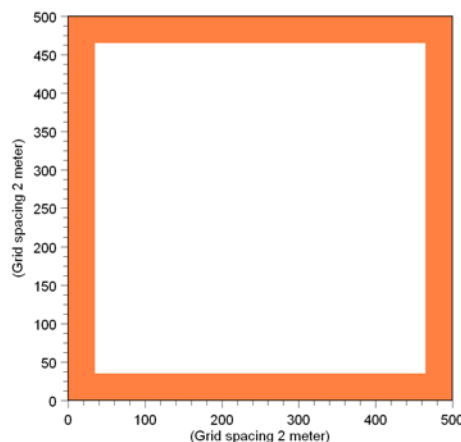
Figure 4.32   Sponge coefficient map (Directional.Sponge.dfs2)

A lowpass filter is included to remove high-frequency waves generated during uprush and downrush at the shoreline and to dissipate the wave energy in areas where the surface roller is not properly resolved. The filter is used at water depths less than approximately 0.5 m.

Model parameters

The deep-water terms are included in both wave situations. It is recommended to always include the deep-water terms when wave breaking and run-up is modelled.

Also in this example we use ''Simple upwinding at steep gradients and near land' for the space discretisation of the convective terms and a time extrapolation factor of slightly less than one (0.8 in this case). Similar settings are used for the Torsminde Harbour example presented in section 4.2.5.

The time step is set to 0.1 s for the regular wave simulation and 0.05 s for directional wave simulation. Due to the smaller wave periods (T_{min}= 4.5 s) considered in the directional wave case the time step is reduce accordingly.

Figure 4.33 shows the wave breaking and moving shoreline parameters used in this application. The roller velocity type is set to 1 as no predominate direction can be identified for the breaking waves. In the previous example (Torsminde Harbour) the type is set to 3 and the predominant wave direction is set to 270 ºN. The remaining parameters are set as recommended in the User Guide and Online Help.

Except for the slot depth, which is set to 16 m, default parameters are used for modelling of the moving shoreline.
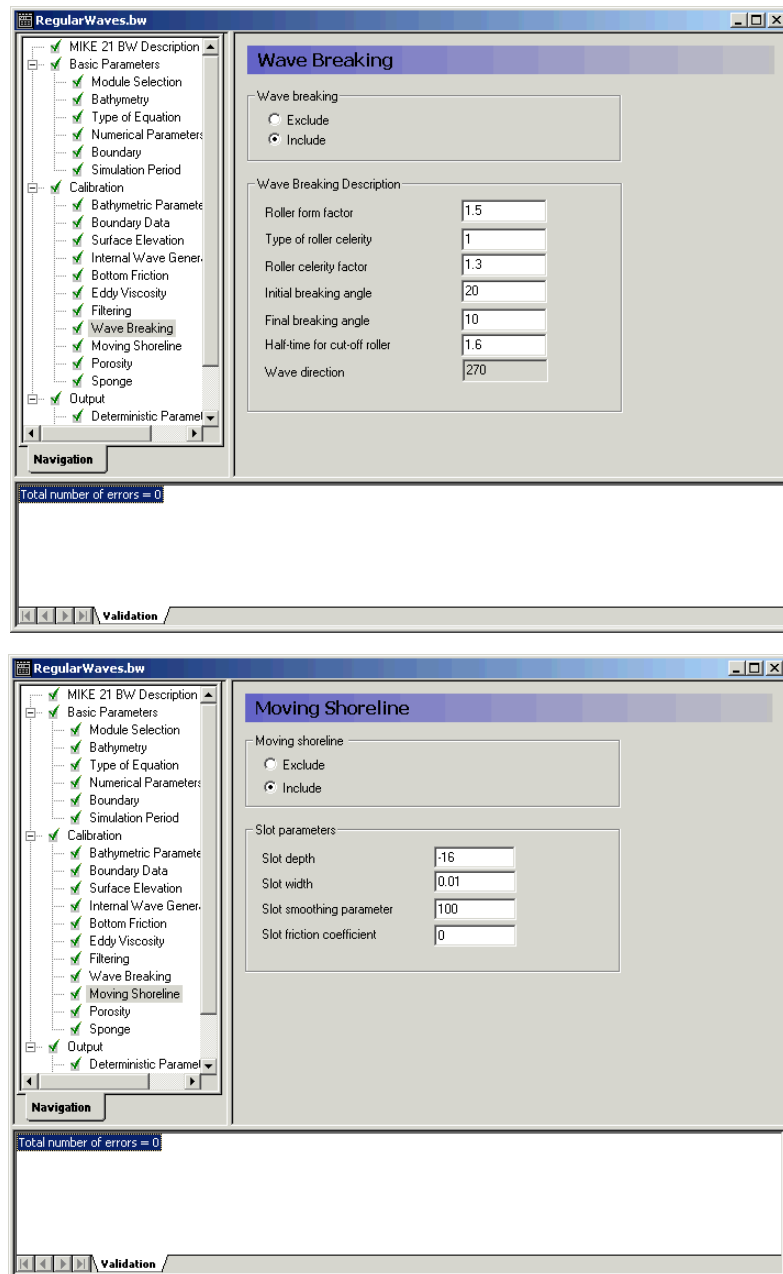
Figure 4.33   Wave breaking and moving shoreline parameters

Model run

Before you execute the MIKE 21 BW model simulations you have to provide internal wave boundary data. Simply drag and drop the file 'Wave_Genera-tion.21t' into your MIKE Zero shell, go to the 'waves' entry, select the 'Setup Name' and click on run button, see Figure 4.34.
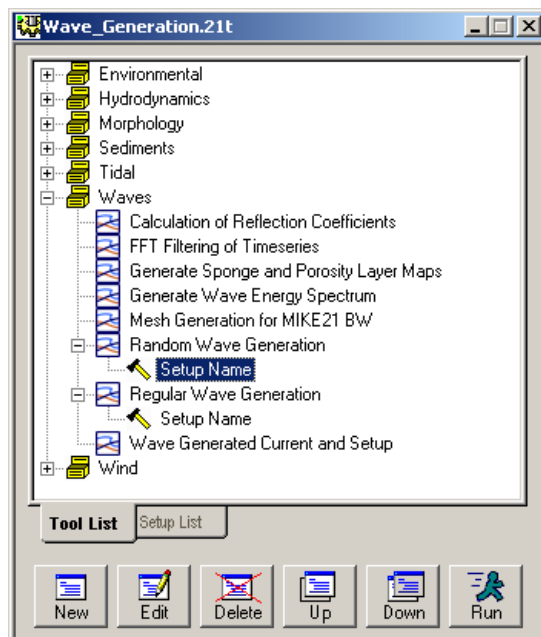
Figure 4.34   Generation of internal wave boundary data using the MIKE 21 Toolbox 'Random Wave Generation' and 'Regular Wave Generation' tool

The length of the simulation is 8 minutes and 20 seconds (corresponding to 5001 time steps) for the regular wave simulation and 10 minutes for the directional wave simulation.

The computation time depends on the speed of your PC. For a laptop IBM T40 Pentium M (Centrino, 1.6 GHz and 1GB DDR RAM) the required system time is approximately 2 hours (the system CPS is approximately 190,000 points/second) for the regular wave simulation and approximately 5 hours (the system CPS is approximately 180,000 points/second) for the directional wave simulation.

You may reduce the length of the simulations in order to reduce the overall CPU time.

Model results

Figure 4.35 shows 2D visualisation of the simulated instantaneous surface elevation for the two wave situations. The most visible wave phenomena are refraction, shoaling, nonlinear wave-wave interaction (generation of higher harmonic waves), wave breaking and run-up.

01/01/90 12:04:08

01/01/90 12:08:00
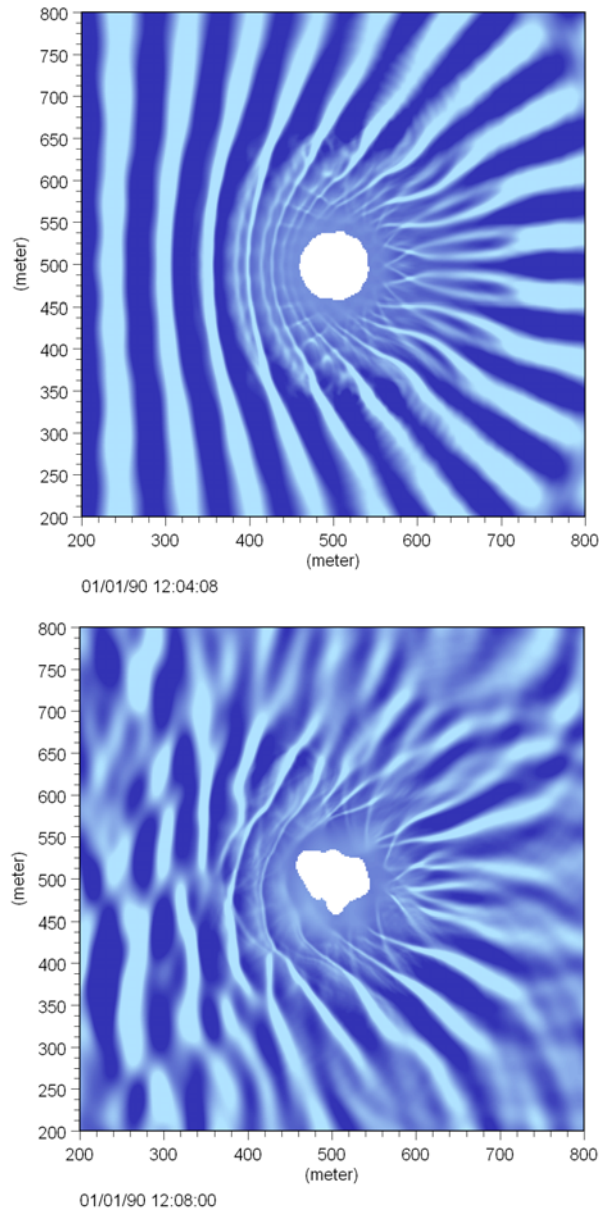
Figure 4.35   Visualisation (2D) of instantaneous surface elevation. Upper: regular waves and lower: directional waves

Figure 4.36 shows 3D visualisation of the simulated instantaneous surface elevation. The surface rollers are shown in white. You can make a similar plot by loading (double-click) the PFS-file 'Anim Regular Waves with Roller.lyt' (or 'Anim Directional Waves with Roller.lyt') into the MIKE Animator Plus program during or after model execution assuming you have a valid licence for MIKE Animator Plus.
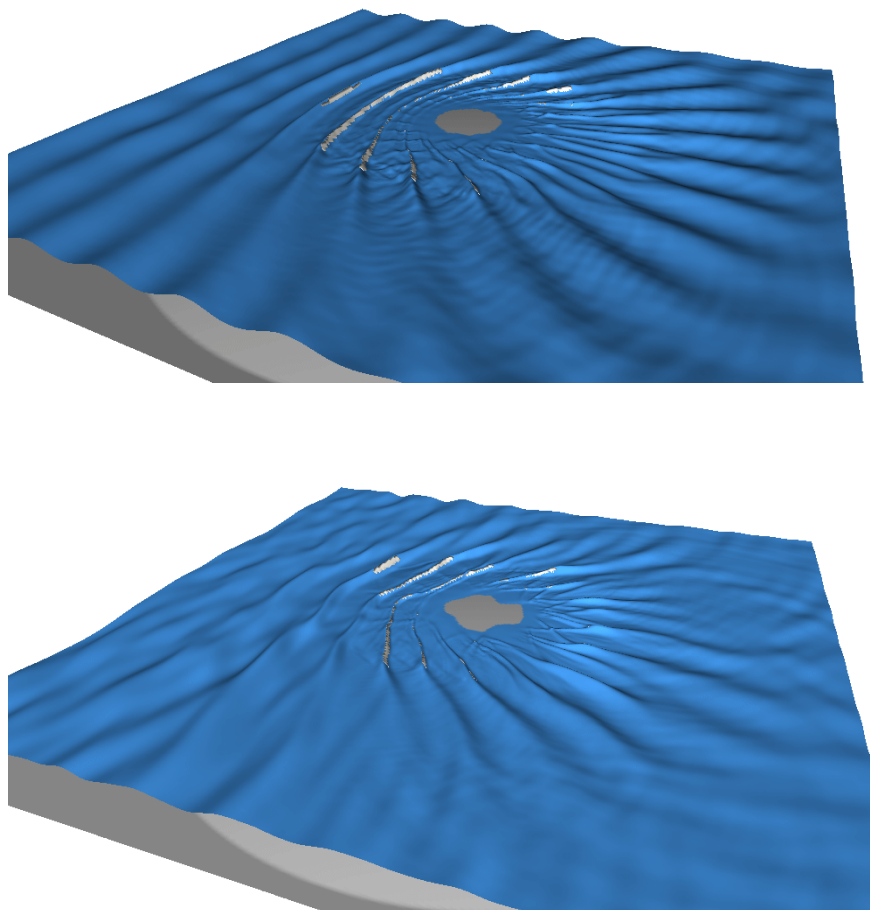
Figure 4.36   Visualisation (3D) of instantaneous surface elevation and the surface rollers. The image (and animation) is made in MIKE Animator Plus. Upper panel: regular waves, lower panel: directional waves

Time series of surface elevation and corresponding wave spectrum are presented in Figure 4.37 (regular waves) and Figure 4.38 (irregular waves). The spectra are calculated by use of the WSWAT Linear Spectral Wave Analysis Module. However, the FFT tool included in the MIKE 21 Toolbox may also be used for calculation of raw spectra.

Both figures clearly illustrate the effect of wave breaking and nonlinear wave-wave interaction (generation of higher harmonic waves).
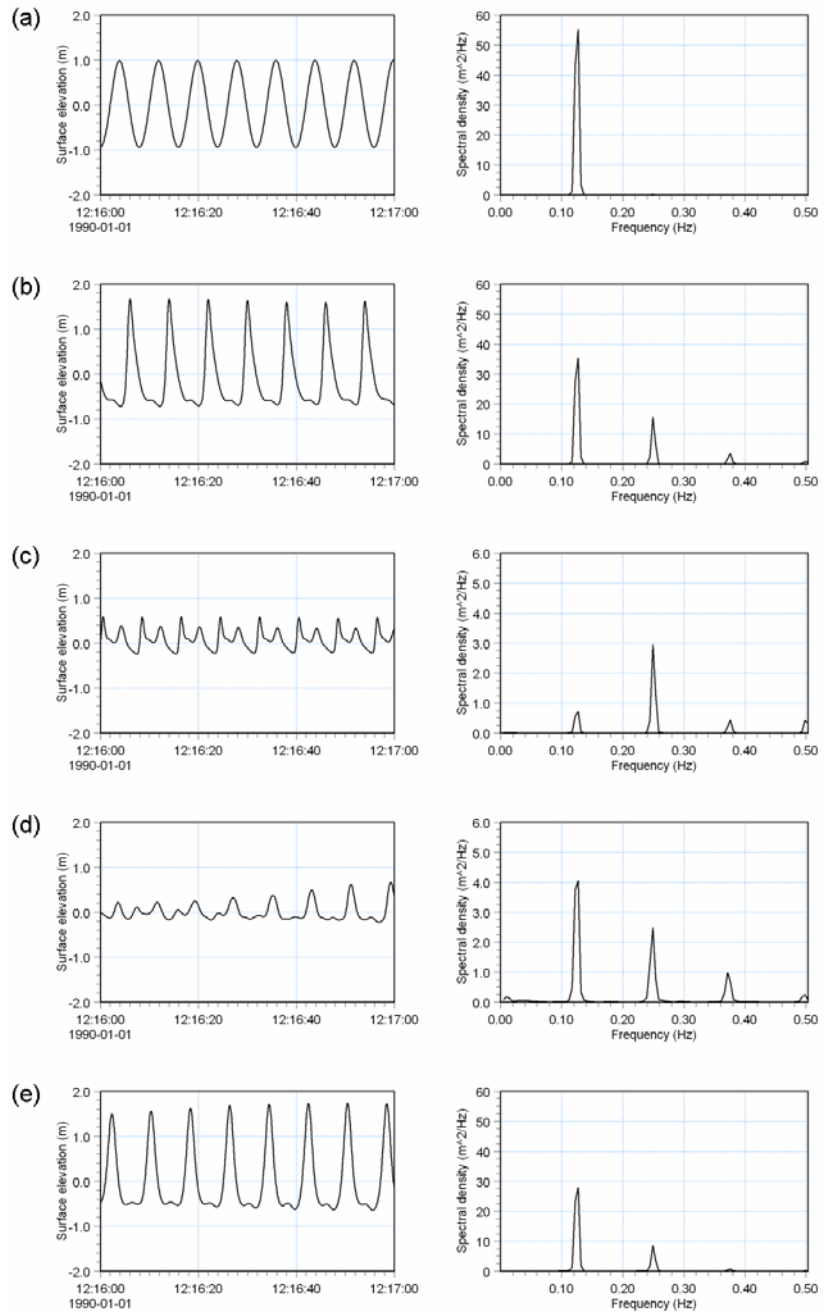
Figure 4.37   Time series of surface elevation (left) and corresponding wave spectrum (right). Regular unidirectional wave case. The data is extracted along the line y= 500 m at water depth (a) 16m, (b) 3m, (c) 1m (all upstream island) (d) 1m and (e) 3m (all downstream island)
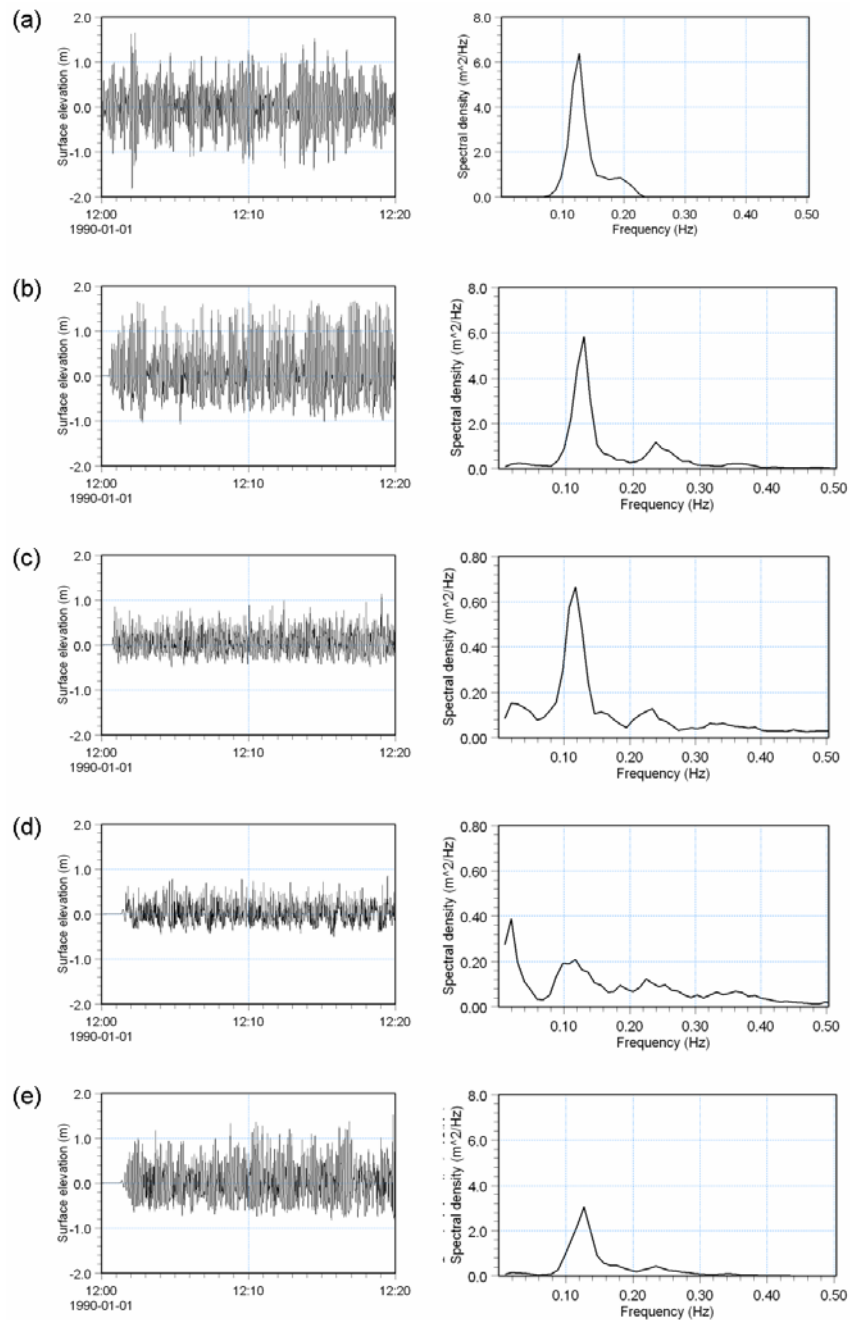
Figure 4.38   Time series of surface elevation (left) and corresponding wave spectrum (right). Irregular directional wave case. The data is extracted along the line y= 500 m at water depth (a) 16m, (b) 3m, (c) 1m (all upstream island) (d) 1m and (e) 3m (all downstream island)

The spatial variation of the significant wave height is presented in Figure 4.39. Please note wave statistics are calculated only in water points, which never dry out. The uprush/downrush area is shown by white colour in Figure 4.39 lower panel.
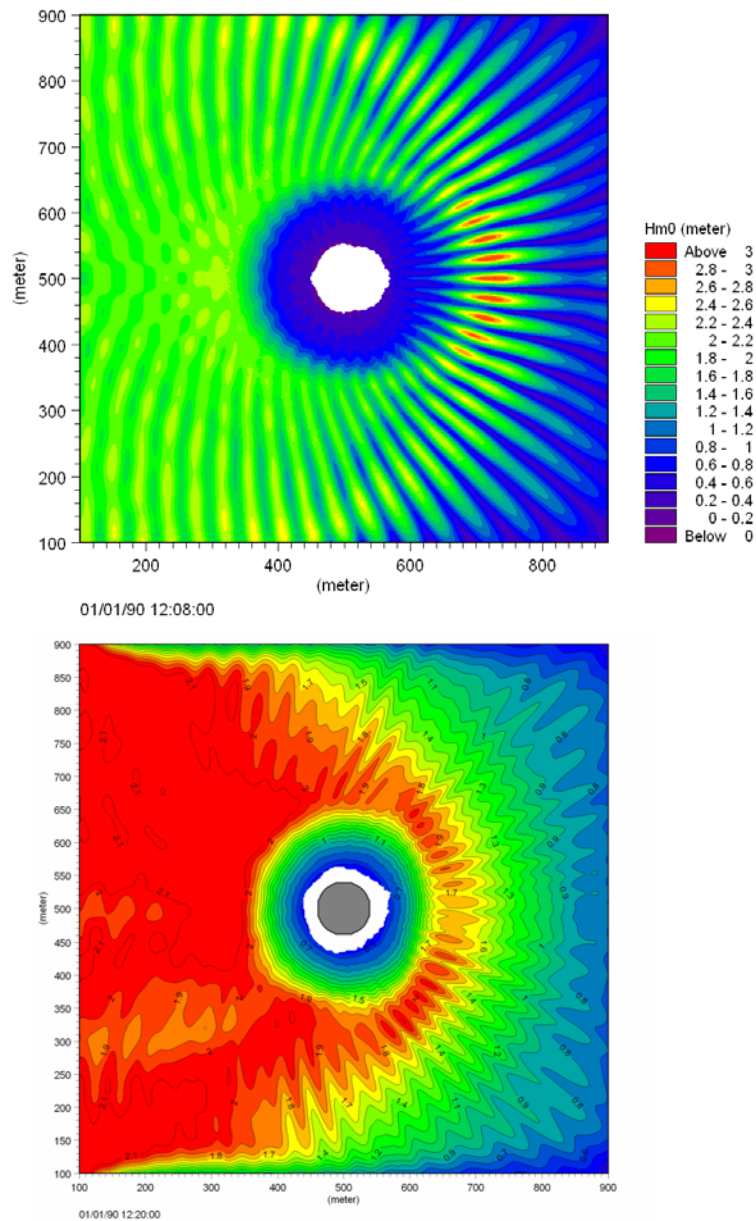
Figure 4.39   Map of wave heights. Upper: regular waves and lower: irregular waves. Please note the directional wave simulation was extended to 20 minutes simulation time

Results from a MIKE 21 SW simulation have been compared to results from the MIKE 21 BW irregular wave case. Figure 4.40 shows maps of significant wave heights and wave directions. In Figure 4.41 a comparison is made between the two models prediction of the significant wave height along two cross-sections. An excellent agreement is seen at in wave breaking zone (x= 300-500m, upper panel). MIKE 21 BW predicts slightly higher waves immediately before breaking which is due to nonlinear shoaling. The observed oscillations offshore the breaking point is caused by reflection from the island. Behind the island the phase-resolving model (MIKE 21 BW) results in larger waves than the phase-averaged model (MIKE 21 SW) which is caused by effects of diffraction, reflection and nonlinearities.
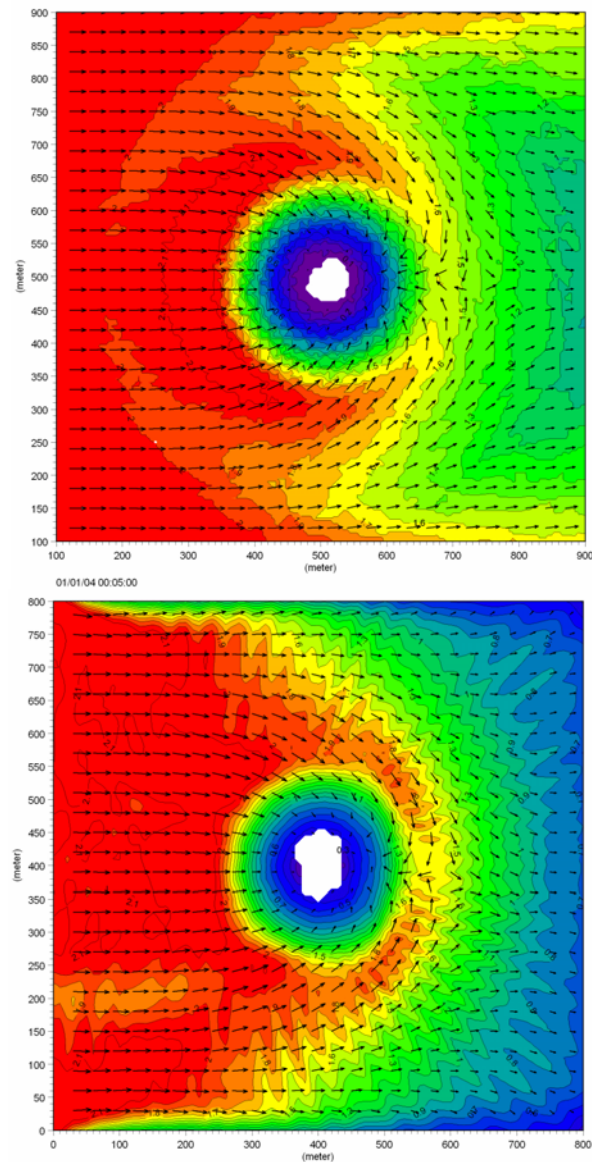
Figure 4.40   Maps of significant wave height and mean wave direction. Upper panel shows MIKE 21 SW results and lower panel shows results for the MIKE 21 BW irregular wave case. The directional analysis of the Boussinesq model results is made by using the WSWAT Directional Wave Analysis Module available in MIKE Zero

01/01/90 12:19:40:000

01/01/90 12:19:40:000
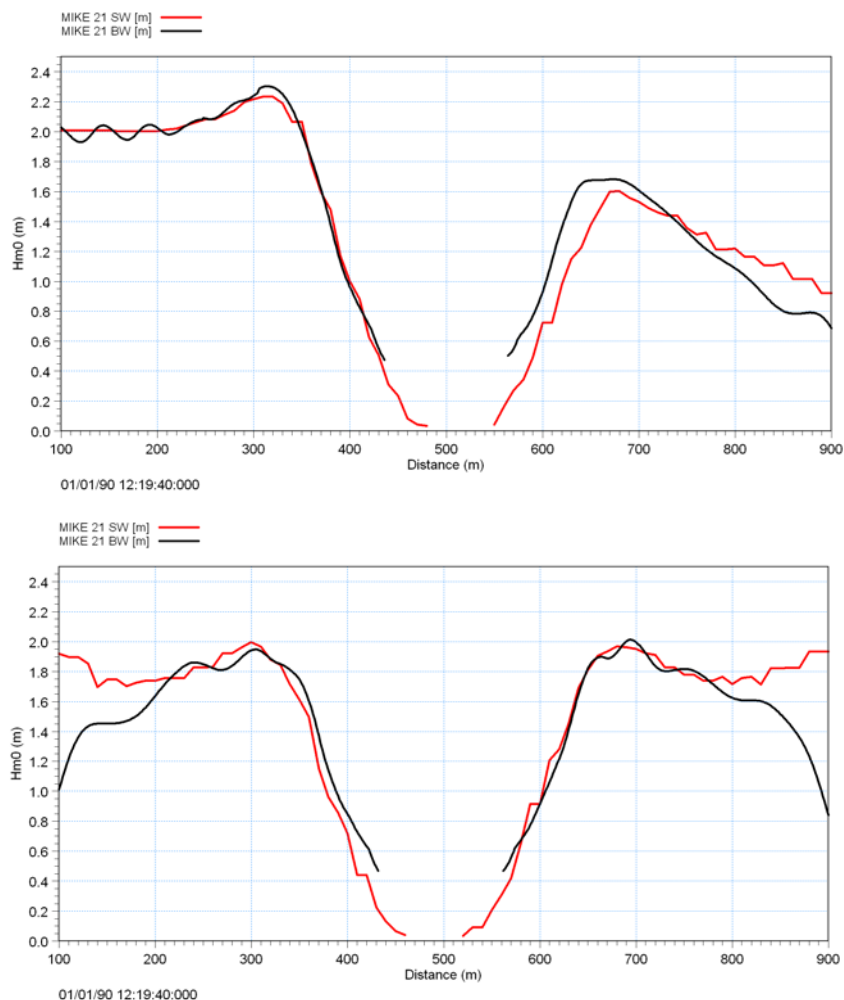
Figure 4.41   Comparison between modelled significant wave height using a phase-resolving model (MIKE 21 BW) and a phase-averaged model (MIKE 21 SW). Upper panel shows a comparison along y= 500 m (west-east direction) and lower panel shows a comparison along x= 500 m (south-north direction)

Vector plot of the nearly steady state wave-induced velocity field (after 20 minutes) is shown in Figure 4.42 for the two wave cases. For the MIKE 21 BW simulation the velocity is computed as the time-average of the depth-averaged velocity below the surface roller. As expected the current speeds are significant larger in case of regular waves. No data is available for qualitative and quantitative comparisons. Sections 4.2.8 (Detached breakwater) and 4.2.7 (Rip channel) deal with examples where experimental data is available.
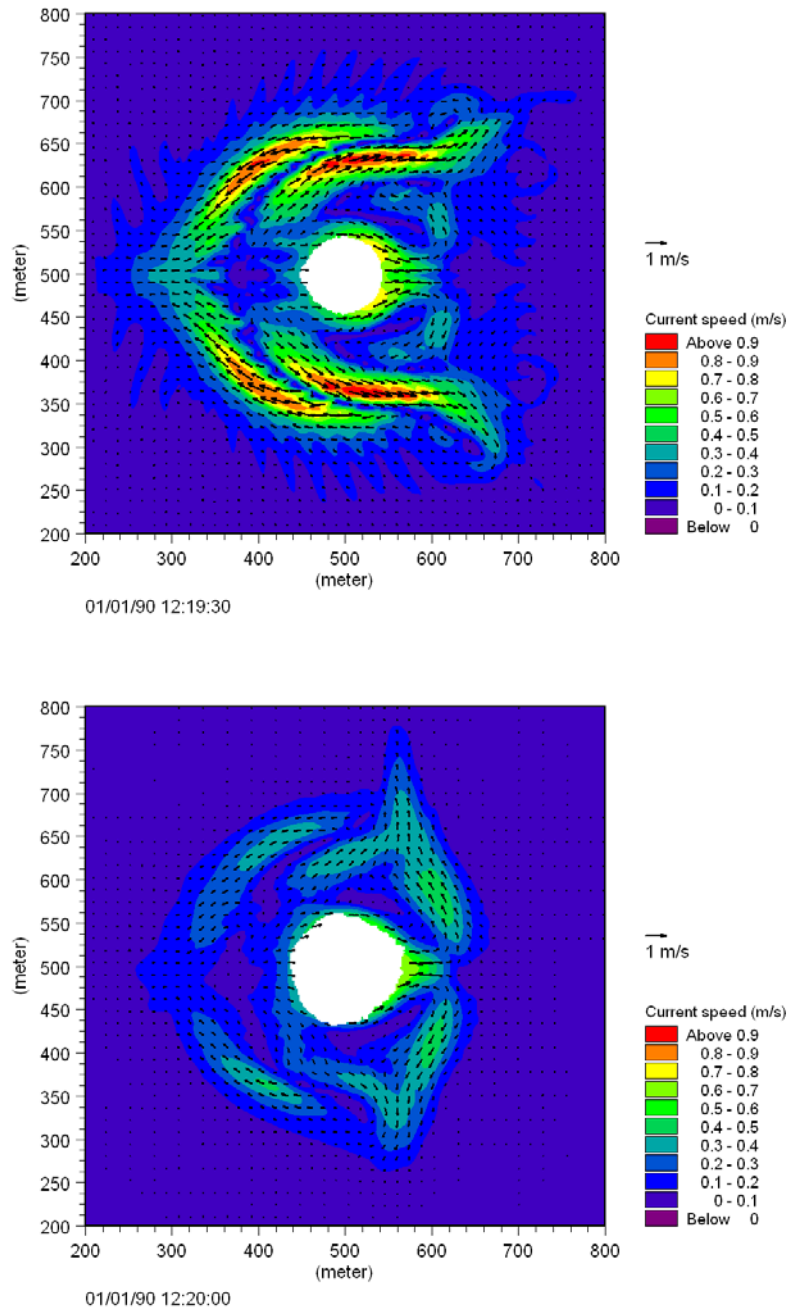
Figure 4.42 Wave induced current fields.
Upper panel: regular waves, lower panel: irregular waves

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files

Name: Bathymetry.dfs2
Description: Bathymetry data

Name: Sponge Directional Waves.dfs2
Description: Map of sponge coefficients (irregular directional waves)

Name: Sponge Regular Waves.dfs2
Description: Map of sponge coefficients (regular unidirectional waves)

Name: Filter.dfs2
Description: Map of lowpass filter coefficients

Parameter files

Name: Tool Waves Generation Lines.21t
Description: PFS file generation of internal wave boundary data

Name: Setup Regular Waves.bw
Description: PFS file for setup of MIKE 21 BW run
                    (regular unidirectional waves)

Name: Setup Directional Waves.bw
Description: PFS file for setup of MIKE 21 BW run
                    (irregular directional waves)

Name: Plot RegularWave Disturbance.plc
Description: PFS for 2D visualisation of the simulated wave disturbance
                    using the MIKE Zero Plot Composer Grid Plot control (regular
                    unidirectional waves)

Name: Plot Directional Wave Disturbance.plc
Description: PFS for 2D visualisation of the simulated wave disturbance
                    using the MIKE Zero Plot Composer Grid Plot control (irregular
                    directional waves)

Name: Anim Regular Waves with Roller.lyt
Description: PFS for 3D visualisation of the simulated surface elevation
                    using MIKE Animator Plus (regular unidirectional waves)

Name: Anim Directional Waves with Roller.lyt
Description: PFS for 3D visualisation of the simulated surface elevation
                    using MIKE Animator Plus (irregular directional waves)

4.2.7 Rip channel

Purpose of the example

The next two examples concentrate on wave breaking, run-up and wave-induced currents. This can be done without the traditional splitting of the phenomena into a wave problem and a current problem. In both examples we consider waves normally incident on a plane sloping beach, but with some alongshore non-uniformity to trigger nearshore circulation. In the first example, a rip channel is present while the second example (section 4.2.8) concerns a detached breakwater parallel to the shoreline.

Laboratory experiments are available for both examples as described in Sørensen et al (1998)[1]. This paper includes detailed comparisons between Boussinesq model results and laboratory measurements. The two examples included in the DHI Software installation are very similar to the cases considered in Sørensen et al (1998). However, the examples here are considered as prototypes of the laboratory experiments by use of a Froude scaling of 1:40.

Model setup

The first example is chosen according to laboratory experiments reported by Hamm (1992a, b), see reference in Sørensen et al (1998). The size of model area is 1200 m x 1200 m and the bathymetry is shown in Figure 4.43. It consists of a plane sloping beach of 1:30 with a rip channel excavated along the centre line. The maximum water depth is 20 m at the offshore boundary.

We will consider regular, unidirectional incident waves with a period of 7.9 s (1.25 s in the laboratory) and a wave height of 2.8 m (0.07 m in the laboratory) at the offshore boundary. Wave direction is 270 ºN.

The spatial resolution is 2 m. As the primary part of the waves is expected to break where Hmax ~ 0.8·h (experience rule for regular waves), the wave breaking is initiated at 3.5-4 m water depth. At this depth the waves are resolved by approximately 25 grid points per wave length.

As illustrated in Figure 4.44 a 50 points wide sponge layer is used for wave absorption at the offshore boundary and behind the slot used to modelling the run-up process. The thickness of the sponge layer is corresponding to one wave length or more. No sponge layer is used along the lateral boundaries in order to avoid wave radiation into the sponge layer.

---

1   A link to this paper is available from the MIKE 21 Documentation Index
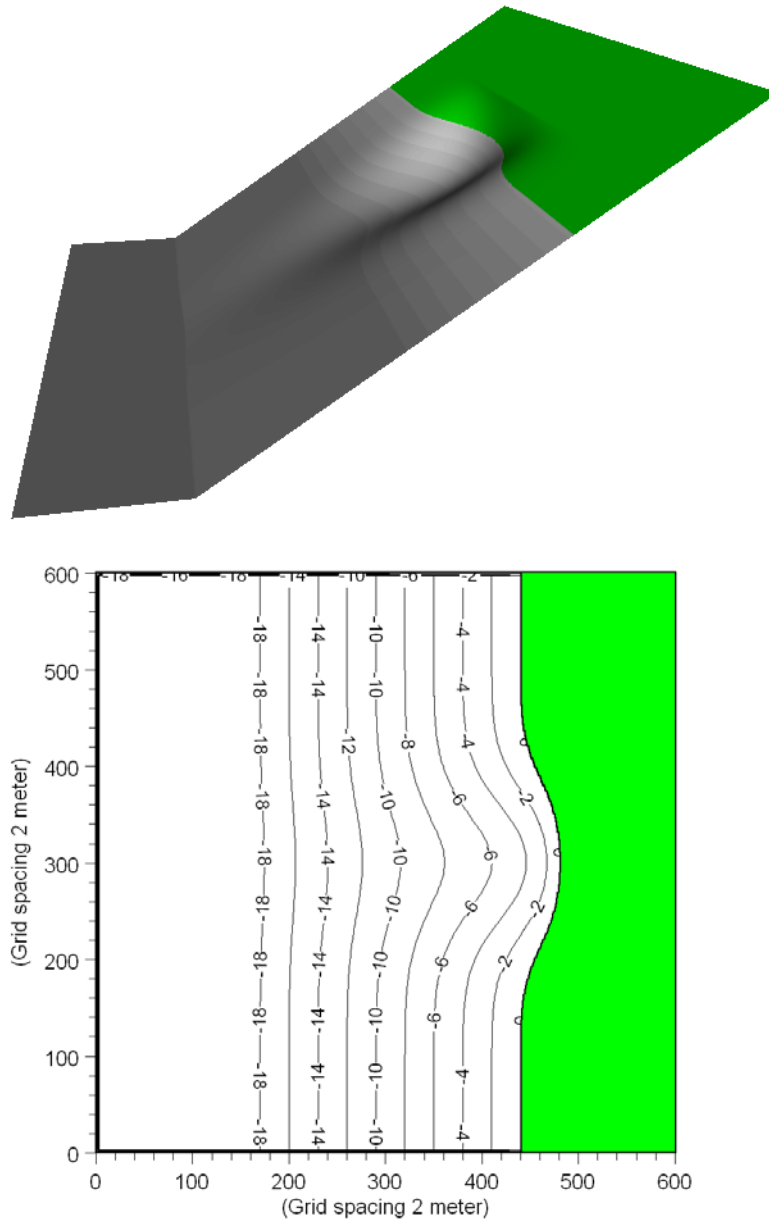
Figure 4.43   Model bathymetry for rip channel example

A lowpass filter is included to remove high-frequency waves generated during uprush and downrush at the shoreline and to dissipate the wave energy in areas where the surface roller is not properly resolved. The filter is used at water depths less than approximately 0.5 m, see Figure 4.44.
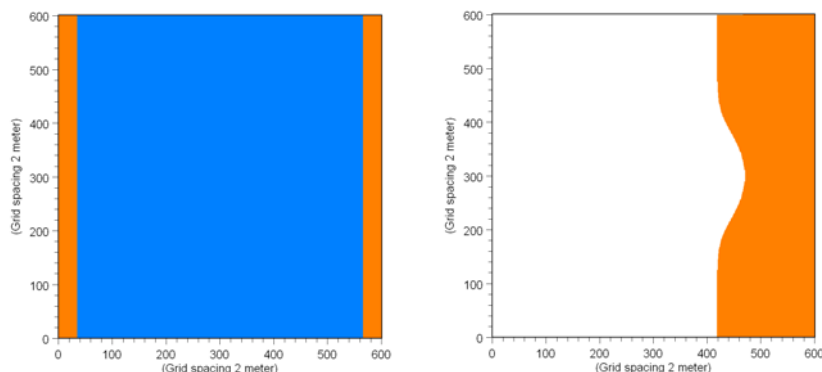
Figure 4.44   Map of absorbing sponge layers (left) and map of low-pass filter coefficients (right)

A constant Chezy number of C= 35 m$^{1/2}$/s is used for the bottom friction dissipation. This corresponds approximately to a constant wave friction factor of f_m= 0.03 used in Sørensen et al (1998).

As opposed to the shore-normal case steady solutions for the mean flow can only exists when the forcing by radiation stress is balanced by bottom friction and mixing processes. Mixing processes are usually modelled by means of an eddy viscosity. In the breaking zone, the viscous shear is to some extent take into account by the additional convective terms due to the surface roller. Outside the breaking zone, the eddy viscosity is usually very small. Hence no eddy viscosity is included in the simulation.

Model parameters

As in the other examples considering wave breaking the deep-water terms are included.

Also in this example we use ˝Simple upwinding at steep gradients and near land' for the space discretisation of the convective terms. The time extrapolation factor is set to zero, which will not cause numerical damping of waves as they propagate in the x-direction. If directional waves are considered a non-zero value close to one should be used.

The time step is set to 0.1 as in the previous two examples.

The roller velocity type is set to 3 and the predominant wave direction is set to 270 ºN. Hence we assume the breaking waves (rollers) will mainly propagate in the positive x-direction, which is a good approximation in this example. The remaining parameters are set as recommended in the User Guide and Online Help. The time scale for the roller is set to T/5= 1.58 s.

Except for the slot depth, which is set to 12 m and the slot width is set to 0.05 default parameters are used for modelling of the moving shoreline. The slot width is fairly large compared to recommendations (0.001-0.01), which will

results in a smaller run-up than if a value within the recommended interval was used. The large value was choosing for numerical stability reasons and will affect the wave-induced current pattern marginally.

Model run

Before you execute the MIKE 21 BW model simulations you have to provide internal wave boundary data. Simply drag and drop the file 'Rip_Channel.21t' into your MIKE Zero shell, go to the 'waves' entry, select the 'Rip channel - H= 2.8m, T= 7.9s, h= 20m and click on run button.

The length of the simulation is 20 minutes (corresponding to 12001 time steps or approximately 150 wave periods).

The computation time depends on the speed of your PC. For a laptop IBM T40 Pentium M (Centrino, 1.6 GHz and 1GB DDR RAM) the required system time is approximately 7 hours (the system CPS is approximately 180,000 points/second).

You may reduce the length of the simulations in order to reduce the overall CPU time.

Model results

Figure 4.45 shows a 2D visualisation of the simulated instantaneous surface elevation. The most visible wave phenomena are refraction, shoaling, nonlinear wave-wave interaction (generation of higher harmonic waves).

Figure 4.46 shows 3D visualisations of the simulated instantaneous surface elevation. The surface rollers are shown in white. You can make a similar plot by loading (double-click) the PFS-file 'Anim Regular Waves with Roller.lyt' into the MIKE Animator Plus program during or after model execution assuming you have a valid licence for MIKE Animator Plus.

01/01/90 12:04:00
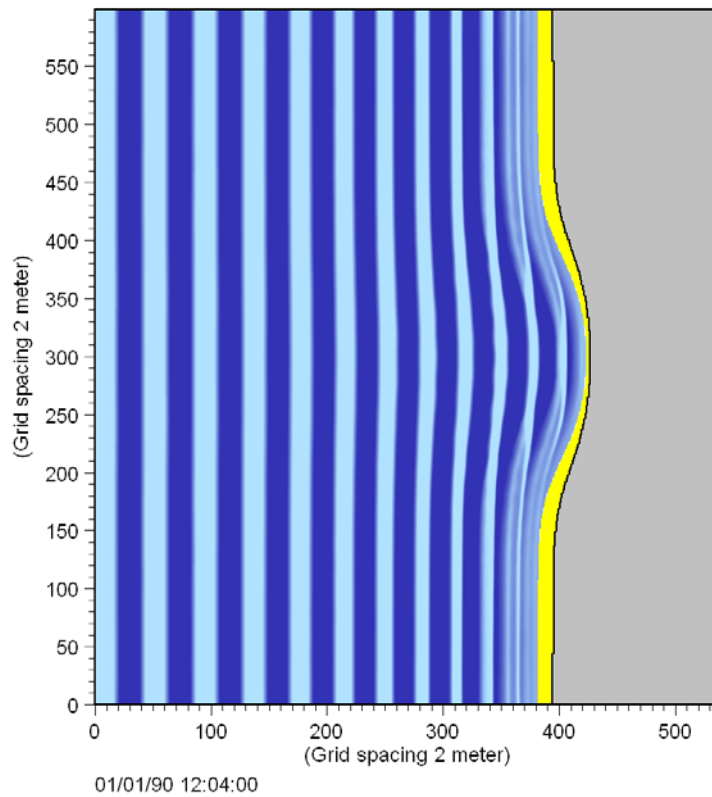
Figure 4.45   Visualisation (2D) of instantaneous surface elevation
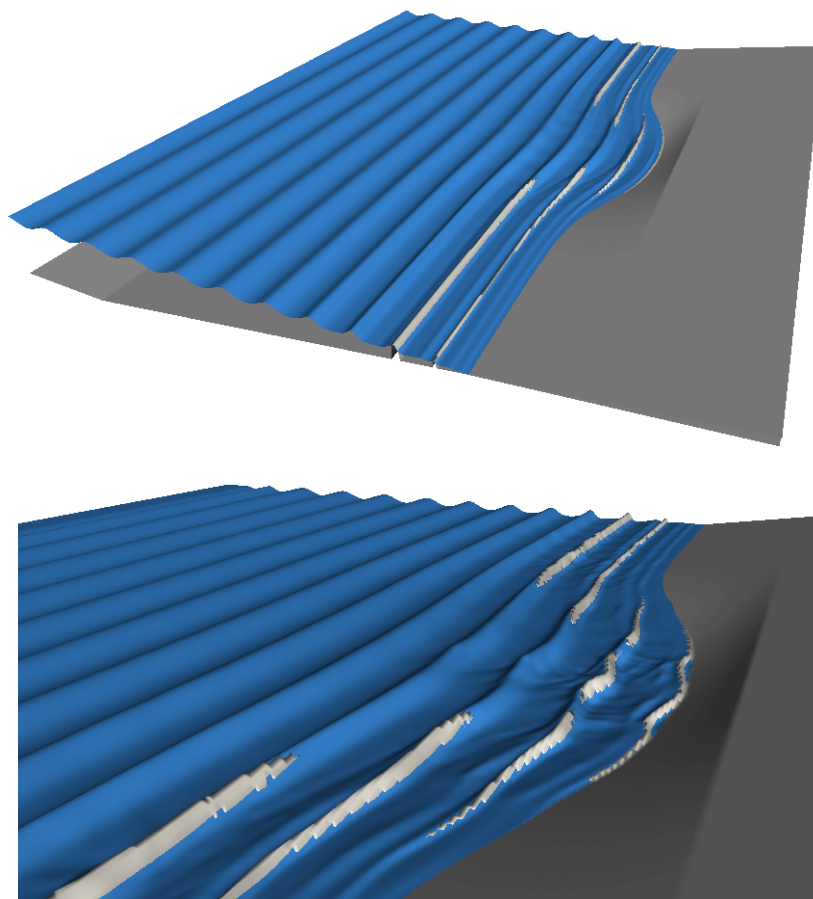
Figure 4.46   Visualisation (3D) of instantaneous surface elevation and the surface rollers. The image (and animation) is created in MIKE Animator Plus

Due to the increased depth and due to depth refraction by the rip channel, incipient wave breaking is seen to occur comparatively close to the shore along the centre line. Here, the wave setup is quite small and the larger setup appearing away from the rip channel gives an alongshore gradient in the mean water surface forcing a current towards the centre line. The flow from the two sides join to form a rip current and two symmetrical circulation cells appear.

A combined vector and contour plot of the nearly steady state wave-induced velocity field is shown Figure 4.47. The velocity is computed as the time-average of the depth-averaged velocity below the surface roller. A pronounced rip current is seen along the centre line of the bathymetry (y= 300 grid points).

Figure 4.48 shows the cross-shore variation of the rip current. A maximum of approximately 1 m/s for the mean depth-averaged return current is seen, which is in good agreement with the experiments reported by Hamm (1992b), see Sørensen et al (1998).
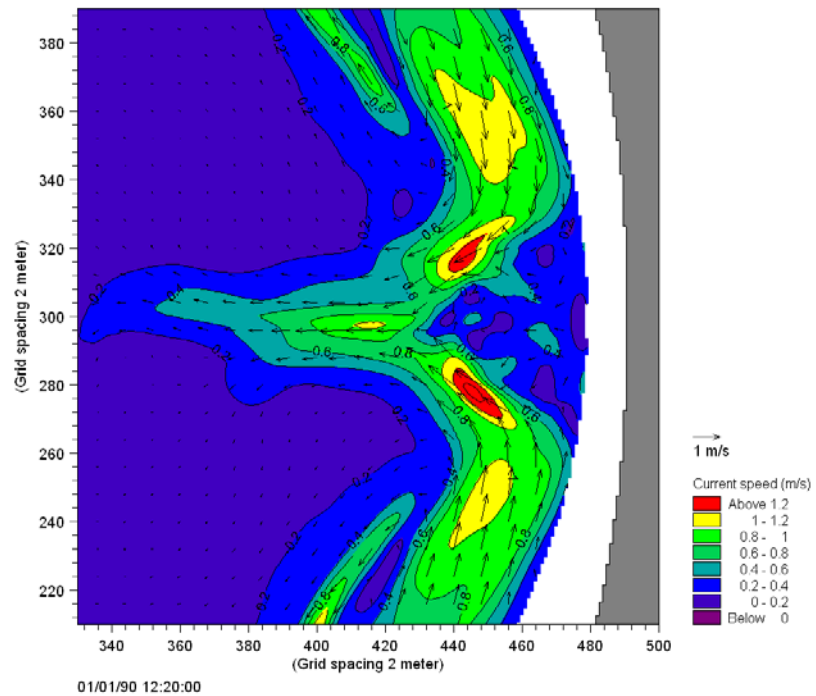
Figure 4.47   Depth-averaged wave-induced current focusing on the symmetrical circulation cell
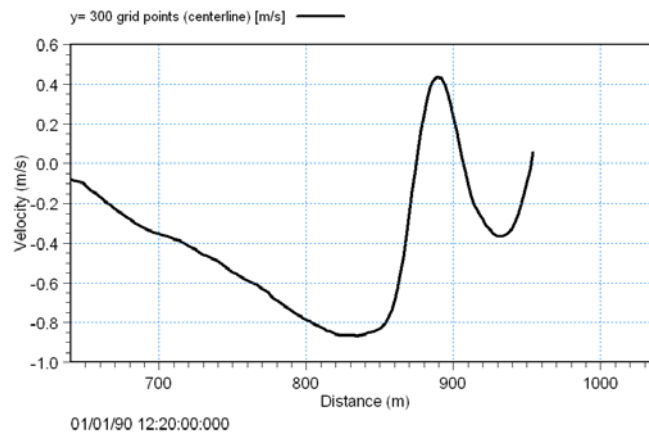
Figure 4.48   Rip current along the rip channel (y= 300 grid points)

The rip current significantly affects the wave motion. The large variation of the rip current causes an increase in the wave height, which can be seen in Figure 4.46 (lower panel). The rip current also causes a small local bend in the wave crest occurring at the centre line as observed by Hamm.

The spatial variation of the relative wave height is presented in Figure 4.49. Please note wave statistics are calculated only in water points, which never dry out. The uprush/downrush area is own by white colour in Figure 4.47 and Figure 4.49.
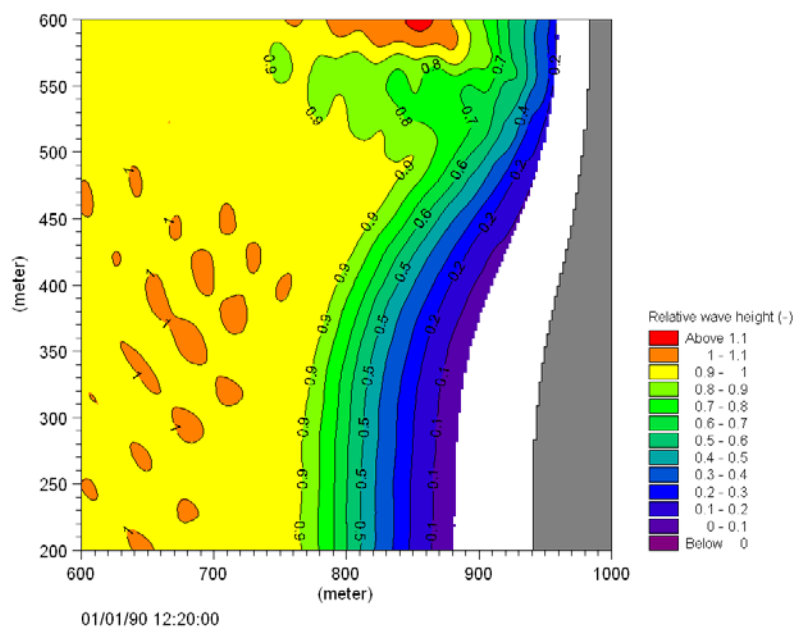
Figure 4.49   Map of relative wave height. Please note only the lower half of the computational domain is shown
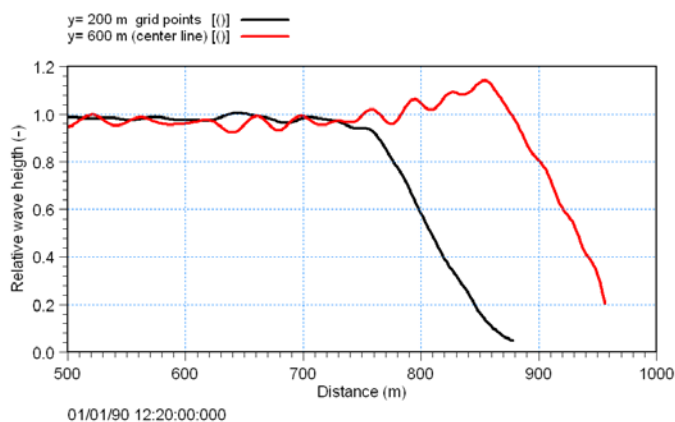
Figure 4.50   Variation of relative wave height along rip channel (y= 600 m) and outside rip channel (y= 200 m)

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files

Name: Bathymetry.dfs2
Description: Bathymetry data

Name: Sponge.dfs2
Description: Map of sponge coefficients

Name: Filter.dfs2
Description: Map of lowpass filter coefficients

Parameter files

Name: Tool Wave Generation line.21t
Description: PFS file generation of internal wave boundary data

Name: Setup Regular Waves.bw
Description: PFS file for setup of MIKE 21 BW run

Name: Anim Regular Waves with Roller.lyt
Description: PFS for 3D visualisation of the simulated surface elevation using MIKE Animator Plus

4.2.8 Detached breakwater

Purpose of the example

In this example we shall consider wave transformation and wave-induced currents around a detached breakwater parallel to the shoreline.

Comprehensive hydrodynamic measurements around a detached breakwater on a plane beach have recently been reported by Mory and Hamm (1997) and Hamm et al (1995), see references in Sørensen et al (1998)[1]. This latter paper also includes detailed comparisons between Boussinesq model results and laboratory measurements.

The present example is considered as a prototype of the laboratory experiments cited above by use of a Froude scaling of 1:40.

Model setup

The size of model area is 1200 m x 1200 m and the bathymetry is shown in Figure 4.51. It consists of three sections: a 176 m wide horizontal section with a water depth of 13.2 m, a plane sloping beach of 1:50 between the horizon-

---

1 A link to this paper is available from the MIKE 21 Documentation Index

tal section and the shoreline, and an emerged plan beach with a slope of 1:20. A half detached breakwater approximately 266 m long and 35 m wide is placed parallel to the shoreline along a vertical wall. The onshore side of the breakwater is located approximately 373 m from the shoreline. An absorbing beach is placed on the offshore side of the breakwater and along the vertical wall opposite to the breakwater.

We will consider two different wave situations:

- Regular unidirectional waves (PFS file: RegularWaves.bw)
  H= 3.2 m, T= 10.7 s and MWD= 270 ºN (normal incidence)

- Irregular directional waves (PFS file: DirectionalWaves.bw)
  H_{m0}= 4.6 m, T_p= 10.7 s, T_{min}= 4 s, MWD= 270 ºN, standard JONSWAP spectrum, cos2-directional spreading (normal incidence)

The spatial resolution is set to 2 m. As the primary part of the waves is expected to break where Hmax ~ 0.8·h (experience rule for regular waves), the wave breaking is initiated at 4 m water depth in case of regular waves. The directional waves will start to break where H_{m0} ~ 0.5·h, i.e. at water depth 9 m. This means the initial breaking waves are resolved by more than 30 grid points per wave length.
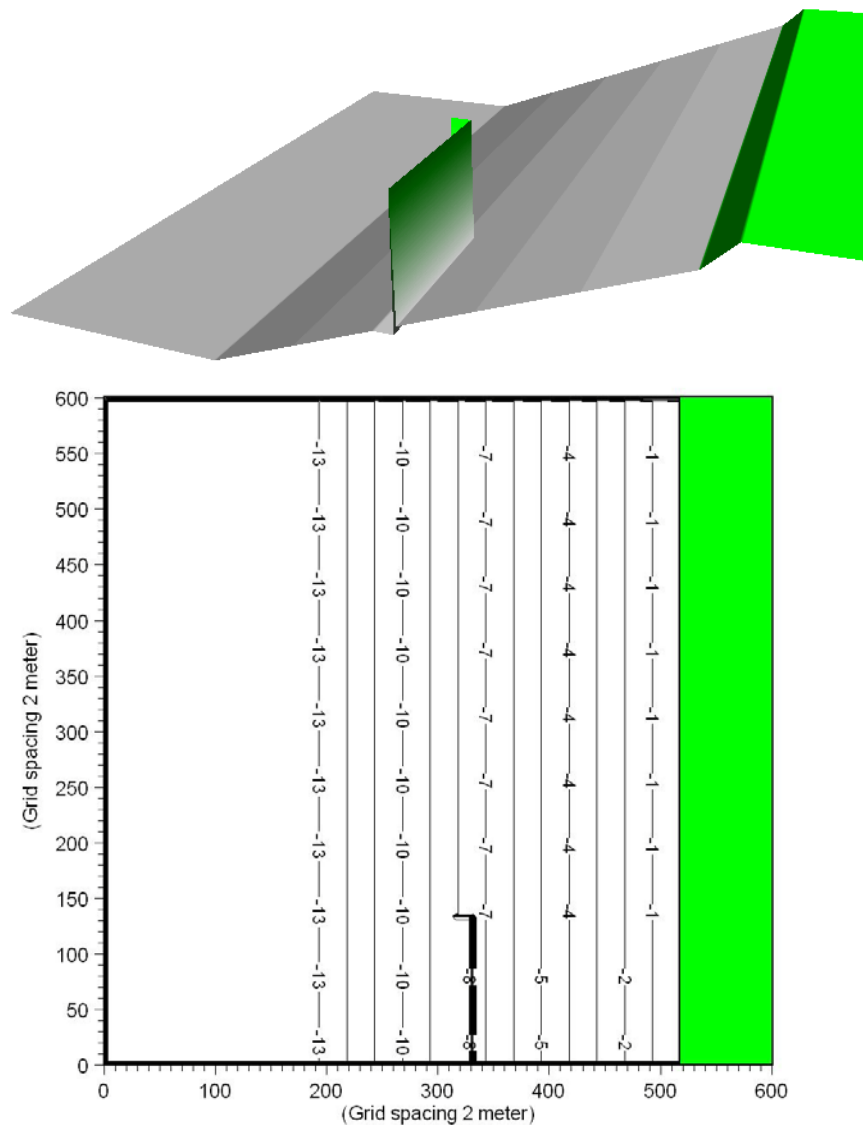
Figure 4.51   Model bathymetry for the detached breakwater example

In the numerical simulations, the physical wave maker is replaced by an internal line of generation, and re-reflection from the boundary is avoided by using a 50 points wide sponge layer offshore from the generation line, see Figure 4.52. The absorbing beach on the offshore side of the detached breakwater is modelled using a sponge layer. The thickness of the sponge layer is corresponding to one wave length or more. Only in case of directional waves a sponge layer is used along the lateral boundaries.

A lowpass filter is included to remove high-frequency waves generated during uprush and downrush at the shoreline and to dissipate the wave energy in

areas where the surface roller is not properly resolved. The filter is used at water depths less than approximately 0.7 m, see Figure 4.53.
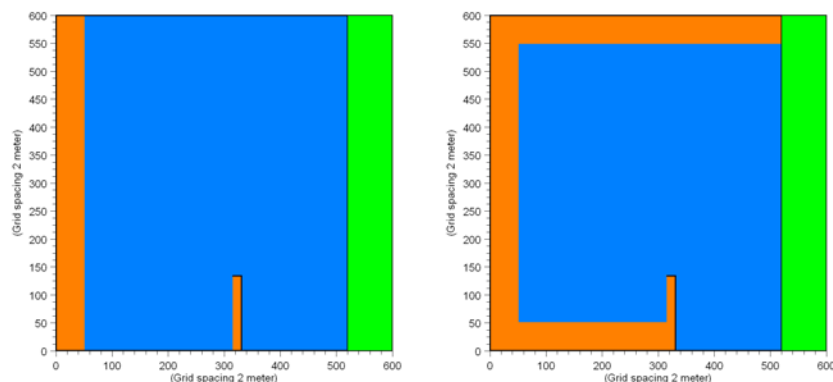
Figure 4.52   Map of absorbing sponge layers for regular wave simulation (left) and directional wave simulation (right)
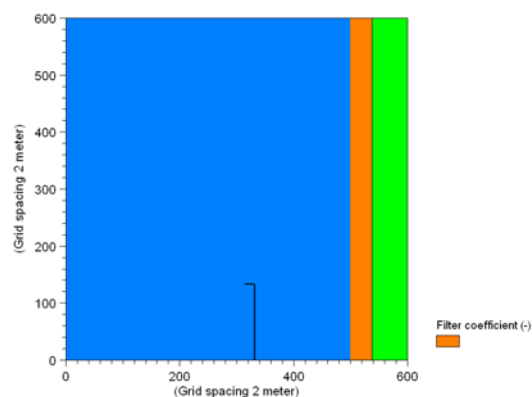
Figure 4.53   Map of low-pass filter coefficients used for both type of waves

A constant Manning number of M= 35 $m^{1/3}$/s is used for the bottom friction dissipation.

Model parameters

As in the other examples considering wave breaking the deep-water terms are included.

In this example we use 'Simple upwinding differencing' for the space discretisation of the convective terms. The time extrapolation factor is set to a value of slightly less than one (0.8 in this case). As discussed in the User Guide we recommend you to use 'Simple upwinding at steep gradients and near land' for the space discretisation of the convective terms. However. for numerical

stability reasons it is sometimes necessary to use the slightly more dissipative simple upwind scheme. As long as the waves are properly resolved the dissipation is usually small.

The time step is set to 0.14 s.

The roller velocity type is set to 3 and the predominant wave direction is set to 270 ºN. Hence we assume the breaking waves (rollers) will mainly propagate in the positive x-direction, which is a good approximation in this example. The remaining parameters are set as recommended in the User Guide and Online help including the time scale for the roller which is set to T/5= 2.14 s.

Except for the slot depth, which is set to 13.2 m and the slot friction coefficient is 0.01 default parameters are used for modelling of the moving shoreline.

Model run

Before you execute the MIKE 21 BW model simulations you have to provide internal wave boundary data. Simply drag and drop the file 'Detached_breakwater.21t' into your MIKE Zero shell, go to the 'waves' entry, select the 'Detached breakwater - H= 3.2m, T= 10.7s, h= 13.2m' (for regular wave generation) and 'Detached breakwater - Hm0= 4.6m, Tp= 10.7s, h= 13.2m' (for random wave generation) and click on run button.

The length of the simulation is 42 minutes (corresponding to 18001 time steps or approximately 235 (peak) wave periods).

The computation time depends on the speed of your PC. For a laptop IBM T40 Pentium M (Centrino, 1.6 GHz and 1GB DDR RAM) the required system time is approximately 13 hours (the system CPS is approximately 130,000 points/second).

You may reduce the length of the simulations in order to reduce the overall CPU time.

Model results

Figure 4.54 shows 2D visualisation of the simulated instantaneous surface elevation for the two wave situations. The most visible wave phenomena are refraction, shoaling, diffraction, nonlinear wave-wave interaction (generation of higher harmonic waves), wave breaking and run-up. You can make a similar plot by loading (drag and drop) the PFS-file 'MzPlot.Elevation.Regular.plc' or 'MzPlot.Elevation.Directional.plc' into the MIKE Zero shell during or after model execution.

Figure 4.55 shows 3D visualisations of the simulated instantaneous surface elevation. The surface rollers are shown in white. You can make a similar plot by loading (double-click) the PFS-file 'Anim Regular Waves with Roller.lyt' (or 'Anim Directional Waves with Roller.lyt') into the MIKE Animator Plus program

during or after model execution assuming you have a licence for MIKE Animator Plus.
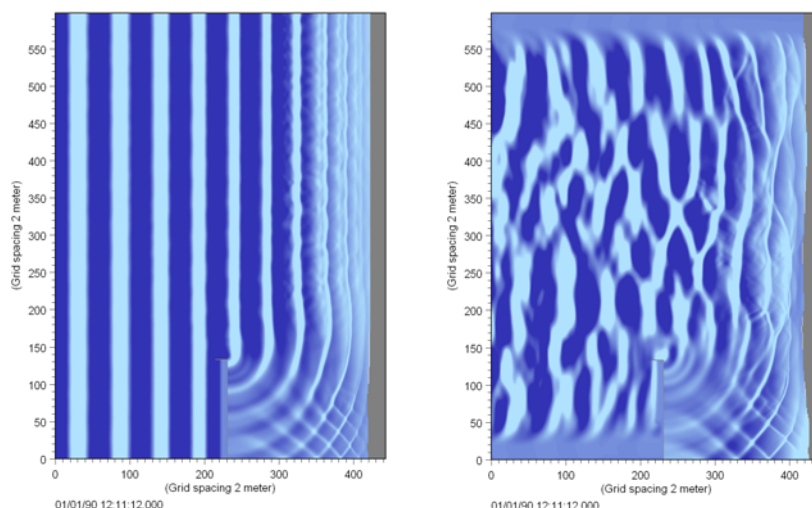
Figure 4.54   Visualisation (2D) of instantaneous surface elevation. Left panel: regular unidirectional waves, right panel: irregular directional waves

The computed wave-induced current field for the two cases is shown in Figure 4.56. Here, the velocities are the depth-averaged velocities under the surface roller obtained by cumulative averaging over approximately 8 (peak) wave periods starting 7 minutes after simulation start. Wave statistics is computed only in points outside the swash zone, which is the white area shown in Figure 4.56. As described in the User Guide it is possible (through a batch execution) to include calculation of wave statistics in the swash zone and an example is shown in Figure 4.57. As the number of sampling values is varying in this wetting and drying zone care should be taken when the results are interpreted.
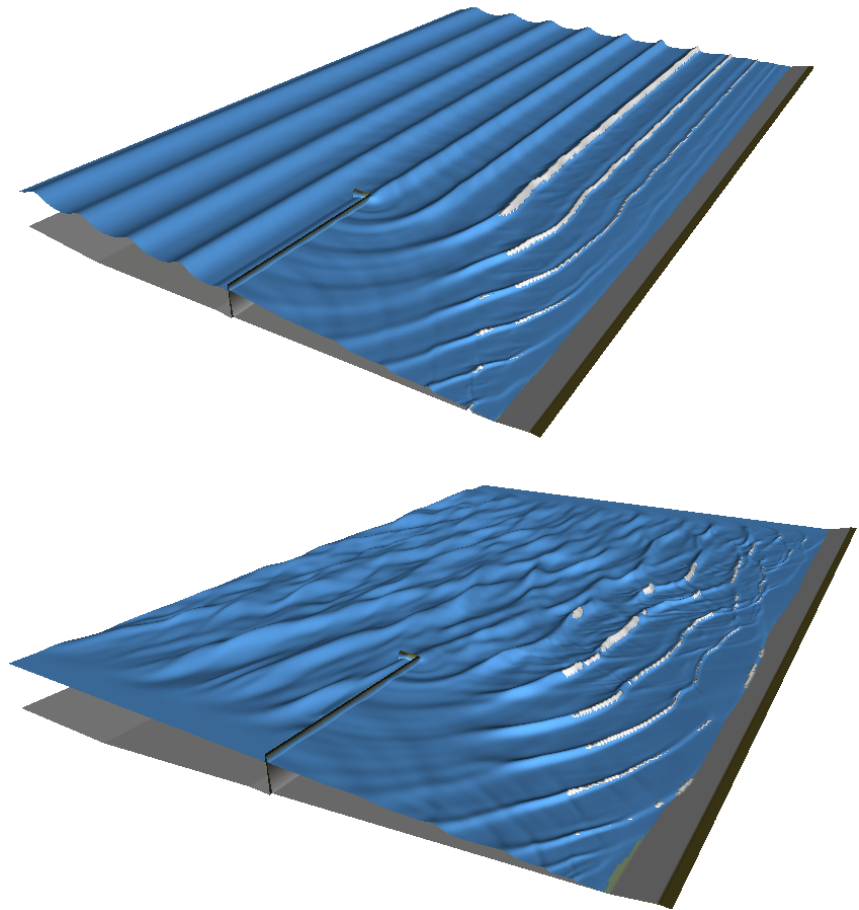
Figure 4.55   Visualisation (3D) of instantaneous surface elevation and the surface rollers. The image (and animation) is created in MIKE Animator Plus. Upper panel: regular unidirectional waves, lower panel: irregular directional waves

By comparing Figure 4.56 with the measurements reported in Sørensen et al (1998, figure 15 p 169) it is seen that the computed eddy structure agrees quite well with the measured structure with respect to the location of eddy, the size of the quiescent area in the centre of the eddy and the maximum speed. As reported in Sørensen et al (1998) the main discrepancy is in the area outside the lee of the breakwater. The model results show that the strong jet along the wall of the breakwater is only slowly disintegrated and bent towards the shoreline, while in the experiment, the jet is bent very strongly at the tip of the breakwater.
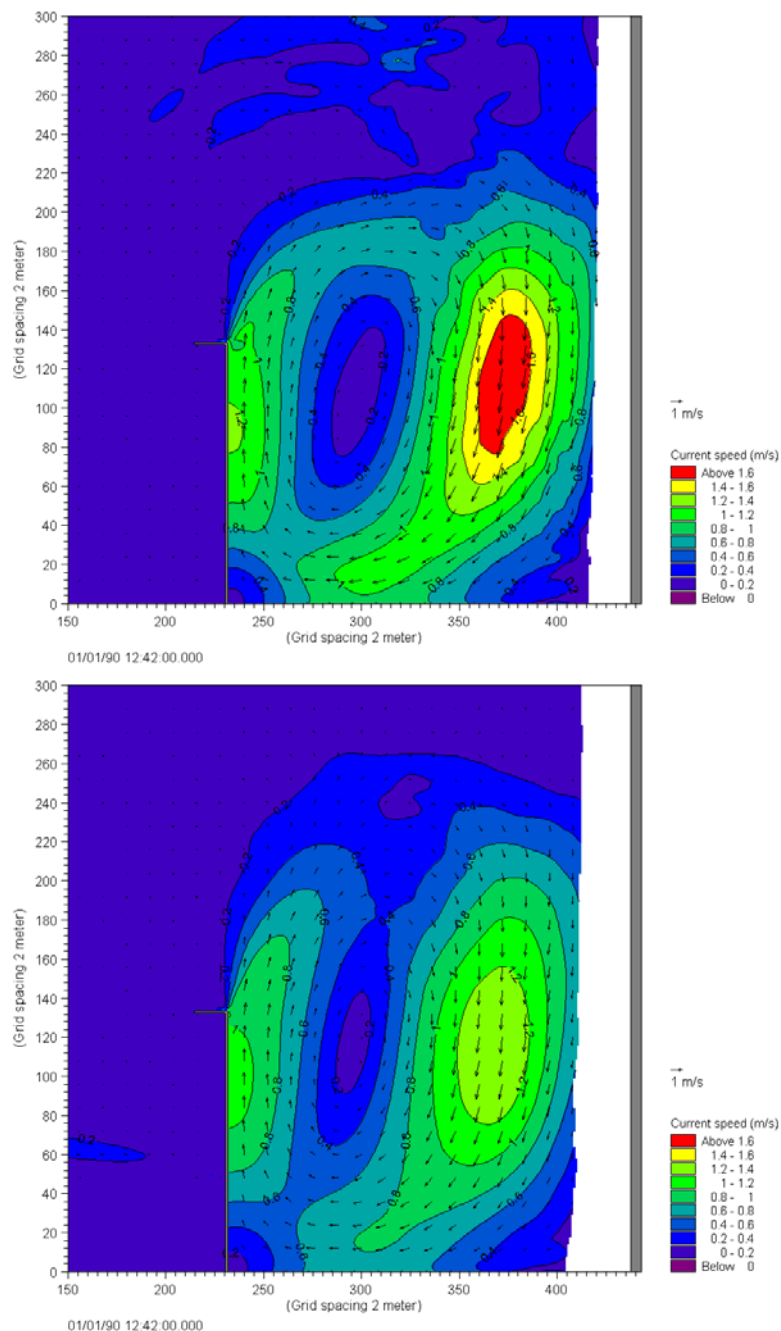
Figure 4.56   Circulation cell behind the detached breakwater for the case of regular, unidirectional waves (upper panel) and irregular, directional waves (lower panel). Computed time-averaged of the velocities beneath the surface rollers. The time-averaging is performed for computational points which always are wet, i.e. outside the swash zone
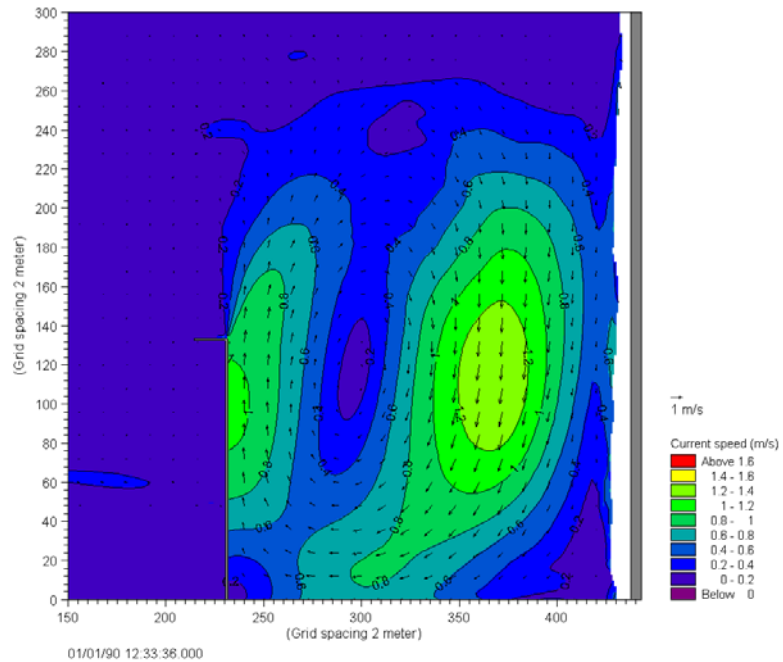
Figure 4.57 Circulation cell behind the detached breakwater for the case of directional waves. Computed time-averaged of the velocities beneath the surface rollers. The time-averaging is performed for computational points which can be partly wet, i.e. also inside the swash zone

Compared to the case with regular waves, the main features of the eddy structure for random waves are that the quiescent area in the centre of the eddy and the maximum speed is reduced.

Figure 4.58 shows a snapshot of the surface elevation for the case of regular waves. A forward bend in the wave fronts can be seen near the breakwater. This phenomenon is caused by the strong following current with a maximum Froude number of 0.2-0.3.
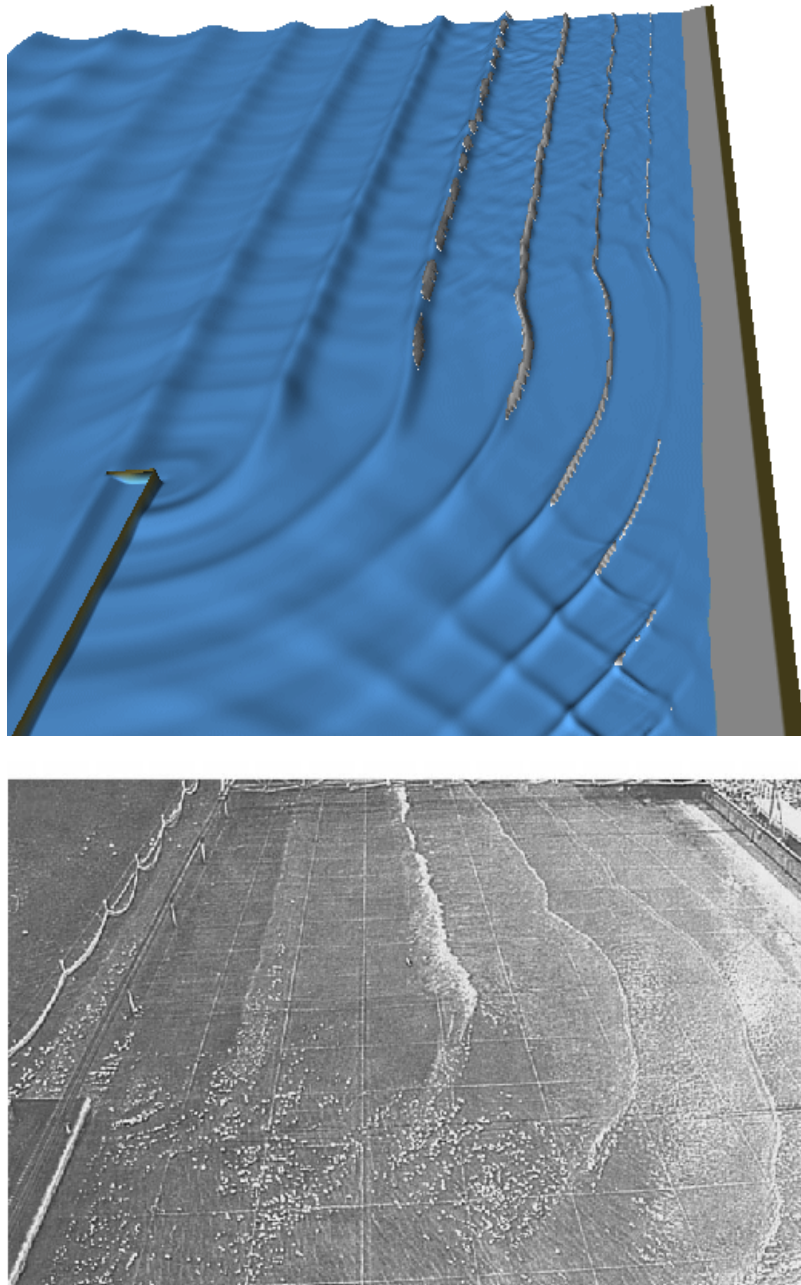
Figure 4.58   Instantaneous surface elevation for the case of regular, unidirectional waves. Upper panel: model results and lower panel: photograph by Mory and Hamm (1997), see Sørensen et al (1998)

The temporal evolution of the wave-induced velocity is shown in Figure 4.59 for the two considered wave situations. The time series is extracted in a point close to the maximum speed. The figure shows a reduction of approximately

30 % in case of irregular, directional waves. The figure also indicates that the time scale for developing the near-steady state flow field is approximately 20 minutes.
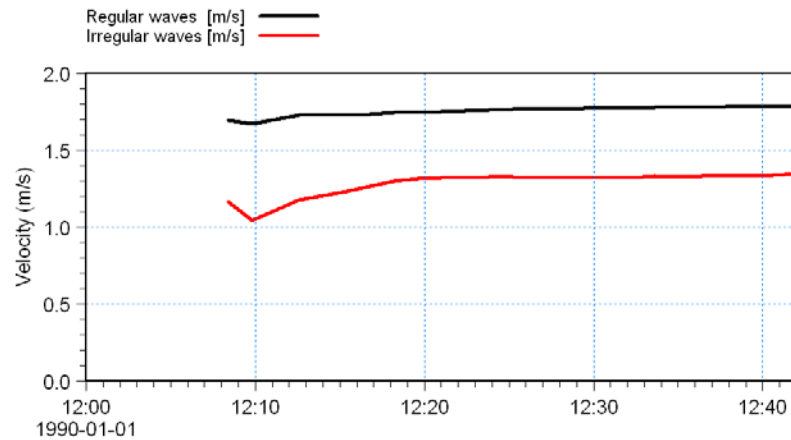
Figure 4.59   Time series of wave-induced current speed. The time series is extracted at P(375,120) with reference to co-ordinate system shown in Figure 4.56

The spatial variation of the relative wave height is presented in Figure 4.60. In general, the breaking zone is wider for the case with random waves than for the case with regular waves and the relative wave height variation is seen to be smoother for the random waves.
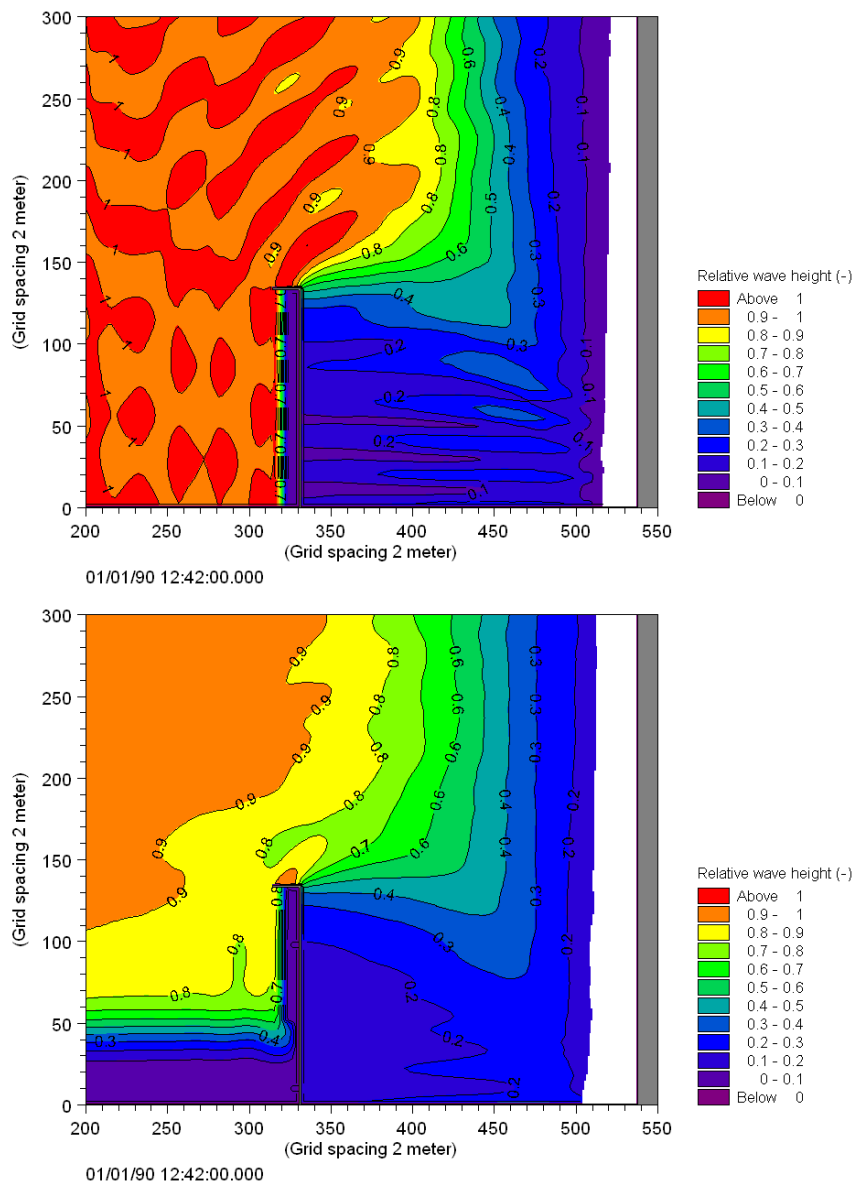
Figure 4.60   Relative wave height in circulation cell for the case of regular, unidirectional waves (upper panel) and irregular, directional waves (lower panel)

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files

Name: Bathymetry.dfs2
Description: Bathymetry data

Name: Sponge Regular Waves.dfs2
Description: Map of sponge coefficients (regular, unidirectional waves)

Name: Sponge Directional Waves.dfs2
Description: Map of sponge coefficients (irregular, directional waves)

Name: Filter.dfs2
Description: Map of lowpass filter coefficients

Parameter files

Name: Tool Waves Generation line.21t
Description: PFS file generation of internal wave boundary data

Name: Setup Regular Waves.bw
Description: PFS file for setup of MIKE 21 BW run
                 (regular, unidirectional waves)

Name: Setup DirectionalWaves.bw
Description: PFS file for setup of MIKE 21 BW run
                 (irregular, directional waves)

Name: Plot Regular Surface Elevation.plc
Description: PFS-file for 2D visualisation the simulated surface
                 elevation using MIKE Zero Plot composer
                 (regular, unidirectional waves)

Name: Plot Directional Surface Elevation.plc
Description: PFS-file for 2D visualisation the simulated surface
                 elevation using MIKE Zero Plot composer
                 (irregular, directional waves)

Name: Plot Regular Wave Induced Currents.plc
Description: PFS-file for 2D visualisation the simulated wave-induced current
                 field using MIKE Zero Plot composer
                 (regular, unidirectional waves)

Name: Plot Directionalr Wave Induced Currents.plc
Description: PFS-file for 2D visualisation the simulated wave-induced current
                 field using MIKE Zero Plot composer
                 (irregular, directional waves)

Name: Plot Regular Wave Disturbance.plc

Description: PFS-file for visualisation the simulated wave disturbance coefficient using MIKE Zero Plot composer
(regular, unidirectional waves)

Name: Plot Directional Wave Disturbance.plc
Description: PFS-file for visualisation the simulated wave disturbance coefficient using MIKE Zero Plot composer
(irregular, directional waves)

Name: Anim Regular Waves with Roller.lyt
Description: PFS-file for 3D visualisation of the simulated surface elevation using MIKE Animator Plus
(regular, unidirectional waves)

Name: Anim Directional Waves with Roller.lyt
Description: PFS-file for 3D visualisation of the simulated surface elevation using MIKE Animator Plus
(irregular, directional waves)

4.2.9   Kirkwall Marina

Purpose of the example

This example is used in the step-by-step training guide document that can be accessed from the MIKE 21 Documentation Index.

The purpose of the step-by-step training guide is to setup a MIKE 21 BW model from scratch and guide you through the various steps in the model setup process, execution and results presentation and visualization.

Figure 4.61   Aerial image of Kirkwall Marina, UK. Photo courtesy Orkney Islands Council

4.2.10   Demo-Diffraction

This simple example simulating wave diffraction is designed for use when running MIKE 21 BW in demo mode.

The following data files and specification file (within the folder of Demo-Diffraction) are supplied with MIKE 21:

Name:	Demo-Layout.dfs2
Description:	Bathymetry and sponge layer

Name:	BC.dfs0
Description:	Boundary conditions

Name:	Demo-Diffraction.bw
Task:	Model: MIKE 21 BW Boussinesq Waves
Description:	Demo simulation, wave diffraction

Please note that in order not to overwrite the specification files you should copy them to your own working folder or rename them.

4.3 1DH Boussinesq Wave Module - Examples

4.3.1 Partial wave reflection

Purpose of the example

This fairly simple example is included to illustrate the process of wave propagation and partial reflection of regular and irregular waves in a numerical wave flume using the 1DH Boussinesq Wave Module. The enhanced Boussinesq equations are solved in this example (as in practically all applications of the 1DH Boussinesq Wave Module), i.e. the deep-water terms are included.

Model setup

The model setup is illustrated in Figure 4.62. The length of the flume is 390 m with a uniform depth of 10 m.
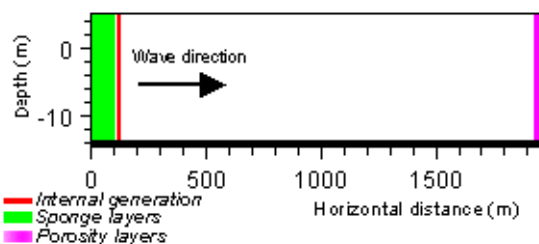
Figure 4.62 Model setup (vertical cross-section). A 20 point wide sponge layer is used at the left-hand side boundary and a 8 point wide porosity layer at the other model extreme. The waves are generated internally at node j = 21

The waves are generated inside the model domain, which is the most common method of forcing the 1DH Boussinesq Wave Module in practical applications. The two model boundaries (i.e. at j= 0 and j= jextr) are considered closed. Opposite to applications of the 2DH Boussinesq Wave Module you do not have to specify land points at the two model extremes.

Although a 20 point thick sponge layer is used in this example, you are in general recommended to apply a sponge layer thickness corresponding to at least once the wave length of the most energetic waves (corresponding to the spectral peak wave period). In practice (with mesh size in the order of 1 m) this means a 50-200 point thick sponge layer. In this example the thickness of the 20 point wide sponge layer is 0.5-0.6 times the wave length corresponding to the spectral peak wave period.

At the generation line (j= 21) a time series of fluxes and surface slope is imposed. This time series is generated using the MIKE 21 *Toolbox* (wave part) program *Regular Wave Generation* and *Random Wave Generation*. For

the regular waves the wave height is H = 1 m and the wave period T =8 s. For the irregular waves the significant wave height is H_{m0} = 1 m and the spectral peak period T_p = 8 s. The waves are synthesised based on a mean JON-SWAP spectrum. The minimum wave period T_{min} = 4 s. Hence the problem can be solved using the enhanced Boussinesq type equations with a dispersion coefficient of B= 1/15.

The thickness of the porosity layer (8 point wide) is about one-quarter of the wave length and the porosity value is set to 0.70. From the reflection-porosity curve shown in Figure 4.63 it is seen that a porosity of 0.70 corresponds to a reflection coefficient of about 0.4, assuming a characteristic wave height and wave period of 1 m and 8 s, respectively.

The model domain is discretized, using a structured mesh with 195 elements and 196 nodes. The mesh size is 2 m and the integration time step is 0.1 s corresponding to a maximum Courant number of about 0.5 s. The simulation duration is 5 minutes (3001 time steps).
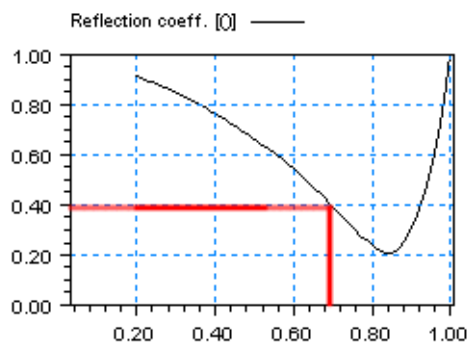
Figure 4.63   Reflection coefficient versus porosity for a 16 m wide absorber in 10 m water depth. The characteristic wave height and wave period is 1 m and 8 s, respectively. Calculated by use of the MIKE 21 Toolbox program Calculation of Reflection coefficient

Model results

Time series of the simulated surface elevation extracted at point P(150) is shown in Figure 4.64 for irregular waves and regular waves. In case of regular waves it is seen that the resulting wave height is increased due to the wave reflection. It is more difficult to identify the increased wave height in case of irregular incident waves.
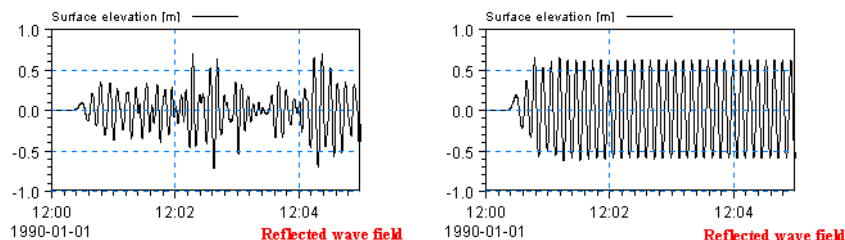
Figure 4.64   Model results. Time series of input and simulated surface elevation at a
              P(150) for irregular (left panels) and regular waves (right panels)

The wave reflection coefficient can be estimated from the model results
shown in Figure 4.65. In case of monochromatic and linear waves the reflec-
tion coefficient R can be estimated, using the formula:

$$R = \frac{a_{max} - a_{min}}{a_{max} + a_{min}} \qquad\qquad (4.1)$$

where a_{max} and a_{min} is the maximum and minimum value of the envelope
amplitude, respectively. From Figure 4.65 is found that $a_{max} \approx 0.65$ m and a_{min}
≈ 0.32, which results in a reflection coefficient of about 0.34. This value is a
little smaller than the expected value of about 0.40 cf. Figure 4.65, which is
mainly due to the wave non-linearity. The incoming waves are not perfectly
linear (as can be seen from the missing wave profile symmetry in
Figure 4.65), which is assumed in the theory for the *Calculation of Reflection
coefficient* program and for the above formula.
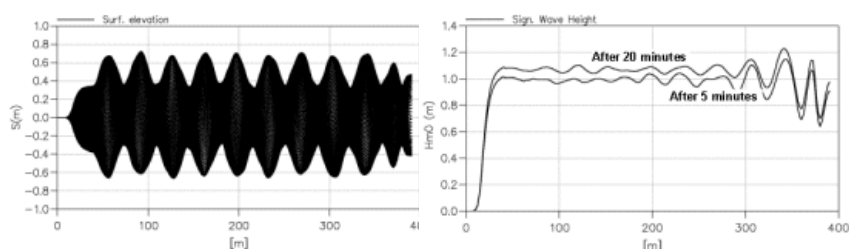
Figure 4.65   Model results. The left panel shows an envelope plot of the surface ele-
              vation (corresponding to one wave period) in case of regular waves.
              The right panel shows line series of the significant wave height at two
              different times in case of irregular waves

In case of irregular and linear waves the reflection coefficient R can be esti-
mated using the formula (based on wave energy considerations):

$$R = \sqrt{\left(\frac{H_{m0}}{H_{m0,i}}\right)^2 - 1} \qquad\qquad (4.2)$$

where H_{m0} and $H_{m0,i}$ are the simulated significant wave height and incident significant wave height, respectively. From Figure 4.65 is seen that after 5 minutes simulation time the significant wave height is close to one (away from the porosity layer). This clearly shows that you need to run the model for a longer period to be able to carry out a proper statistical analysis. If you change the running period from 5 minutes (3001 time steps) to 20 minutes (12001 time steps) you will obtain a variation of the wave height as shown in Figure 4.66. With an average value of the significant wave height of 1.07-1.08 m and the incident significant wave height of 1.0 m the wave reflection coefficient is estimated to 0.38-0.41. This estimate is in excellent agreement with the theory.

Figure 4.66   Model results. 3D pictures of the simulated instantaneous surface elevation. Upper: regular waves. Lower: irregular waves

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files

Name:	Partial_Reflection_mesh.dfs1
Description:	Mesh

Name:	Partial_Reflection_model.dfs1
Description:	Sponge and porosity layer coefficients

Name:	RegularWave_H1.0T8.0h10.0.dfs0
Description:	Wave generation data in case of regular waves

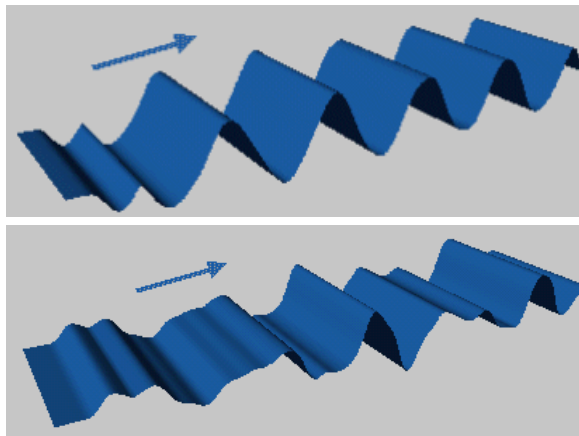|	
Name:	IrregularWave_Hs1.0Tp8.0h10.0
Description:	Wave generation data in case of irregular waves

Parameter files

Name:	Regular_wave_simulation.bw
Description:	PFS-file for the regular wave case

Name:	Irregular_wave_simulation.bw
Description:	PFS-file for the irregular wave case

4.3.2   Sloping beach with wave breaking and moving shoreline

Purpose of the example

Wave breaking and wave run-up on a gently sloping plane beach is considered in this example. The example concentrates on shoaling of regular waves and spilling type of wave breaking.

Model setup

The model setup follows the experimental setup by Ting and Kirby (1994). They presented measurements for spilling and plunging type of breakers on a plane sloping beach with a slope of 1/35 starting in depth of 0.40 m. The model setup is illustrated in Figure 4.67. Waves are generated at internal points by source terms, representing the volume flux in progressive waves. The wave period is 2.0 s and the wave height 0.121 m. The seaward boundary is treated as nonreflective, using sponge layer (100 points). A moving shoreline is included in the simulation, using a slot width of $\varepsilon = 0.01$ and a smoothing parameter of $\beta = 100$ (default values in MIKE 21 BW). A 50 point wide sponge layer is used in the slot in order to damp the oscillations in the slot, see Figure 4.67. With respect to the parameters of the breaker model the following standard (default) values are applied: initial breaking angle $\phi_b = 20°$, final breaking angle $\phi_o = 10°$, half-time for cut-off roller $t_{1/2} = T/5 = 0.4$ and roller form factor $f_d = 1.5$. An explicit filter is introduced near the still water shoreline to remove short-wave instabilities during uprush and downrush and to dissipate the wave energy in the model area where the surface roller cannot be resolved.

As usual the enhanced Boussinesq equations are solved (i.e. deep-water terms included) in 1DH applications.
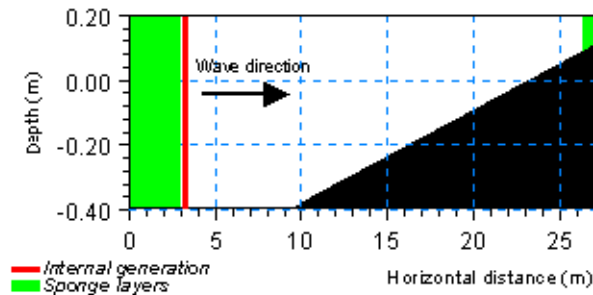
Figure 4.67   Model setup

A <u>structured mesh</u> with elements with an edge length of 0.02 m is used. For the 27.5 m long channel this results in 1375 elements and 1376 nodes. The time step is 0.005 s and the simulation duration 50 s (10001 time steps).

Model results

Figure 4.68 shows a line series of the simulated surface elevation on top of the bathymetry. The wave breaking and wave run-up processes are clearly seen on this figure.

In Figure 4.68 the spatial variation of a number of phase-averaged quantities are presented. The maximum wave height is 0.121 m at the generation point and increases towards the break point, see Figure 4.68. Inside the surf zone the wave height is decreasing. The significant wave height is $\sqrt{2}$ times larger than the regular (linear) wave height. Figure 4.68 also shows the spatial variation of the crest and trough elevation and of the mean water level. In Sørensen and Sørensen (2001)[1] a comparison is made between measurements and simulated results for the crest and trough elevation.

In Figure 4.68 is also seen that the mean water level increases in the surf zone in order to balance the decrease in momentum due to the wave height decay.

---

1   Paper included in the *Scientific Background*
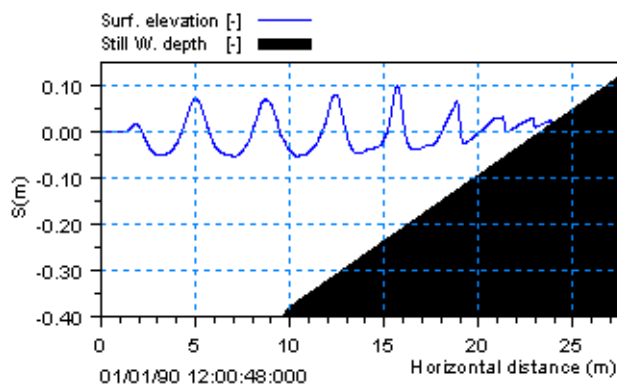
Figure 4.68   Deterministic model results. Line series of surface elevation on top of the bathymetry
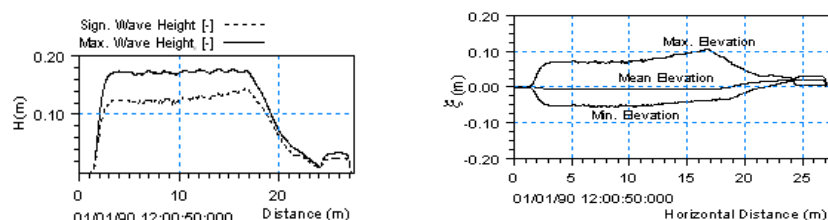
Figure 4.69   Phase-averaged model results. Line series of various quantities

The 1DH Boussinesq module also allows for an estimate of the depth-averaged undertow, which is the return flow below the wave trough. Outside the surf zone the undertow is small and equal to Stokes drift. The calculation and verification of the undertow inside the surf zone is presented in Madsen et al (1997a)[1] p. 276ff. An example of the spatial variation of the undertow is shown in Figure 4.70.

---

1   Paper included in the *Scientific Background*
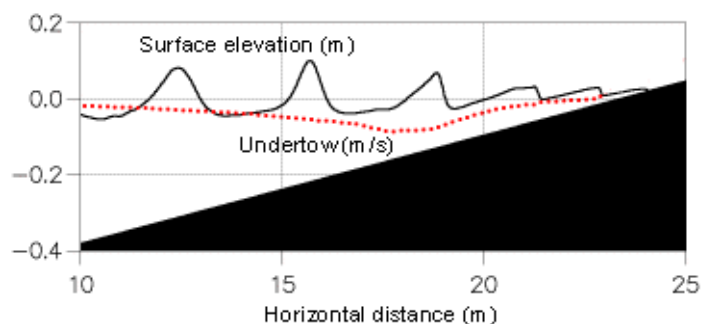
Figure 4.70   Phase-averaged model results. Spatial variation of the undertow
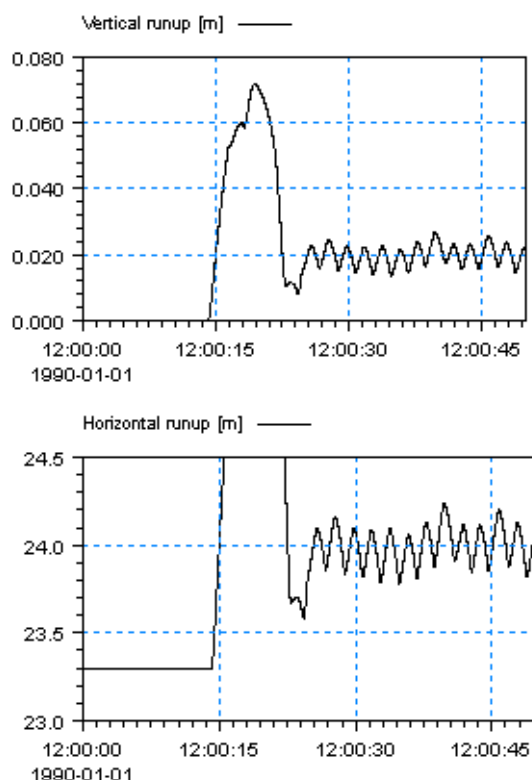
Figure 4.71   Moving shoreline model results. Temporal variation of the vertical and horizontal run-up height. The still water shoreline is located at x_r= 23.3 m

The simulated shoreline motion is converted into a vertical and a horizontal displacement as illustrated in Figure 4.71. The large displacement at time approximately 12:00:20 is caused by the warm-up of the model and should be disregarded. The shoreline motion is seen to oscillate around   0.02 m

(vertical motion) and +0.70 m (horizontal motion), which is due to the wave setup, see Figure 4.69.

Finally, Figure 4.72 shows a 3D picture of the instantaneous surface elevation inside the surf and swash zone.
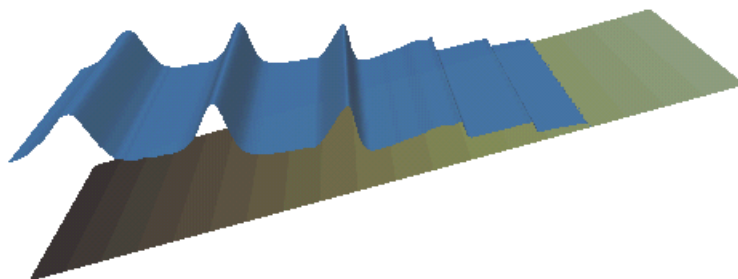
Figure 4.72   Model results. 3D picture of the simulated instantaneous surface elevation

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files
Name: Sloping_Beach_Bathymetry.dfs1
Description: Bathymetry

Name: Sloping_Beach_sponge.dfs1
Description: Sponge layer coefficients

Name: Sloping_Beach_filter.dfs1
Description: Filter layer coefficients

Name: Sloping_Beach_BC.dfs0
Description: Boundary conditions

Parameter files
Name: Regular_waves.bw
Description: Sloping beach with wave breaking and moving shoreline

4.3.3   Torsminde barred beach

Purpose of the example

The purpose of this example is to illustrate how to model wave breaking in case of irregular waves propagating over a natural barred beach profile. The

beach profile is taken from a 2D-bathymetry map covering the area north of Torsminde harbour, see Figure 5.6.

Model setup

The model setup is illustrated in Figure 4.73. In the first part of the profile the slope is approximately 1:100, which is increased to approximately 1:50 on the first bar and further to approximately 1:30 on the second bar. The slope of the still water shoreline is approximately 1:15. Waves are generated at internal points by source terms representing the volume flux in progressive waves (at water depth 13.7 m). The irregular wave time series is generated on basis of a mean JONSWAP spectrum. The spectral peak wave period is 9.0 s and the significant wave height 3.0 m. The minimum wave period included in the time series is 2.5 s.

As the surf similarity parameter is less than 0.5 spilling breakers would also occur in this example. The surf similarity parameter (ξ) is defined as the beach slope divided by the square root of the deep water wave steepness. Based on the classification of Galvin (1968) of wave breaking, Batjjes (1974) found that spilling breakers would occur for $\xi < 0.5$, see Madsen et al (1997a)[1] p. 295.

The seaward boundary is treated as nonreflective, using sponge layer (100 points). Also an absorbing sponge layer is used at the shoreline in this example, see Figure 4.73, i.e. the moving shoreline is not modelled in this example. With respect to the parameters of the breaker model the following standard (default) values are applied: initial breaking angle ϕ_b= 20°, final breaking angle ϕ_o= 10°, half-time for cut-off roller $t_{1/2}$= T_p/5 = 1.8 and roller form factor f_d = 1.5. Also in this example an explicit filter is introduced near the still water shoreline to remove potential.

A structured mesh with elements with an fixed edge length of 1.0 m is used. For the 1980 m long channel this results in 1980 elements and 1981 nodes. Significant wave breaking will occur at the two bars. At the first bar (water depth about 4 m) approximately 54 nodes resolve the most energetic and breaking waves (9 s). At the shallower bar (depth 2.1 m) approximately 41 nodes resolve the waves (9 s). The time step is 0.05 s (results in a maximum Courant number of 0.58) and the simulation duration 1/2 hour (36001 time steps). As usual the enhanced Boussinesq equations are solved (i.e. deep-water terms included) in 1DH applications.

---

1   Paper included in the *Scientific Background*
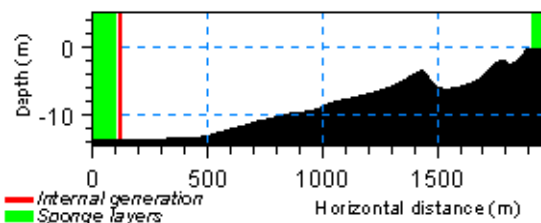
Figure 4.73   Model setup

Model results

Figure 4.74 shows a line series of the simulated surface elevation on top of the bathymetry. Spatial variation of the significant wave height and the maximum wave height is shown in Figure 4.75. In Figure 4.76 the variation of the two nonlinear measures, skewness and atiltness are presented.
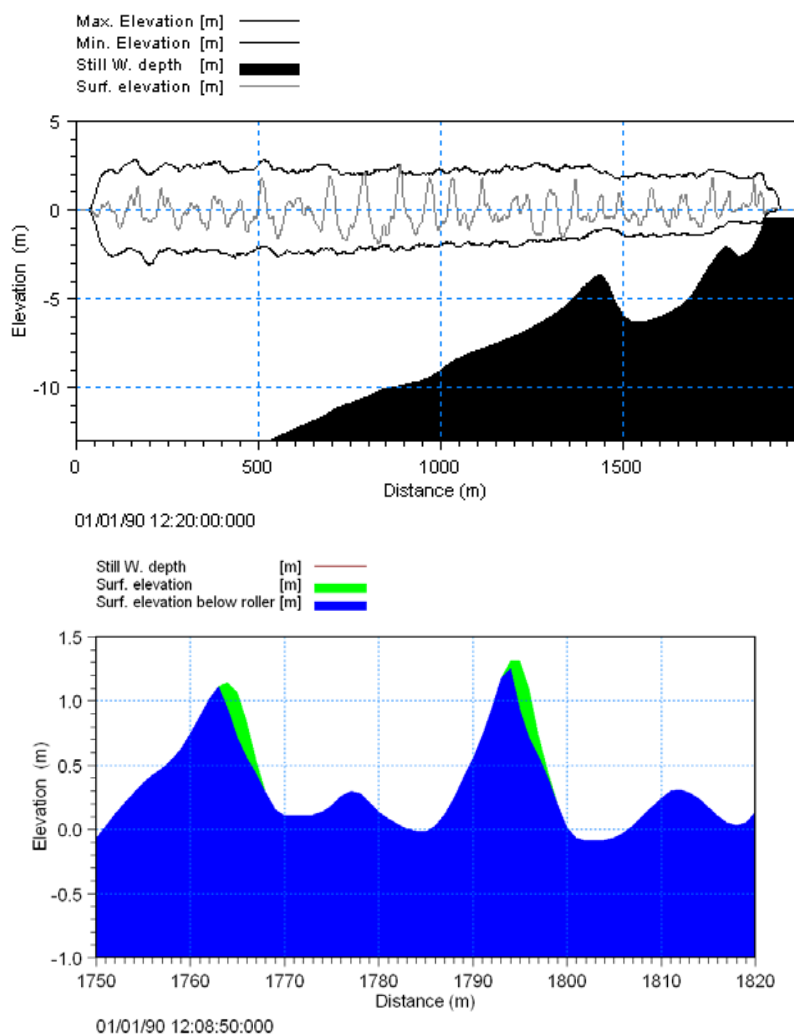
Figure 4.74   Deterministic model results. Upper: Line series of surface elevation on top of the bathymetry. Also the simulated extreme levels of the surface elevation (taken throughout the entire simulation time) is indicated. The dashed line indicates the variation of the original bathymetry at the still water shoreline. Lower: Line series of surface elevation and surface roller for a subset

Sign. Wave Height [m]
Max. Wave Height [m]

01/01/90 12:30:00:000
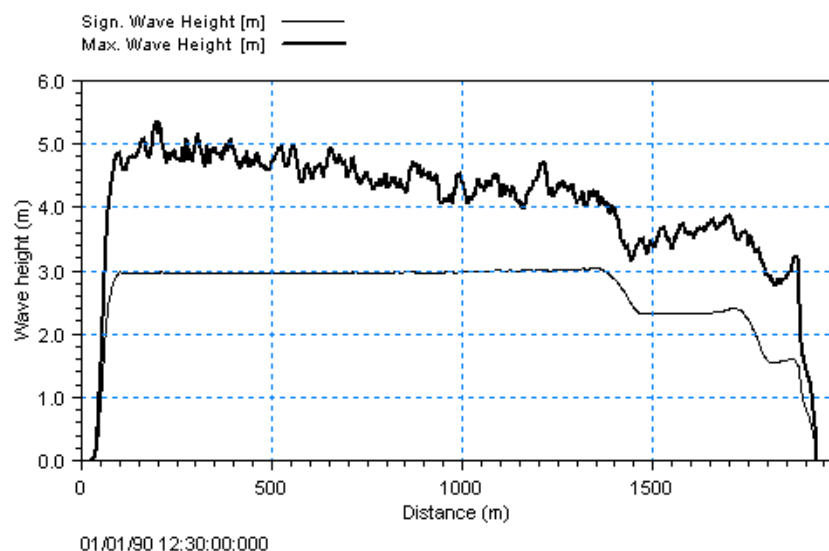
Figure 4.75   Phase-averaged model results. Spatial variation of the significant and maximum wave height over a barred beach profile

Skewness   [0]
Atiltness   [0]
Still W. depth [m]

01/01/90 12:20:00:000
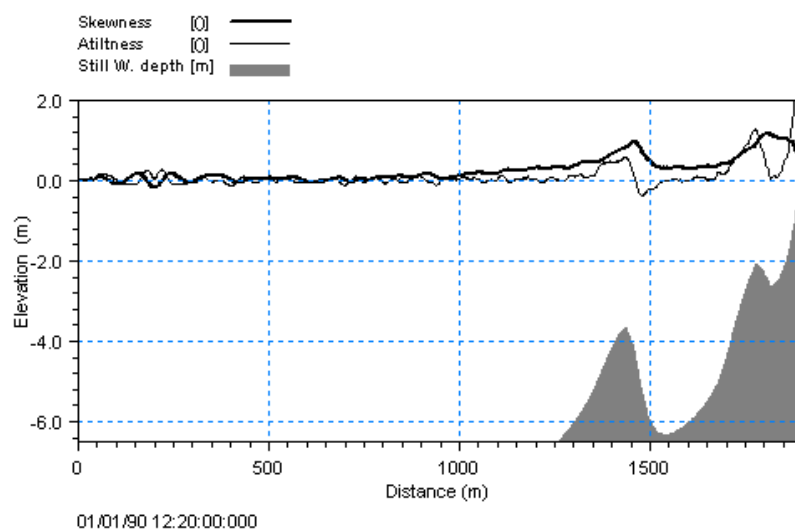
Figure 4.76   Phase-averaged model results. Spatial variation of the skewness and atiltness over a barred beach profile
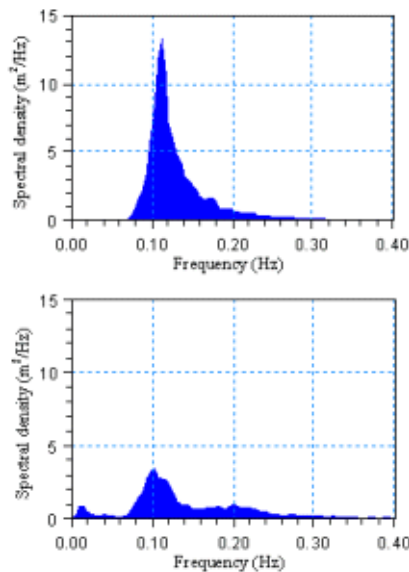
Figure 4.77 Model results. Offshore (upper) and onshore (lower) wave spectra. The spectra are calculated using the WS Spectral Analysis module in MIKE 21
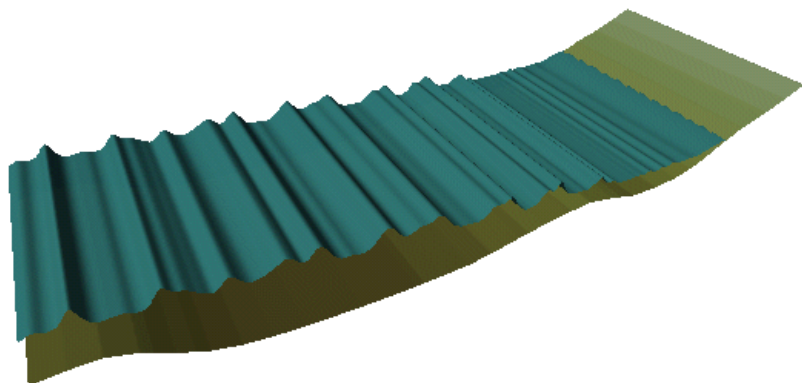
Figure 4.78 Model results. 3D pictures of the simulated instantaneous surface elevation. Please note that this graph is based on a calculation including a moving shoreline

List of data and parameter files

All data required for this example are included in the default installation:

Binary data files

Name:	Torsminde_mesh.dfs1
Description:	Bathymetry

Name:	Torsminde_model.dfs1
Description:	sponge, porosity, resistance and filter coefficients

Name:	H3.0Tp9Tm2.5.dfs0
Description:	Wave Boundary conditions

Parameter files

Name:	Torsminde.bw
Description:	Torsminde Barred beach

5 Reference Manual

5.1 Introduction

It is intended that you use this manual when you are doing model applications with MIKE 21 BW and need to know how various input, output, etc. are specified or defined. The sections are organised in the order in which they appear in the MIKE 21 BW editor (Figure 5.1). Thereafter, all other entries are arranged alphabetically in Section 5.5.
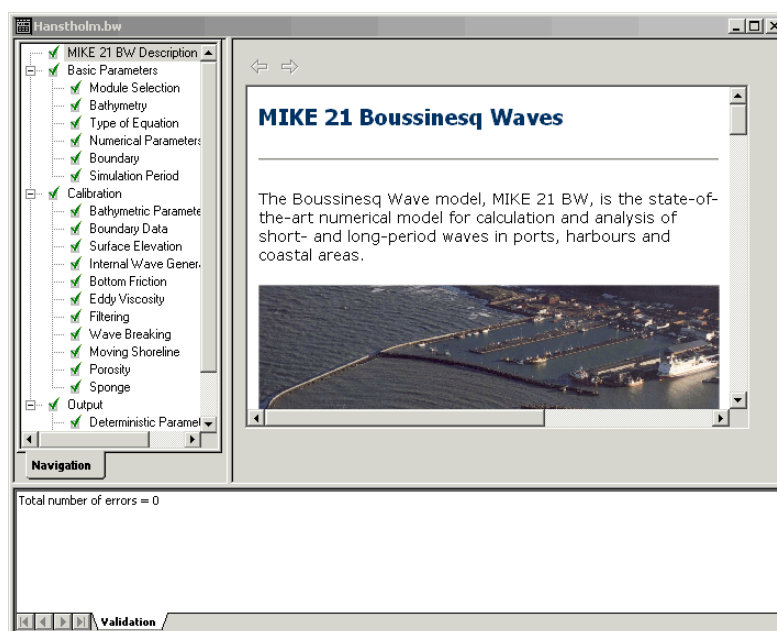
Figure 5.1   MIKE 21 BW editor

The Reference Manual entries are also available in the Online Help for MIKE 21 BW.

It is assumed that you are familiar with the operation of MIKE 21 under the MIKE Zero platform.

5.2 Basic Parameters

The basic parameters group consists of the following sections:

● Module selection

● Bathymetry

● Type of equation

- Numerical parameters (2DH only)
- Boundary
- Simulation period

5.2.1 Module selection

This version of MIKE 21 BW includes two modules:

- 2DH Boussinesq wave module
- 1DH Boussinesq wave module

The 2DH module (two horizontal space coordinates) solves the enhanced Boussinesq equations by an implicit finite difference techniques with variable defined on a space-staggered rectangular grid. This module is typically selected for calculation of short and long period waves in ports, harbours and coastal areas, see Examples in Section 4.2.
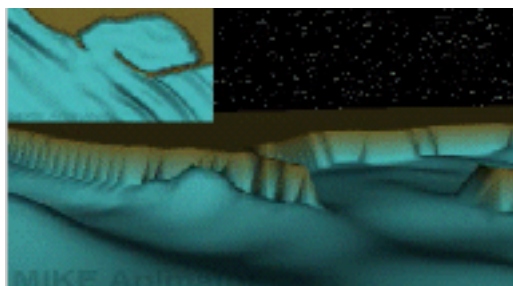
Figure 5.2   Example of a MIKE 21 BW 2DH application

The 1DH module (one horizontal space coordinate) solves the enhanced Boussinesq equations by a standard Galerkin finite element method with mixed interpolation on an unstructured grid. This module is typically selected for calculation of wave transformation from offshore to the beach for studying surf zone and swash zone dynamics, see Examples in Section 4.3.
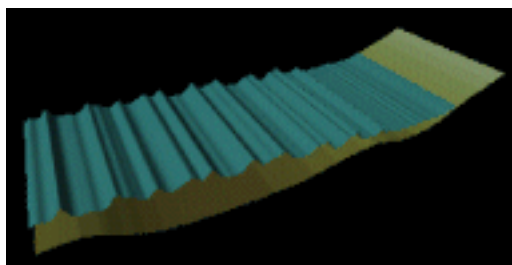
Figure 5.3   Example of a MIKE 21 1DH application

Temporary file for 1DH applications

On basis of your model parameter specifications (saved in a PFS file, e.g. run01.bw) the 1DH module generates a new file.

PFS file
The file named run01.bw_FEM.pfs includes all specifications needed to run your 1DH model.

As default this file will be deleted when the model simulation stops. However, on the Module Selection dialog you can specify that you would like to save the file.

5.2.2   Bathymetry

Providing MIKE 21 BW with a suitable model bathymetry is essential for obtaining reliable results from your model. Setting up the bathymetry requires more than just specifying a 2D/1D array of accurate water depths covering the area of interest. It also includes the appropriate selection of the area to be modelled, the grid spacing, location and type of boundaries etc.

You can start a simulation in two different ways:

- as a cold start
- as a hot start (2DH module only)

For a cold start the flux is initialized to zero. When choosing this option you have to specify a 2D grid for 2DH applications (type 2 data file with extension dfs2 or dt2) or a 1D profile for 1DH applications (type 1 data file with extension dfs1) containing the bathymetry/mesh.
Note: It is not possible to use a bathymetry specified for a Geographical Coordinate System such as e.g. LONG/LAT.

The hot start facility requires a so-called hot start data file. This must originate from a previous simulation. The hot start file contains all necessary information to continue a simulation. The hot start facility is particularly useful for 2DH applications involving wave-current interactions, where the wave simulation is hot started from a run with currents only.

When setting up the bathymetry it should be kept in mind that the water depths determine which wave conditions that can be modelled. The maximum water depth restricts the minimum wave period that can be modelled and the minimum water depth may restrict the wave height if wave breaking is not included. Further, shallow water results in small wave lengths which implies small grid spacing which again results in increased CPU time. Therefore it can be necessary to modify the bathymetry to reach an acceptable compromise between a correct bathymetry and correct wave conditions.

Selecting a model area (2DH)

In deciding the area to be included in your model, you must consider the area of interest, and the positions and types of the model boundaries to be used.

In many short-wave applications, such as enclosed harbour basins, the area of interest is relatively well defined. Here, the selection of the area to be modelled reduces to deciding on the position and type of the boundaries.

A typical situation is schematised in Figure 5.4.
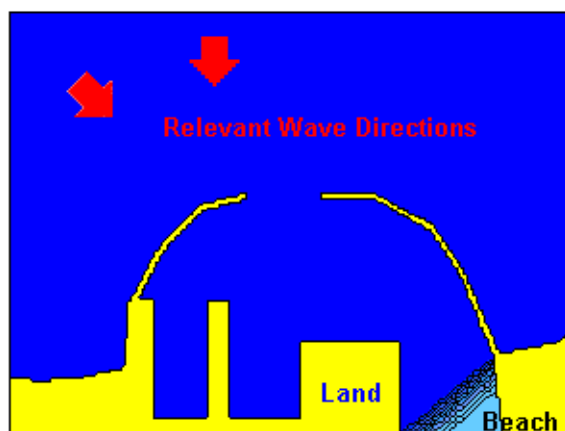
Figure 5.4   Enclosed Harbour

In many cases involving breakwaters, what happens in part of the area between the outer face of a breakwater and an adjacent model boundary may have no influence on the main area of interest. These areas can be turned into (**artificial) land** without affecting the accuracy of the results, as illustrated in Figure 5.5. This can result in significant savings in computation time.

It is stressed that the selection of model area and **artificial land** areas is closely connected to the wave conditions to be modelled. In this context the wave direction and directional spreading is important (see section 5.3.4 on Setup of internal generation lines).
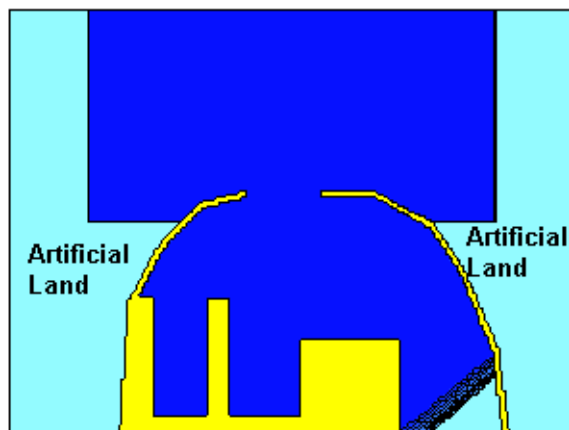
Figure 5.5   Enclosed harbour with reduced computational area

In the example, the shallow area inside the harbour is also converted to **artificial land** values, for two reasons:

● It is assumed that the shallow area/beach will absorb the wave energy. Therefore sponge layers will be placed in front of the land values.

● The minimum water depth in the model area defines the grid spacing, as the minimum wave length is defined by the minimum water depth in combination with the minimum wave period (see Selecting the grid spacing below). Therefore very shallow areas should be avoided (if wave breaking is not included), where possible, to reduce the CPU demand.

The areas, unimportant for the propagation into the harbour, may differ for different incoming wave directions. This is described in Setup of internal generation lines, section 5.3.4.

In other applications, such as semi-enclosed harbours, along exposed coastlines, or around offshore structures, the area to be modelled is not as well defined. Then, it is important that the model is extended far enough from the area of interest to ensure that model results will not be influenced by close proximity to the model boundaries.

As a rule-of-thumb, the minimum distance between the end of a structure and an adjacent closed boundary should preferably be five or six characteristic wave lengths.

Selecting a coastal profile (1DH)

In deciding the coastal profile to be included in your model, you must consider the profile of interest, and the positions and types of the model boundaries to be used. You also have to consider whether wave breaking and moving shoreline should be included.

In most applications the profile of interest is relatively well defined. Hence, the selection of the profile to be modelled reduces to deciding on the position and type of the offshore boundary. Usually you would apply an internal wave generation boundary. A typical situation is schematised in Figure 5.6.
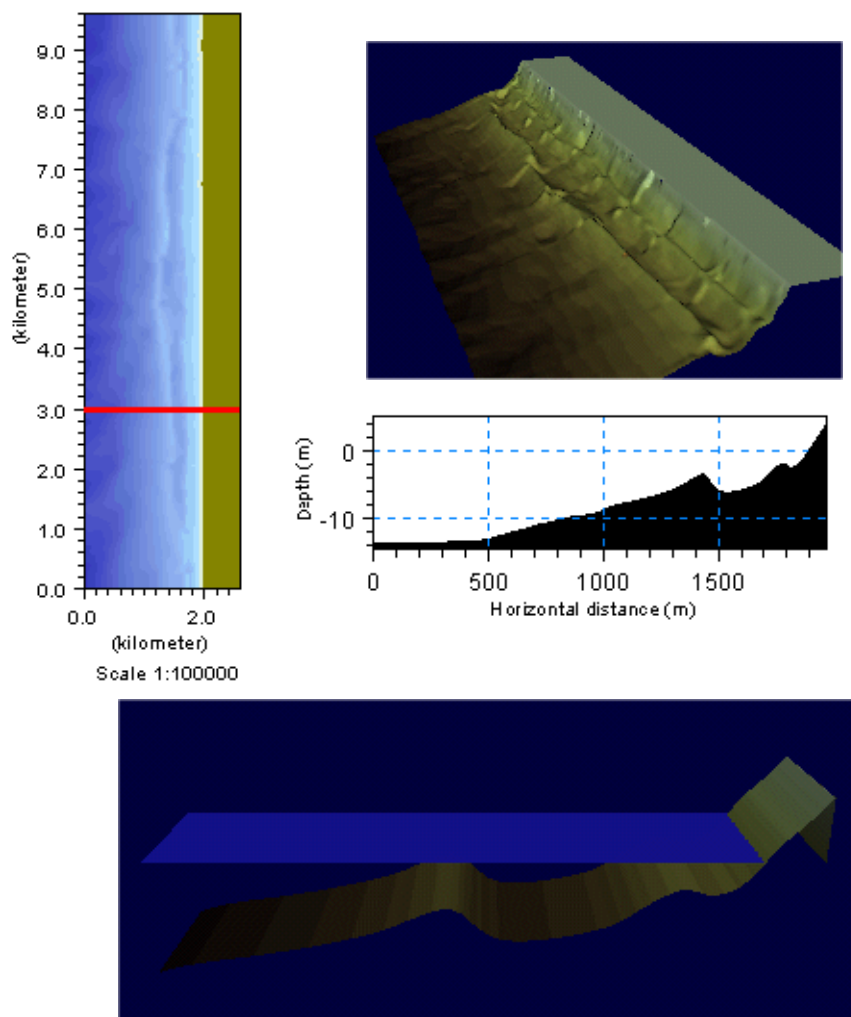
Figure 5.6   Selection of a coastal profile for use in MIKE 21 BW 1DH

The line on top of the 2D bathymetry plot indicates the location of the extracted profile. It is noticed that the bathymetric profile includes depths larger than zero, which means wave breaking and wave run-up have to be included in this model setup. More information about this example can be found in section 4.3.3 Torsminde barred beach.

As in 2D applications the minimum water depth in the model area defines the grid spacing, as the minimum wave length is defined by the minimum water

depth in combination with the minimum wave period, see Selecting the grid spacing below.

Selecting the grid spacing

It is important that the grid size is selected to provide adequate resolution of the main characteristics of the waves under consideration. For short wave applications the main requirement is usually that the grid size is sufficiently small to adequately resolve the individual waves to be used in each model simulation.

For most practical 2 DH applications 7-10 grid points per wave length will be sufficient. Hence, it is necessary to estimate the smallest wave length in the model area. This will be found for the shortest wave period in the most shallow part of the model bathymetry. A typical spatial resolution is 2-5 m.

When wave breaking and wave run-up is included, the minimum water depth is theoretically zero and a very small spatial resolution is required to resolve the wave field. **In practical applications it has been found that the most energetic waves should be resolved by at least 20-40 grid points in the surf and swash zone**. In practical 2DH applications with wave breaking and moving shoreline a spatial resolution of 1-2 m is often used. For cross-shore simulations using the 1DH module an unstructured mesh is often used where the relative resolution is fixed, say, 40 nodes per wave length for the entire profile.

Further it is required that the grid spacing in combination with the time step is selected to fulfil the CFL (Courant number) criterion.

Important: The Courant number should always be equal to, or less than, 1 in 2DH applications and less than about 0.5 in 1DH applications.

For an efficient determination of the model setup resolution in space, time and model execution, you can use the MIKE 21 BW Model Setup Planner included in the Online Help.
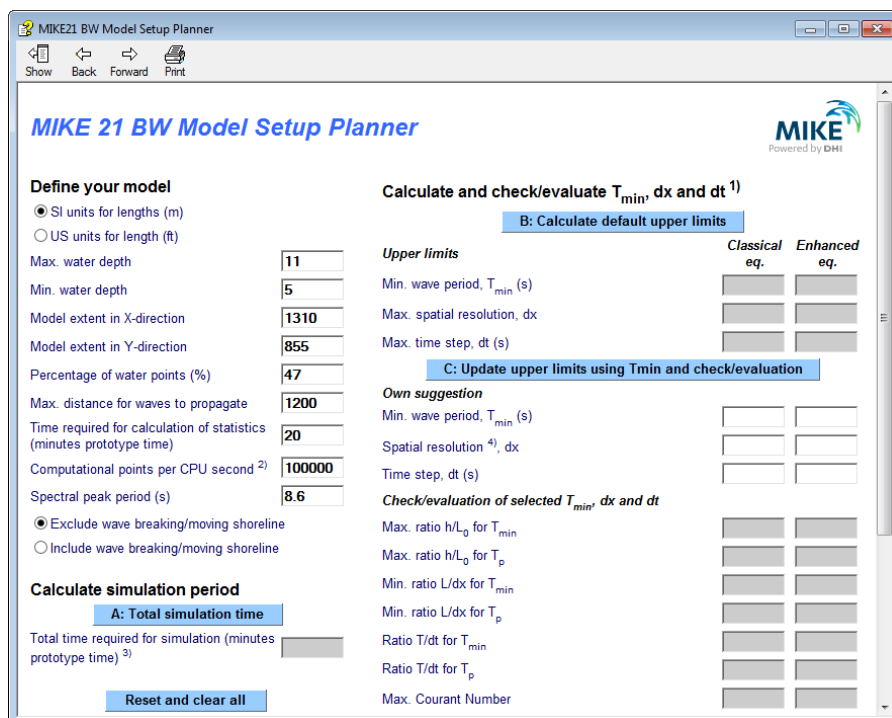
Figure 5.7   MIKE 21 BW Model Setup Planner efficient model setup and execution

Example

A minimum wave period of T_{min} = 6 s and a minimum water depth h_{min} = 4 m gives a minimum wave length of L_{min} = 35 m.

Grid Spacing: The grid spacing should then be at least 35/7 = 5 m, in order to ensure 7 grid points per wave length.

Sign convention

Bed levels (water points) are specified as negative values when they are below the bathymetry datum, and positive values when they are above it.

Note that simulations are carried out using the still water level (SWL) as reference. SWL = bathymetry value + shift of reference level, see Figure 5.8.

Example

As an example, consider a bathymetry created using LAT as datum. You wish to run a simulation for mean sea level (MSL). The MSL is 1.0m above LAT, so you specify +1.0m as shift of reference level. At a point where the bathymetry bed level is -7.0m, the total water depth becomes
-(-7.0m) +1.9m = 8.0m.
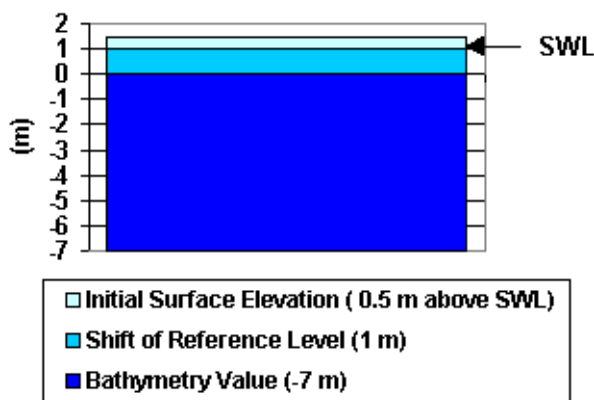
Figure 5.8   Sign convention of bed levels

Simulation start type

In most cases you will start the simulation as a cold start, meaning the simulation will start "from scratch", and not as a continuation of an earlier simulation, i.e. a hot start simulation.

Breaking waves

As a wave shoals it steepens, increasing in height until the crest becomes unstable and the wave breaks. Once the wave breaks, excess wave energy is dissipated through turbulence. The onset of the physical instability occurs when the water particle velocity at the crest of the wave begins to exceed the speed of wave propagation.

Figure 5.9   Wave breaking

Boussinesq simulations without wave breaking

By comparison, the Boussinesq equations in MIKE 21 BW use the depth-integrated velocity as the main velocity variable. This can never exceed the speed of wave propagation. As a consequence, shoaling waves in MIKE 21 BW will continue to shorten and steepen until they can no longer be resolved adequately by the computational grid. Ultimately this leads to numerical instability. Depending on the size of the grid, the onset of the numerical instability may occur well beyond the point at which a real wave would have broken.

In deep water the enhanced Boussinesq equations become effectively linear. In more shallow water, the enhanced equations converge towards the classical equations, and the non-linear properties will be similar to the ones obtained by the classical equations. These are known to provide good results for finite amplitude waves up to at least 75% of the breaking height assuming wave breaking is not included in your model setup

For most practical applications where wave breaking is not included you should thus make sure that the ratio between the significant wave height and water depth is less than 0.3-0.4.

Boussinesq simulations with wave breaking

Wave breaking and moving shoreline have been implemented in a both modules of MIKE 21 BW (1DH and 2DH module). The inclusion of wave breaking is based on the surface roller concept for spilling breakers following the formulation by Schäffer et al (1993), Madsen et al (1997a,b) and Sørensen et al (2004), see references and scientific documentation.

Successful modelling of the wave breaking process requires an adequate resolution of the surface rollers. In most 2DH applications where wave breaking is included a grid spacing of 1-2 is used for wave events having a spectral peak period larger than, say, 7 s. It is recommend to resolved the most energetic waves with minimum 20-40 grid points per wave length in the surf zone. In 1DH applications you would typically use an unstructured grid/profile in such way that the relative resolution is constant, say, 40 grid points per wave length. This means a coarse spatial resolution offshore (e.g. 10 m) and a fin resolution at the shoreline (say, 0.5 m).

Independent of the choice of module, the deep-water terms should always be included, i.e. the enhanced model equations are solved. Otherwise the non-linear interactions between harmonics in shallow water may be underestimated.
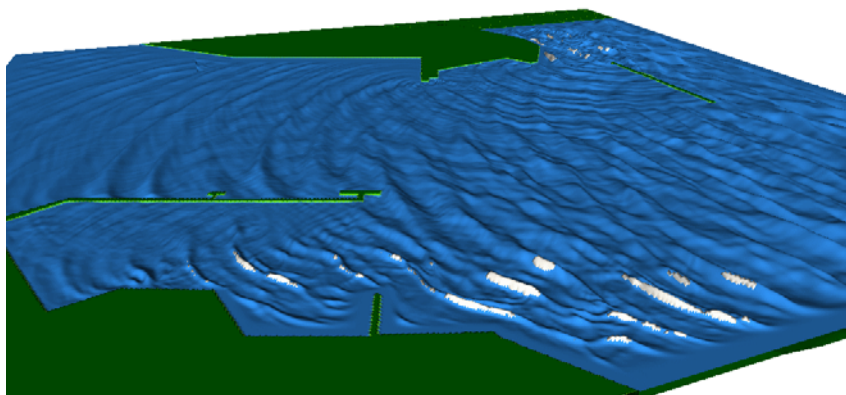

Figure 5.10   Instantaneous surface elevation simulated with the 2DH module including wave breaking (the surface rollers are shown in white)

Remarks and Hints

Time domain wave modelling including wave breaking and moving shoreline is a difficult numerical task. You may sometimes see numerical instabilities and eventually blow-up in some of your applications. In such case please follow the guideline given in Section 5.5.4.

We strongly recommend you to carefully walk through the application examples included in this installation prior to your first application with wave breaking and moving shoreline.

5.2.3 Type of equation

You can choose between the following two types of equations:

- Classical Boussinesq equations
- Enhanced Boussinesq equations

The classical Boussinesq equations are used if the ratio between the maximum water depth and the deep water wave length, h_{max}/L_0, is less than 0.22. The enhanced Boussinesq equations include the so-called **deep water terms** allowing to extend the model into deeper water/smaller wave period, say, $h_{max}/L_0 = 0.5$.

It should, however, be noted that including the deep water terms requires substantially more **CPU time**, as both the grid spacing and time step must be reduced.

Remarks and hints

The 1DH applications requires the deep water terms to be included

For 2DH applications, including wave breaking and moving shoreline, the deep water terms should be included for improved accuracy and numerical stability.

Linear dispersion factor

For most practical applications a value of B = 1/15 = 0.067 is recommended.

5.2.4 Numerical parameters (2DH only)

On this dialog you specify the method for space and time integration of the convective terms in the Boussinesq equations.

Discretisation of the convective terms

Default settings

As default the 2DH Boussinesq equations are solved by implicit finite difference techniques with variable defined on a space-staggered rectangular grid. The convective terms are discretized using central differences and the normal ADI algorithm with 'side-feeding' is used for numerical integration. The 'side-feeding' technique (see e.g. Abbott and Minns, 1998, p 314ff) is introduced to centre the cross-momentum derivatives (the so-called **cross-terms**) without numerical dissipation.

Alternative schemes

In applications with large gradients in the convective terms the default approach may results in numerical instabilities and eventually model **blow-up**. Those applications are characterised by having high spatial resolution (e.g. dx= dy= 1-2 m) and large incident short period waves. To circumvent instabilities alternative discretisation methods of the convective terms have been implemented:

- central differencing with simple upwinding at steep gradients and near land

- quadratic upwinding with simple upwinding at steep gradients and near land

- simple upwinding differencing

All three schemes are characterised by having some numerical dissipation, which may damp possible instabilities. The simple upwind scheme is the most dissipative scheme. The dissipation is in general much less than the damping caused by using backward time centring of the **cross-terms**.

Recommendations

If instability occurs it is recommended to first apply the "Central differencing with simple upwinding at steep gradients and near land". In case of a blow-up you can try the quadratic upwinding scheme. If the instability still persists please try the simple upwind scheme.

Please notice that the **CPS (Computational Points per Second)** is lower for the upwind schemes than using central differences and the normal ADI algorithm with "side-feeding". The **CPS** is typically reduced by a factor of 0.5-0.8.

For stability reasons it is recommended to use the scheme 'Simple upwinding at steep gradients and near land' for the space discretisation of the convective terms in connection with simulations including wave breaking (and moving shoreline).

Time discretisation of cross-Boussinesq terms

The representation of the **Boussinesq cross terms** (i.e. Q_{xyt}, Q_{yt} and Q_{xt} in the x-sweep and P_{xyt}, P_{xt} and P_{yt} in the y-sweep) has required special attention.

Time-extrapolation factor of 1

In order to obtain the correct time-centering, we have used linear time-extrapolation of these terms. A straightforward numerical representation leads to a backward centering of these terms and will result in artificial dissipation of waves propagating with an angle to the grid. This corresponds to a time-extrapolation factor of 1 (default value).

Time-extrapolation factor of 0

A straightforward numerical representation leads to a backward centering of these terms and will result in artificial dissipation of waves propagating with an angle to the grid. This corresponds to a time-extrapolation factor of 0.

Solution of the enhanced Boussinesq equations (deep-water terms included)

Even though approximately 35 time steps resolve the minimum wave period, the time-integration of the enhanced Boussinesq equations may sometimes result in instabilities, which eventually may cause a model blow-up. Often the instability (high frequency noise) appears in the computational domain with the largest water depth (typically near an internal generation line or at an open boundary). As explained in "**Blow-up** after some time steps" (see link below) the time step should be reduced to avoid this instability. Alternatively, you can use a depth-dependent time-extrapolation factor.

Recommendations

If instability appears include 'Depth-dependent time-extrapolation' and set the time-extrapolation factor to 0.8 in the area (determined by the water depth) where the high-frequency noise appears. If this still does not help reduce the time-extrapolation factor to 0.5. If problems still occur you can set time-extrapolation factor to zero (for a local area).

It is important to limit the area where a time-extrapolation factor different from one is used. Application of a time-extrapolation factor of 0 in the entire computational domain should only be used as a last resort.

In most 2DH applications, including wave breaking and moving shoreline, a time extrapolation factor of slightly less than one is recommended for numerical stability. Time extrapolation factors within 0.8-0.9 (for all water depths) have successfully been used in a number of wave breaking applications.

5.2.5   Boundary

In most cases boundary positions and number of boundaries detected by the model can be used. But in special cases you might want to define the posi-

tions explicitly. This can be due to internal boundaries, a boundary stretching over a series of small islands, etc.

The model requires you to specify either the surface elevation or the flux at all open boundaries.

Recommendations

As open boundaries are treated as fully reflective in MIKE 21 BW it is recommended to use internal wave generation. In 2DH applications involving wave-current interaction an open boundary may be used, e.g. an open flux boundary.

5.2.6 Simulation period

On the Simulation Period dialog you specify information on

- Number of time steps
- Time step interval
- Warm-up Period

Furthermore, you must specify the date and time for the simulation.

Number of time steps

The number of time steps is the total simulation time divided by the time step.

The total simulation time should allow for waves to reach all model areas and from this time (start-up period) the simulation should continue at least 15-20 min. for calculation of statistical parameters such as the significant wave height, H_{m0}. If the study includes investigation of e.g. long periodic wave disturbance (e.g. harbour resonance and seiching or wave-induced currents) the simulation time should be longer.

For typical short wave applications the simulation time is 20 - 30 minutes.

The figure below shows a typical surface elevation time series extracted from inside a harbour. Note that no elevations are present in the beginning as no waves have reached the basin of the harbour. Therefore, this period should not be included in the calculation of the wave statistics.
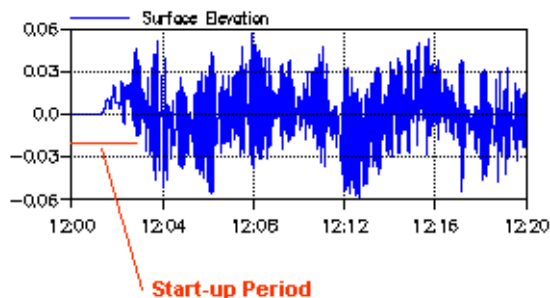
Figure 5.11   Example of time series of simulated surface elevation

Time step interval

The selection of the time step depends on selected module and the form of the Boussinesq equations. The guidance and recommendations listed below is mainly related to the 2DH Boussinesq wave module. The time step constraint for the 1DH Boussinesq wave module is that the Courant number is less than about 0.5.

- **Excluding the deep water terms:**

The main requirement is that the time step should be sufficiently small to resolve the shortest individual wave periods. For most practical applications 8-10 time steps per wave period will be OK, applied to the minimum wave period to be considered. Further, it is also required that the time step (in combination with the grid spacing) is selected so that the Courant number is less than or equal to 1.

- **Including the deep water terms:**

To avoid numerical damping and instability, the minimum wave period should be resolved by 25-35 time steps. Again it should be checked if the Courant criterion is fulfilled.

It is recommended to use the MIKE 21 BW Model Setup Planner for determination of the optimal time step.

- **Including wave breaking and moving shoreline:**

In practical applications involving wave breaking and moving shoreline the grid space is typically 1-2 m and the time step is typically 0.05-0.1 s depending on the time scale of the waves. The lower values are required when a moving shoreline is included.

Warm-up Period

The warm-up period is a period where the forcing functions (boundary conditions) gradually are built up to their prescribed value. Often a warm-up period can be used to avoid blow-ups during the first time steps.

In case of application with moving shoreline a warm-up period is most often required. Consider a warm-up period corresponding to 5-10 characteristic wave periods.

The warm-up period should not be confused with the "Start-up period" discussed under section 5.4.3 Wave disturbance parameters.

5.3 Calibration Parameters

The calibration parameters group consists of the following sections:

- Bathymetric parameters
- Boundary data
- Surface elevation
- Internal wave generation
- Bottom friction
- Eddy viscosity
- Filtering
- Wave Breaking
- Moving shoreline
- Porosity layers
- Sponge layers

5.3.1 Bathymetric parameters

On this dialog you specify information related to the bathymetry i.e. the definition of land points, and shift of reference level, if any.

Bathymetry value representing land

This value is the minimum value you have used for land points when you prepared the bathymetry. In model calculation grid points with values higher than this value are always considered as land.

Shift of reference level

The still water level (SWL) is used as the main reference level for Boussinesq wave simulations.

You can, however, use any convenient datum for setting up the bathymetry of your model. This may for example be Chart Datum, lowest astronomical tide (LAT), or mean sea level (MSL). The actual datum you use is not important. What is important, is that for each simulation you must provide the model with

the correct height of the still water level, i.e. the model reference level, relative to the datum used in setting up your bathymetry, see the figure below.

This is done by specifying the Shift of Reference Level. Then, the model will automatically make any internal adjustments necessary for that particular model simulation. In this way it is possible to carry out model simulations using a range of different water levels, without having to alter the bathymetry.

NOTE: When using the Shift of Reference Level option (in the Calibration/Bathymetric Parameters dialog) you should remember also to change the input wave data as the water depth at the generation lines change accordingly to the change in water level.

Often the 'safest' thing to do is to make new (copies) of the bathymetry files and change the depths in the different bathymetry files.
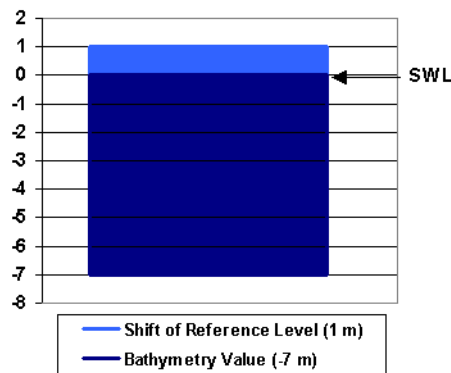

Figure 5.12   Example of shifting the reference level

Example

As an example, consider a bathymetry created using LAT as datum. You wish to run a simulation for mean sea level (MSL). The MSL is 1.0m above LAT, so you specify +1.0m as shift of reference level. At a point where the bathymetry bed level is –7.0m, the total water depth becomes
-(-7.0m) +1.0m = 8.0m.

5.3.2   Boundary data

At an open boundary the incident waves are specified as time series of surface elevations, or flux densities, perpendicular to the boundary. If the Deep Water Terms are included, the time series of the surface curvature or the surface slope must also be specified.

The input data files can be given as a time series (i.e. dfs0) or a line series (.dfs1)

Open boundary

At an open boundary the incident wave energy is given as a time series of surface elevations or flux densities perpendicular to the boundary. If the deep water terms are included, the time series of the surface curvature or the surface slope must also be available.

With respect to wave modelling it is important to notice that an open boundary is fully reflective. This is the main reason that internal wave generation is often preferred as opposed to 'open boundary wave generation'.

To prepare the boundary input files the wave tools in the MIKE 21 Toolbox may be applied.

Closed boundary

A closed boundary (land points) does not allow for flow across the boundary. A closed boundary is fully reflective when not combined with sponge or porosity layers. For a closed boundary, boundary conditions (such as surface elevations) can not be specified.

Absorbing boundary

An absorbing boundary allows wave energy to pass out of the model area without reflections propagating back into the model area.

In the model an absorbing boundary consists of a number of sponge layers typically backed up by a closed boundary (land values), see section 5.3.11 Sponge layers.

Radiating boundary

When applying internal wave generation the wave energy propagating out of the model area has to be absorbed. This is done by placing sponge layers behind the generation line.

Remarks and Hints

Open Boundary or Internal Generation?
In most cases internal wave generation should be applied for wave generation, as open boundaries are fully reflective and will therefore reflect any wave energy radiating out of the model area back into the model.

5.3.3 Surface elevation

The initial surface elevation for your simulation can be specified either as a constant over the entire model area, or as a two-dimensional map read in from a data file.

The surface elevation option is only relevant in special cases where you want to model e.g. a landslide-generated wave or a tsunami wave or other type of transient wave phenomena. In such cases you can specify a spatial varying initial surface elevation map to model a "drop of a water mass".

For Boussinesq wave simulations it is essential that the initial surface elevation is specified relative to the still water level, SWL (bathymetry value + shift of reference level).
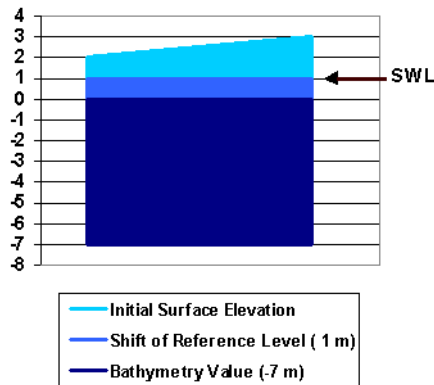

Figure 5.13   Sign convention of bed levels

5.3.4   Internal wave generation

Internal generation of waves is performed by adding the discharge of the incident wave field along the specified generation line. One of the advantages of using internal generation is that sponge layers can be placed behind the generation line, to absorb waves leaving the model area.

The wave data needed depends on the form of the Boussinesq equations, - that is: whether **deep water terms** are excluded or included.

- If the **deep water terms** are excluded you must specify the flux density perpendicular to the generation line for all grid points (along the line) and at each time step.

- If the **deep water terms** are included you must also specify the slope of the surface elevation perpendicular to the generation line in all grid points along the line at each time step.

To prepare the wave input data the wave tools in the MIKE 21 Toolbox should be applied.

You specify the number of internal generation lines to be used in your simulation, and then you edit each of the internal generation lines. First you must

define the location of the internal generation line in your model coordinate system by giving the grid coordinates for the first point and the last point of the line.

The internal generation line must be parallel to either the x- or y-axis.

Setup of internal generation lines

Using internal generation it is possible to model waves propagating with an angle to the generation line, and also directional (3D) wave fields.

The positions of the internal generation line(s) and combined sponge layers are dependent upon the incoming wave conditions - especially the wave direction and directional spreading.

One generation line
The use of directional waves requires a reasonably wide model area and a generation line depending on the specified directional distribution. This is illustrated in Figure 5.14.

The main direction of the waves is perpendicular to the generation line. If the bound of the directional distribution is $+\theta_{max}$ and $-\theta_{max}$, only the area within the thin red lines will contain all the wave components in the distribution limited by $+\theta_{max}$ and $-\theta_{max}$. Hence, the area of interest should be within this area. The useful model area is further restricted due to diffraction of waves into the shadow area outside the thin red lines.

Note, that the directional information on the waves is specified when generating the input wave files using the MIKE 21 Toolbox tool Random Wave Generation.
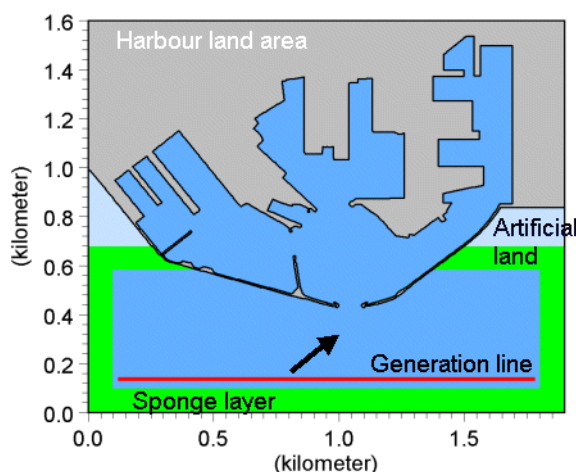

Figure 5.14   Model setup for modelling directional waves

The correct generation of the incident wave conditions can only be obtained on the assumption that the water depth is constant along the open boundary or the internal generation line. Therefore, the open boundaries and internal generation lines should be placed in areas where there is a small variation in the water depth.

The example Rønne Harbour uses one generation line.

Two generation lines

To model oblique waves, relative to the model grid a very wide generation line would be required if using only one generation line. Therefore two generation lines is preferred in many cases.

An example is shown in Figure 5.14, where it can also be seen that the area with artificial land on the right-hand side of the harbour entrance has been enlarged compared to the example above. In Figure 5.18 is shown how to join internal generation lines in a corner.
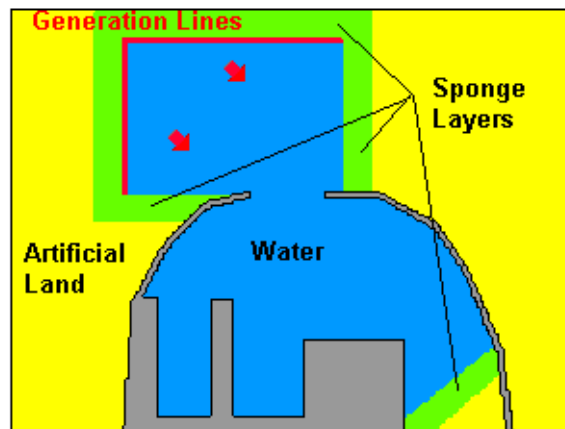

Figure 5.15   Model setup with two internal wave generation lines

The example Hanstholm Harbour uses two generation lines.

Position of generation line

Definition

The (internal) generation line should be parallel to either the x-axis or y-axis of your bathymetry. As illustrated in Figure 5.16, the waves are always sent off from the left hand side of the line as seen from the starting point, see also Figure 5.17. You should be careful only to send off waves to one side of the generation line, or in other words $\Delta\theta_{max}$ should be less than θ_{main}.

NOTE: The wave direction should always be related to the general (or global) x-axis of your model bathymetry and not the internal generation line itself.

Two or more generation lines
Figure 5.18 shows how to join internal generation lines in a corner.

Grid spacing
You must also specify the grid spacing in the x- and y-directions corresponding to the bathymetry used for the Boussinesq wave simulation.
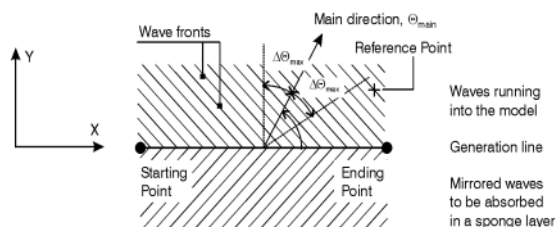

Figure 5.16   Definition of main wave direction θ_{main} and maximum deviation from main wave direction $\Delta\theta_{max}$. Please note that the co-ordinate system refers to the general (or global) co-ordinate system.
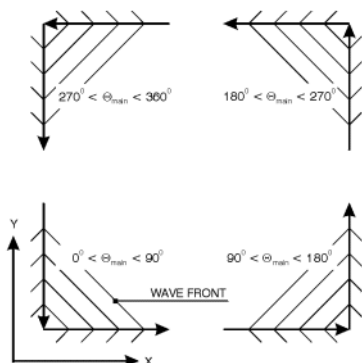

Figure 5.17   Definition of generation lines and main wave directions θ_{main} , when using two generation lines to generate a wave field.
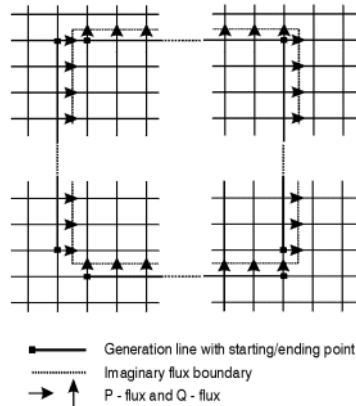
Figure 5.18   How to join internal generation lines in a corner

Examples of wave fields

MIKE 21 BW can model regular as well as irregular waves.

Normal incident irregular waves

Irregular waves can be applied both using internal wave generation and an open boundary in case of normal incidence, i.e. for waves having an angle of 90 degrees relative to the generation line/open boundary. In both cases a .dfs0 data file is used as input. An example is shown in Figure 5.19.
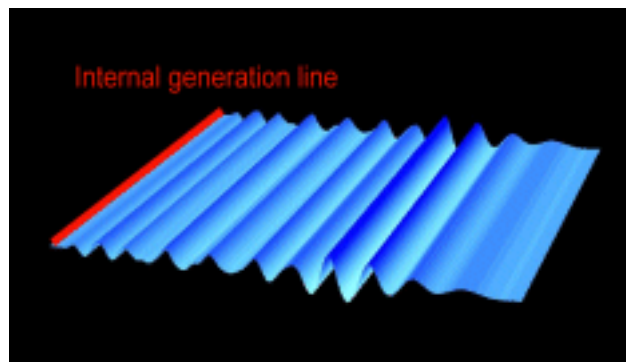

Figure 5.19   Boussinesq wave simulation using one-dimensional wave generation input (dfs0), see the MIKE 21 Toolbox tool Random Wave Generation

Oblique incident irregular waves

For oblique incident unidirectional irregular waves, i.e. waves having an angle different from 90 degrees relative to the generation line, only internal wave generation is possible. In this case a .dfs1 line series data file is used as input. An example is shown in Figure 5.20.
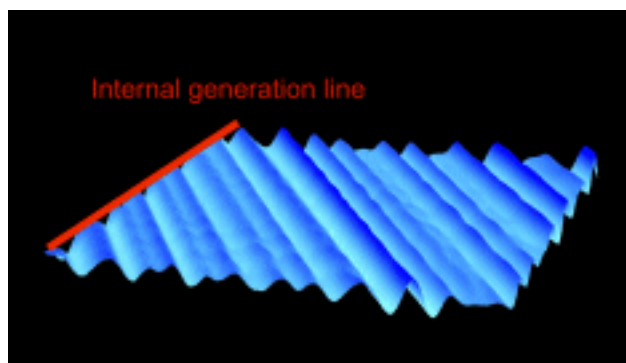
Figure 5.20   Boussinesq wave simulation using unidirectional wave generation input (dfs1), see the MIKE 21 Toolbox tool Random Wave Generation

Incident directional irregular waves

For incident directional irregular waves, i.e. waves having an angle different from 90 degrees relative to the generation line, only internal wave generation is possible. In this case a .dfs1 line series data file is used as input. An example is shown in Figure 5.21.
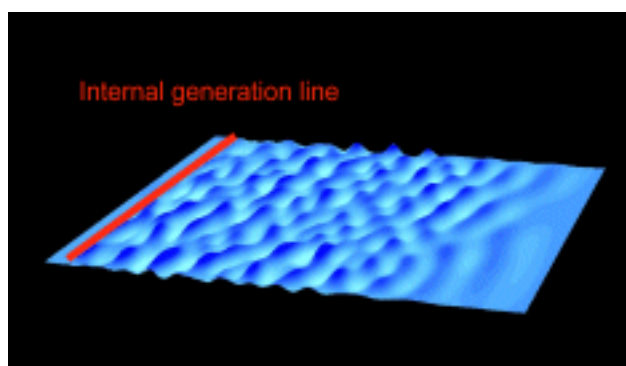

Figure 5.21   Boussinesq wave simulation using directional wave generation input (dfs1), see the MIKE 21 Toolbox tool Random Wave Generation

5.3.5   Bottom friction

Short periodic waves

Usually the effects of bottom friction are relatively unimportant in simulation of short waves in ports and harbours. This is because the area covered by most short wave models is relatively small (typically less than a few square km's), and with the exception of high waves and/or very shallow water there is usually not a sufficient distance for the bed resistance to attain any significant effect on short wave propagation. In these applications the bed resistance can usually be excluded, without the need for detailed evaluations.

Long periodic waves

For modelling of long wave transformation (e.g. harbour resonance and seiching) the effect of bottom friction may be important.

Wave-induced currents

As opposed to the shore-normal case (1DH applications) steady solutions for the wave-induced mean flow can only exists when the forcing by radiation stress is balanced by bottom friction and mixing processes. Therefore you need to include bottom friction in simulations of wave-induced flow fields, see examples in Section 4.2.6 (Island), Section 4.2.7(Rip channel) and 4.2.8 (Detached breakwater).

The bottom friction can be formulated using

- Chezy number formulation
- Manning number formulation

Chezy number

The bottom friction formulation used in MIKE 21 BW has been set up using the Chezy bed friction rule.

With the Chezy bed friction rule, the shear stress at the bed can be expressed in terms of the Chezy number, C, as

$$\tau_b = \frac{\rho g U |U|}{C^2} \tag{5.1}$$

where U is the depth-averaged velocity, ρ is the water density, and g is gravity. The unit of the Chezy number is $m^{1/2}/s$.

The Chezy number may be approximated by the following expression:

$$C = \frac{U}{U_b}\sqrt{\frac{2g}{f_w}} \tag{5.2}$$

where U_b is the velocity at the bed, and f_w is the wave friction factor appearing in the bottom shear stress.

For small waves the wave friction factor may be determined by:

$$f_w = \exp(-5.977 + 5.213(a_b/k_N)^{-0.194}) \tag{5.3}$$

where a_b is the amplitude of the wave particle motion at the bed, and k_N is the bed roughness parameter (Svendsen and Jonsson (1980)).

Recommended values

The relationship between the depth averaged velocity, U, and the velocity at the bed, U_b, can vary from unity by as much as 30% for high waves and 50% for high waves in relatively deep water.

Example

Wave height, H = 2.0 m; wave period, T = 8.0 s; water depth, h = 10.0 m; and roughness height, k_N = 0.001 m (a_b/k_N = 1000). With U/U_b = 1.3, we get f_w = 0.01, which corresponds to an equivalent Chezy number of C = 58.

Manning number

You can convert a Chezy number, C, to a Manning number, M, using the following relationship

$$M = \frac{C}{h^{1/6}} \qquad (5.4)$$

where the Manning number has the unit [$m^{1/3}$/s], and h is the water depth.

Please note that the Manning number used in MIKE 21 BW is the reciprocal value of the Manning number described in some hydraulic text books.

5.3.6 Eddy viscosity

The decomposition of the prognostic variables into a mean quantity and a turbulent fluctuation leads to additional stress terms in the governing equations to account for the non-resolved processes both in time and space. By the adoption of the eddy viscosity concept these effects are expressed through the eddy viscosity and the gradient of the mean quantity. Thus the effective shear stresses in the momentum equations contain the laminar stresses and the Reynolds stresses (turbulence).

The eddy viscosity can be specified in three different ways:

- No eddy
- Constant eddy formulation
- Spatial varying formulation

The eddy viscosity is calculated using two different formulations:

- Eddy coefficient
- Smagorinsky coefficient

For the case with values varying in domain you have to prepare a data file containing the eddy viscosity coefficient or the Smagorinsky coefficient before

you set up the Boussinesq wave simulation. The file must be a 2D grid data file (dfs2).

You also have to choose between velocity (default) or flux based eddy viscosity formulations.

Recommended values

It is recommended to use a Smagorinsky velocity-based formulation with constant coefficient of 0.25-1.

It is recommended to use the velocity formulation as the flux formulation is only correct at a constant depth and in general should be applied with great care in order to avoid falsification of the flow pattern. The velocity based formulation, which is more correct, is unfortunately also more difficult to implement in the numerical algorithm. This is because the model uses the fluxes as the unknown parameters and not the velocities. Therefore the velocity-based formulation is implemented by using the velocities from the previous time step. This can, however, lead to stability problems when the eddy viscosity coefficient becomes large. The eddy coefficient must fulfil the criterion

$$\frac{v \cdot \Delta t}{\Delta x^2} \leq \frac{1}{2} \qquad (5.5)$$

It is **not** recommended to use eddy viscosity to damp out numerical instabilities in MIKE 21 BW simulations.

Remarks and hints

Eddy viscosity is mainly introduced in MIKE 21 BW for modelling of wave-current interaction, where sub grid effects in the current field are not resolved. Please also note that a wave-induced flow field can only exist when the forcing by radiation stress is balanced by bottom friction and mixing processes. Mixing processes are usually modelled by means of an eddy viscosity. In the breaking zone, the viscous shear is to some extent taken into account by the additional convective terms due to the surface roller. Outside the breaking zone, the eddy viscosity is usually very small. Therefore eddy viscosity should usually not be used in connection with modelling of wave-induced currents.

5.3.7 Filtering

An explicit numerical lowpass filter is introduced to remove high-frequency instabilities during uprush and downrush and to dissipate the wave energy in the area where the surface roller can not be resolved.

If wave breaking and moving shoreline is included in your simulation you most often also have to specify a data file including filter coefficients. Near the

still water shoreline it is recommended to use a filter coefficient between 0 and 1, e.g. 0.25 or 0.5. Outside this area the filter coefficient should be zero (no filtering).

Figure 5.22 shows the position of a non-zero filter applied in one of the sloping beach examples in the installation.
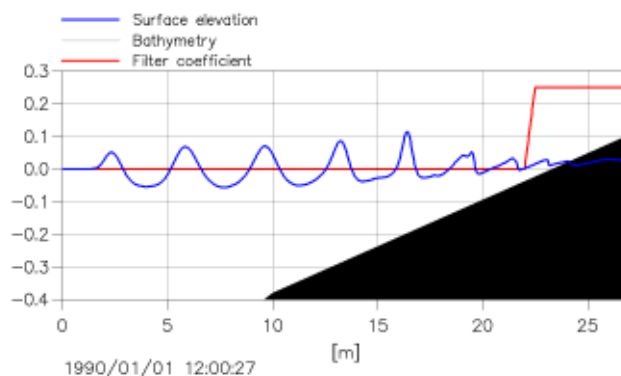

Figure 5.22   Application of a spatial varying filter coefficient in the uprush/downrush zone.

How to prepare a filter file?

The data file including the filter coefficients (1DH applications: dfs1-file and 2DH applications: dfs2-file) can easily be prepared by making a copy of the bathymetry/profile file: replace values within a certain depth range (say, -3 m to +1 m) with a constant filter value (say, 0.25) and change remaining depth values to zero. This manipulation can be undertaken in the Grid Editor (dfs2) or in the Profile Editor (dfs1). To obtained a smooth transition between "0" and "0.25" it is recommended to delete some "0"s around the area with filter coefficient 0.25 and subsequently make an interpolation.

5.3.8   Wave Breaking

The incorporation of wave breaking in MIKE 21 BW is based on the concept of surface rollers.

Surface roller concept

The wave breaking is assumed initiated if the slope of the local water surface exceeds a certain angle, in which case the geometry of the surface roller is determined. The roller is considered as a passive bulk of water isolated from the rest of the wave motion, while being transported with the wave celerity. The influence of the roller is taken into account through an additional convective momentum term arising from the non-uniform vertical distribution of the horizontal velocity. The approach is illustrated in the figure below. For a thorough description of the determination of the surface rollers, reference is

made to Schäffer et al (1993) and Madsen et al (1997a). The latter paper is included in the scientific documentation. Although the breaker model is not designed for to handle plunging breakers, successful results have been obtained even with the default breaker parameters listed below.
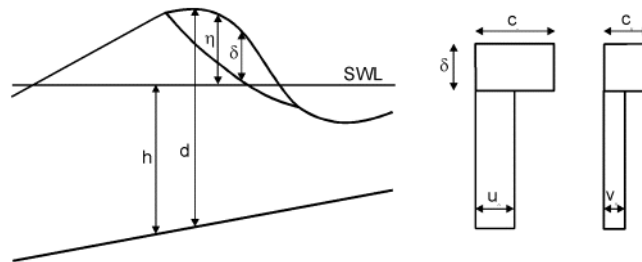

Figure 5.23   Cross-section of a breaking wave and the assumed vertical profile of the horizontal velocity components

Recommended wave breaking parameters

The surface roller for a breaking wave is calculated using the following parameters:

- Roller form factor

    The roller thickness is determined as the water above the tangent of slope and the resulting thickness is multiplied by a form or shape factor. The default value is 1.5. For plunging breakers (see section Type of breakers) Ozanne et al (2000) suggest a value of about 2.0.

- Type of roller celerity

    The roller celerity is assumed to be proportional to the linear shallow water celerity. The roller direction can either be determined interactively from the instantaneous wave field (type 1) or set to a prescribed wave direction (type 3). The first procedure (type 1) may sometime cause stability problems why the second type is usually recommended. However, in some applications the wave propagation direction cannot be set to a characteristic direction as e.g. in the island examples, see Section 4.2.6.

- Roller celerity factor

    For the type 1 of roller celerity the default factor is 1.3.

- Initial breaking angle

    Breaking is predicted to occur when the local slope of the surface elevation exceeds the initial breaking angle. The default value is 20 degrees.

- Final breaking angle

    During the transition from the initial breaking to a bore-like stage in the inner surf zone, the critical angle is assumed to change gradually from the initial breaking angle to a smaller terminal angle, the final breaking angle. The default value is 10 degrees.

- Half-time for cut-off roller

    This time defines the transition between the two breaker. The default value is Tp/5, where Tp is the wave period of the most energetic wave components.

- Wave direction

    This direction is the main or characteristic wave propagation direction of the breaking waves relative to true North. The default direction is 270ºN corresponding to waves from West.

Type of breakers

Depth-induced wave breaking of individual waves starts when the wave height becomes greater than a certain fraction of the water depth. As a rule of thumb, the wave height of an individual wave at breaking is often said to be around 80% of the water depth, but this is a very approximative number.

Breaking waves are generally divided into three main types, depending on the steepness of the waves and the slope of the shoreface:
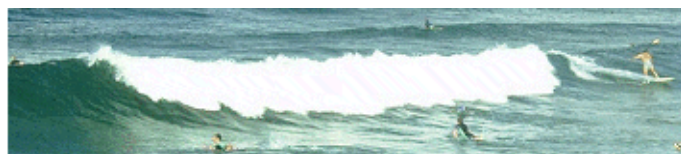

Figure 5.24   Spilling

Spilling takes place when steep waves propagate over a flat shoreface. Spilling breaking is a gradual breaking process which takes place as a foam bore on the front topside of the wave over a distance of 6-7 wave lengths.

Figure 5.25   Plunging

Plunging is the form of breaking where the upper part of the wave breaks over its own lower part in one big splash whereby most of the energy is lost. This form of breaking takes place in case of moderately steep waves on a moderately sloping shoreface.

Figure 5.26   Surging

Surging is when the lower part of the wave surges up on the shoreface in which case there is hardly any surf zone. This form of breaking takes place when relatively long waves (e.g. swell) meet steep shorefaces.

White-capping or top-breaking is steepness-induced wave breaking which occurs on deeper water when the wave height becomes to large compared to the wave length. White-caps can be observed in the figure below.

Figure 5.27   White caps in open water

5.3.9   Moving shoreline

The incorporation of a moving shoreline in MIKE 21 BW is based on the following approach: the computation domain is extended artificially by replacing the solid beach by a permeable beach characterised by a very small porosity. Near the moving shoreline the water surface will insect with the sea bed and continue into the porous beach. Hence the instantaneous position of the shoreline is simply determined by this intersection. Please read pp. 263-266 in Madsen et al (1997a) included in the scientific documentation.

Moving shoreline using slot technique

The vertical cross section comprises a physical regime with clear water on top of an artificial porous flow regime. The idealised vertical variation of the porosity is unity in the physical domain and a very small porosity in the porosity regime below. In order to avoid numerical instabilities it is necessary to introduce a transition between the two regimes. Here an exponential transition has been used. For a thorough description of the determination of the moving shoreline using slots reference is made to Madsen et al (1997a). This paper is included in the scientific documentation.

Recommended moving shoreline parameters

The moving shoreline is calculated using the following parameters:

- Slot depth

    In practice the slot depth is normally chosen as the datum of the toe of the slope. It is important to keep slot so deep that it is not dried out during a down-rush.

- Slot width

    In principle the slot width (or the minimum porosity) should be as small as possible in order to avoid a distortion of the mass balance and a disturbance of the flow in the physical domain. On the other hand, the numerical solution becomes unstable for extreme values of the slot width. In practice, it turns out that the slot width should be chosen in the interval of 0.01 to 0.001. For steep slopes an even higher slot width is sometimes required. Too high values of the slot width may results in an underestimation of the run-up.

- Slot smoothing parameter

    In principle the slot smoothing parameter should be as large as possible in order to avoid a distortion of the mass balance and a disturbance of the flow in the physical domain. On the other hand, the numerical solution becomes unstable for extreme values of the slot smoothing parameter. In practice, it turns out that the parameter should be order of 100. For steep slopes an a small slot smoothing parameter is sometimes required. Too small values of the parameter width may results in an underestimation of the run-up.

- Slot friction coefficient

    In most applications this parameters should be set to zero. Non-zero values may damp out high-frequency noise, but may also end up in an underestimation of the run-up.

Please note
Numerical modelling of a moving shoreline is a very difficult task. You may sometimes see numerical instabilities and eventually blow-up in some of your

applications. In such situations we recommend you to introduce a slot friction coefficient, e.g. 0.01-0.1. You may also increase the width and/or the depth of the slot.

5.3.10 Porosity layers

Porosity values are used to model either partial reflection and/or transmission through structures. If porosity values are backed up by land, partial reflection will take place. Conversely, (partial) transmission will also take place if the porosity values are not backed up by land points.

The porosity values at open water grid points should be set to unity (i.e. porosity = 1.0).

The porosity should only be set to less than unity along structures where you want to include the dissipation effect of porous flow.

2DH Boussinesq wave module

In the figure below is shown an example where a porosity layer is applied to model partial reflection in case of a wave disturbance study.
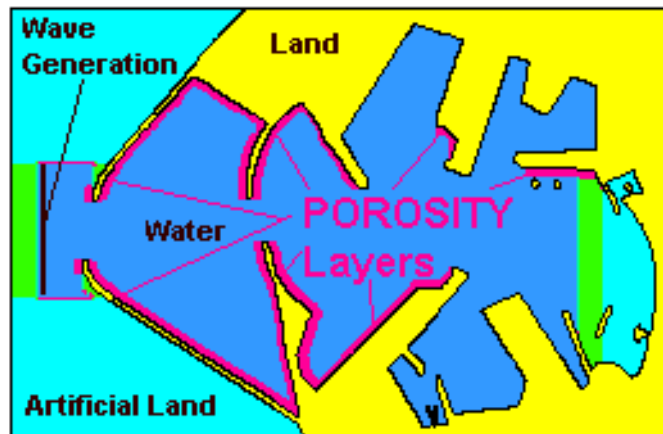
Figure 5.28   Example of porosity map

The width of the porosity layer should normally be at least 1/4 of a wave length (for 1 DH application too) corresponding to the most energetic waves. In 2D applications this most often corresponds to a three to five grid points wide layers.

The water depth in the porous layer should be almost uniform.

1DH Boussinesq wave module

The figure below shows an example where a porosity layer is applied to model partial reflection in case of a numerical wave flume, see Section 4.3.1.
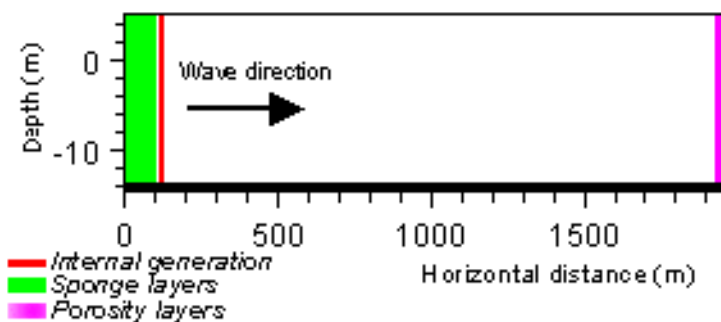

Figure 5.29   Model setup (vertical cross-section). An 8 point wide porosity layer is used at the left-hand side boundary

Specifying porosity layer

For a given set of porous flow parameters (see section Recommended porosity layer parameters below), the porosity values required to obtain a desired reflection coefficient, can be determined by the use of the MIKE 21 Toolbox program **Calculation of Reflection Coefficients**.

Important! The porosity should never be smaller than 0.2.

For impermeable structures, the porosity layer should be backed by an impermeable land grid point.

Recommended porosity layer parameters

When determining the porosities to be used in your simulation, the values of the porous flow parameters are not that important. Unless you have a specific reason for using other values, these parameters can usually be left at their default values, which are:

- Kinematic viscosity: $1.4 \cdot 10^{-6}$ m^2/s
- Stone diameter: 0.2 m
- Laminar resistance parameter: 1000
- Turbulent resistance parameter: 2.8

For uniform particle sizes, Engelund (1953) suggests the following range of values for the laminar and turbulent resistance coefficients:

	Laminar	Turbulent
Spherical particles	780	1.8
Rounded particles	1000	2.8
Angular particles	up to 1500	up to 3.6

Porosity - scientific background

For Boussinesq wave simulations, the equations in MIKE 21 BW have been modified to include porosity and the effects of non-Darcy flow through porous media. In this way, it is possible for MIKE 21 to model partial reflection, absorption and transmission of wave energy at porous structures such as rubble mound breakwaters (see e.g. Madsen and Warren (1984) and Madsen (1983)).

The main effects of porosity are introduced by additional laminar and turbulent friction terms for describing losses due to flow through a porous structure. In most practical cases the pore sizes are relatively large (typically 0.1 m to 1.0 m), and the turbulent losses will dominate. The laminar loss term has also been included to allow the simulation of small scale physical model tests.

The flow resistance inside the porous structure is described by the non-linear term

$$(\alpha + \beta|U|)U \tag{5.6}$$

where α and β account for the laminar and turbulent friction loss, respectively, and U is the velocity. α and β are determined by the empirical expressions recommended by Engelund (1953):

$$\alpha = \alpha_0 \frac{(1-n)^3}{n^2} \frac{\nu}{d^2} \tag{5.7}$$

$$\beta = \beta_0 \frac{(1-n)}{n^3} \frac{1}{d}$$

where:

$n =$ porosity (pore volume/unit volume)
$\nu =$ kinematic viscosity of water
$d =$ a characteristic diameter of the structure units (the grain size)
$\alpha_0 =$ a laminar particle form resistance coefficient
$\beta_0 =$ a turbulent particle form resistance coefficient

Porosity values are used to model either partial reflection and/or transmission through structures. If porosity values are backed up by land, partial reflection will take place. Conversely, (partial) transmission will also take place if the porosity values are not backed up by land points.

The porosity values at open water grid points should be set to unity (i.e. porosity = 1.0).

The porosity should only be set to less than unity along structures where you want to include the dissipation effect of porous flow.

Recipe for generation of porosity maps

An effective procedure for establishment of porosity maps in connection with application of MIKE 21 BW 2DH in wave disturbance analysis is outlined below.

Step 1

Open your bathymetry file (dfs2-file) and change the land value (e.g. 5) along **all** structures having partial reflective properties to another value/code (e.g. 3). This is illustrated in the figure below (for a sub-area of the bathymetry) in a case with a pile-type quay structure, where partial reflection will occur.
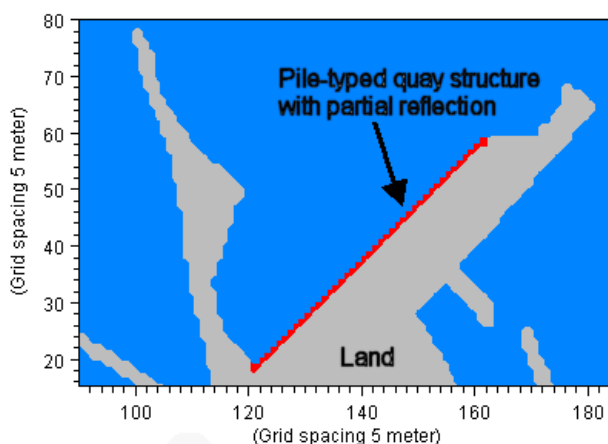
Figure 5.30   The bathymetry land value next to a water point is changed to a value/code representing stretches with partial reflection

Step 2

The porosity value required to obtain the desired reflection coefficient (in this case approximately 0.4) is determined using the MIKE 21 Toolbox programme Calculation of Reflection Coefficient as shown in Figure 5.31. In this example we assume the wave and depth conditions along the quay can be considered as almost uniform ($H_{m0} \sim 0.2$ m, $T_p \sim 8$ s and $h \sim 8$ m). An absorber width of 15 m (3 grid points wide porosity layer) results in a porosity of $n \sim 0.5$.
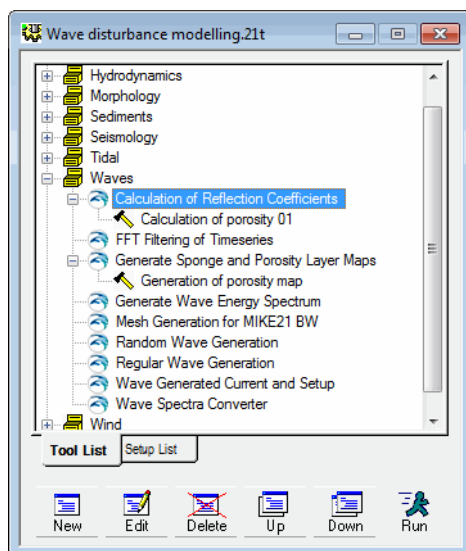
Figure 5.31   The MIKE 21 Toolbox programme Calculation of Reflection Coefficients is used to estimate the porosity value to be used in MIKE 21 BW.

Step 3

The next step is to establish the porosity map. Most often this is generated using the MIKE 21 Toolbox programme **Generate Porosity Map** as shown in Figure 5.31. In this tool you specify the calculated porosity value of 0.5. The resulting map is shown in Figure 5.32, which can be used directly in MIKE 21 BW 2DH.
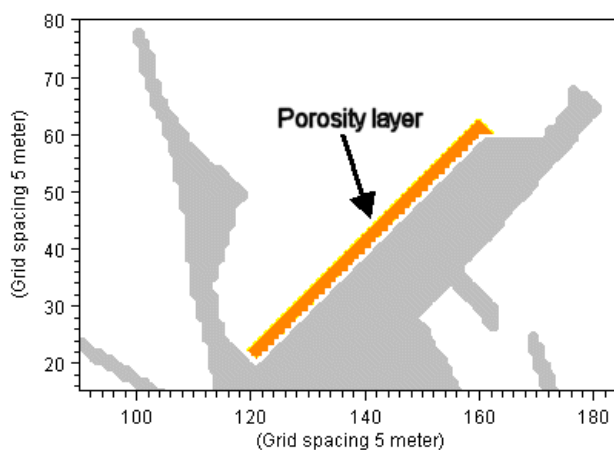

Figure 5.32   Example of porosity map for modelling of partial reflection along an open pile-type quay

Step 4

If other parts of your model area include porous structures the generated porosity map may look like the map shown in Figure 5.33. As the wave and depth conditions typically vary inside a harbour most often you have to use different values of porosity along the porous structures. You simply divide your entire model area into a number of sub-areas characterised by having reasonable uniform wave and depth conditions (typically 10-20 sub-areas). For each of these sub-areas a porosity value is determined using the MIKE 21 Toolbox programme Calculation of Reflection Coefficient. For each of the defined sub-areas the original porosity (0.5 in this example) is replaced by the new and more representative value using the Grid Editor.
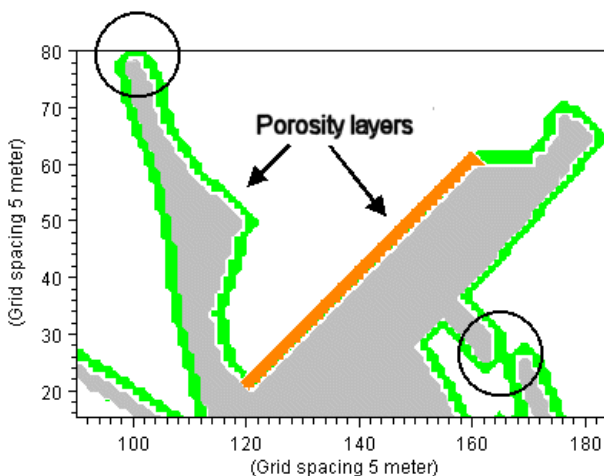
Figure 5.33   Example of a porosity map covering other areas where partial reflection will take place

Step 5

Finally, you ensure that the overall porosity map looks "OK". For harbour entrance areas as illustrated in Figure 5.33 (by the circle) it is recommended to reduce or remove the porosity layer as illustrated in Figure 5.34 (using the Grid Editor). Otherwise the wave penetration may be underestimated. This is particularly the case for small entrance widths.
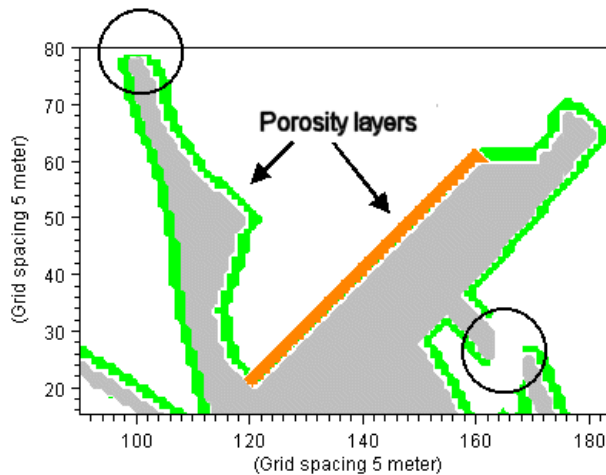
Figure 5.34   Example of porosity map correction at harbour entrance areas.

Hint

In order to obtain a quick rough estimate of the wave disturbance inside a harbour it is a good idea to run a simulation using a porosity map with a constant porosity along all porous structures. Such a map can usually be established within less than one man-hour. The results of this simulation (wave height distribution) can be used subsequently for an improved estimate of the porosity.

Wave reflection

Wave reflection from e.g. structures, or a beach, can be modelled by applying porosity values (less than 1) at the grid points representing the structures. If no porosity values are specified, the reflection will be 100% from land points.

As the porosity results in dissipation of energy, the reflection may be controlled by the porosity values and the number of porosity layers.

If the porosity values are not backed up by land values, the porosity layers will result in both reflection and transmission through the layers representing e.g. a permeable breakwater.

If no reflection is wanted, sponge layers should be applied - absorbing all wave energy entering the layers.

What is a typical value of the reflection coefficient?

Typically, the reflection coefficient for a rubble mound structure is 30-50%. More information about reflection, absorption and transmission characteristics can be found in Thompson et al (1996). Figure 5.35 shows typical reflection coefficient ranges for short period waves.
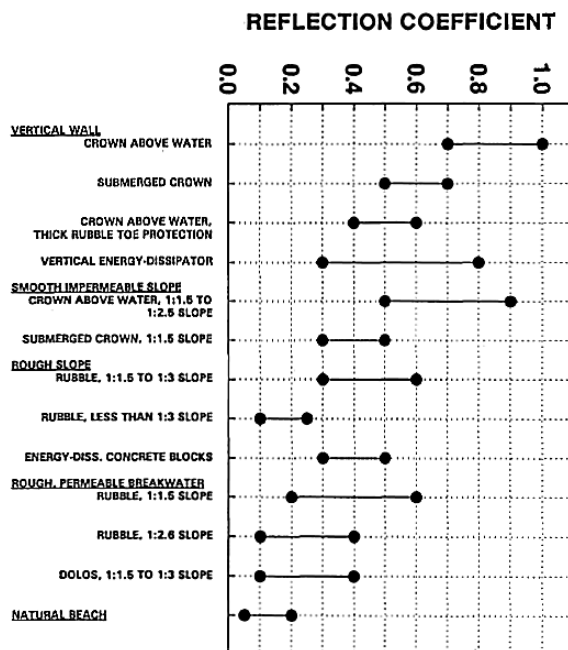
Figure 5.35   Typical reflection coefficient ranges for short period waves (T<20s).
               From Thompson et al (1996)

Transmission through Structures

Transmission through porous structures are modelled by applying porosity values at grid points representing the structures.

To determine the porosity values and the number of porosity layers to represent a given transmission coefficient the MIKE21 Toolbox tool **Calculation of Reflection Coefficients** should be used.

5.3.11 Sponge layers

Sponge (or absorbing) layers can be used as efficient numerical wave absorbers in Boussinesq wave simulations. These may e.g. be set up along model boundaries to provide radiation boundary conditions, which absorb wave energy propagating out of the model area.

2DH Boussinesq wave module

In the figure below sponge layers are used to model a fully absorbing beach and to absorb waves radiating out of the harbour area.
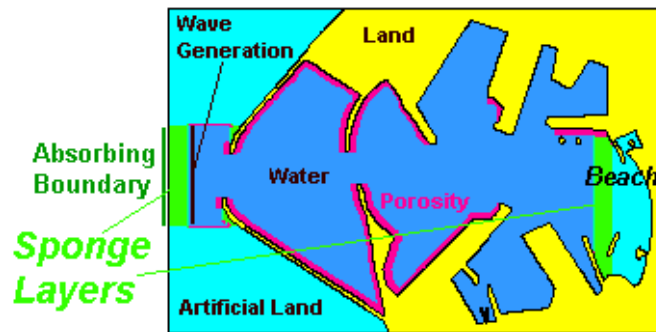
Figure 5.36   Example of a sponge layer map

1DH Boussinesq wave module

In connection with modelling of non-linear wave transformation on coastal profiles sponge layers are usually applied at the two model domain extremes. As for 2D applications the sponge layers at $j= 0$ are used for absorption of wave energy propagation out of the model domain. The purpose of having sponge layers at the other extreme (i.e. at $j= jextr$) is to either absorb incoming waves in areas where no computation is requested (see left-hand side figure below) or to reduce oscillations in the slot (applied in connection with Moving shoreline, see right-hand figure below).
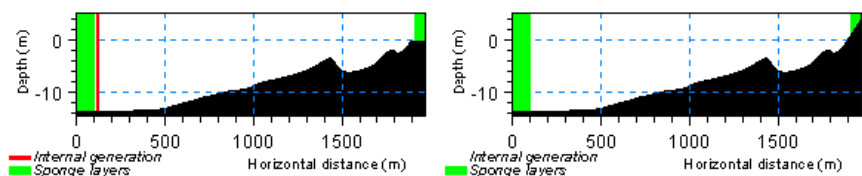
Figure 5.37   Example of sponge layer profiles. The panel to the left shows a profile without moving shoreline and the right panel shows profile including moving shoreline.

Specifying sponge layer

You can choose not to include sponge layers, or to specify the sponge layers by providing your model setup with a map/profile including sponge layer coefficients, in much the same way as you specify porosity values. The map/profile is read from a data file (dfs2 or dfs1).

General guidelines for the preparation of this map/profile are as follows:

- very good absorbing characteristics are obtained for a sponge layer width of one to two times the wave length corresponding to the most energetic waves

- for typical short-wave studies (grid spacings of 3-10 m) the sponge layers should be at least 20 lines wide

- for 2D studies involving longer waves (e.g. harbour resonance studies) sponge layers may need to be 50 or even more grid lines wide

- in most wave transformation applications (say, mesh size <1 m) the number of sponge layers is usually within the range 50-200 grid lines

- the sponge layers should normally be backed by a closed boundary (i.e. land grid points)

- the sponge layer coefficients at open water grid points should always be set to unity (i.e. sponge coefficient = 1.0)

- to minimise reflections, the values of the sponge layer coefficients should be close to unity along the first grid line, and should increase smoothly towards the boundary/land

Given the guidelines above, you are free to select your own values for the sponge layer (damping) coefficients.

Tool for generation of files including sponge layers

Most often you will use the MIKE 21 Toolbox program **Generate Sponge and Porosity Layer Maps** for generation of sponge layers:

Recommended sponge values

When selecting the sponge layer coefficients, the following formula has been found to work well:

$$C_{sponge} = a^{(r^{i-1})}, \quad i = 1, N_{sponge} \tag{5.8}$$

where a and r are assigned constant values, and N_{sponge} is the number of sponge lines (counting from the closed boundary towards the first grid line).

The implemented method is based on the sponge layer technique introduced by Larsen and Dancy (1983).

For twenty grid lines, it is recommended to use a = 7, and r = 0.7. The sponge layer coefficients are then obtained by setting the value at the line closest to the closed boundary equal to a (the base value) and successively taking the

square root for the subsequent points. The following values are obtained:

Grid Point	Sponge Value
Land Point	Land Point
1	7
2	3.9045
3	2.5947
4	1.9492
5	1.5955
6	1.3868
7	1.2572
8	1.1738
9	1.1187
10	1.0816
11	1.0565
12	1.0392
13	1.0273
14	1.0190
15	1.0132
16	1.0092
17	1.0064
18	1.0045
19	1.0031
20	1.0022
Open Water	1

Remarks and hints

An absorbing sponge layer damps the waves (per default) relative to the still water level (surface elevation = 0) and zero flux. If you require a radiation boundary with another reference value, you can specify the relevant level (most often you only specify a surface elevation) through an option:

Sponge_Reference_s = 0 (default)
Sponge_Reference_p = 0 (default)
Sponge_Reference_q = 0 (default)

Please see Section 5.5.15 Option parameters for how to use an OPTION.

5.4 Output Parameters

Deterministic parameters
The basic results from a MIKE 21 BW simulation consist of arrays containing the total water depth, h, the depth-integrated velocity, P, in the x-direction, and the depth-integrated velocity, Q, in the y-direction. For the 1DH module an auxiliary variable, w, is saved as well. These dependent variables are available at the end of every time step of your simulation. However, you can also select the corresponding water surface elevations as output directly.

Phase-averaged parameters
Based upon the three dependent variables a large number of phase-averaged parameters are calculated, e.g. maximum wave height, wave crest parameters, wave setup and wave-induced velocities. Also a number of higher order measures such as skewness, kurtosis and atiltness is calculated. The temporal averaging can be performed on cumulative or subseries basis including or excluding the swash zone.

Wave disturbance parameters
A commonly used output type from wave simulations is the wave disturbance or relative wave height. In addition to maps of wave disturbance coefficient you can perform detailed statistical analysis for a number of user-defined areas within the computational domain.

Moving shoreline parameters
For MIKE 21 BW 1DH applications including moving shoreline you can save time series of the horizontal, vertical and total run-up and subsequently perform statistical analysis of these data.

Hot start parameters
Hot start data (2DH module only) can be saved. The hot start file contains all necessary information to continue a simulation.

5.4.1 Deterministic parameters

In this output dialog you define the number of output files. The specification of the individual output files is made subsequently. For each output file you specify which type of data you would like to store, the temporal/spatial range and frequency as will as a name and title of the file.

Output type

You can choose between following output types:

- Type 0: point series data (dfs0 file)
- Type 1: line series data (dfs1 file)
- Type 2: grid series data (dfs2 file)

For point series data (type 0) you can choose a number of different points in each file.

You can view the results easily using the relevant MIKE Zero tools by clicking on the "View" button or by e.g. presenting the results using Plot Composer as in Figure 5.38.
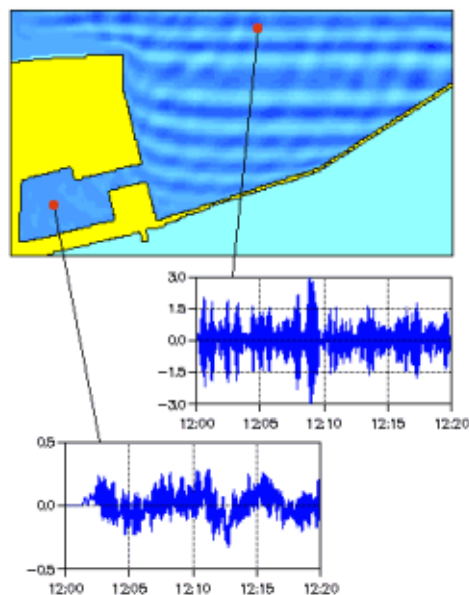

Figure 5.38   Example of 2D map and time series of surface elevation in two selected points

Output parameters

Figure 5.39   The list of deterministic output parameters includes a number variable for easy visualization of e.g. the wave breaking process

You can choose following output parameter types:

- Surface elevation
- Water depth
- P flux
- Q flux
- Still water depth
- Auxiliary variable (1DH module only)
- Roller thickness (2DH module only)
- Roller angle (2DH module only)
- Roller celerity, c_x (2DH module only)
- Roller celerity, c_y (2DH module only)
- Water level (roller) (2DH module only)

Surface elevation

The surface elevation is a basic model parameter and should always be included in your model specifications. It is calculated on basis of computed water depth and bathymetry. You can use this item for 3D visualisation of the instantaneous surface elevation (see e.g. Figure 5.40) using MIKE Animator Plus.

Water depth

The water depth is a prognostic variable similar to the P flux and Q flux.
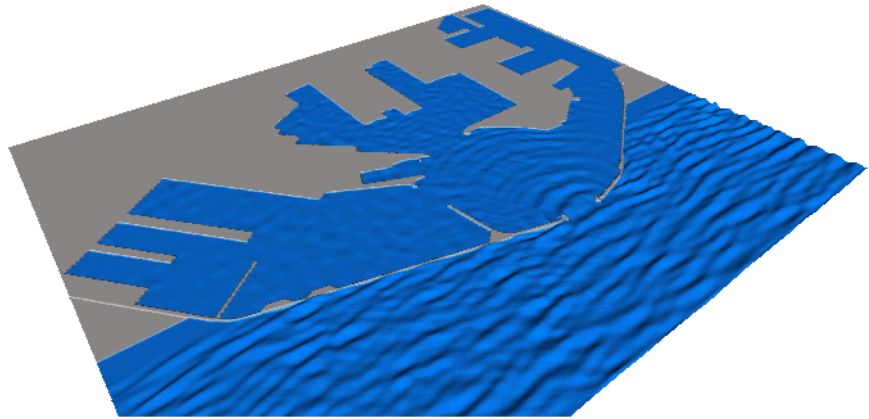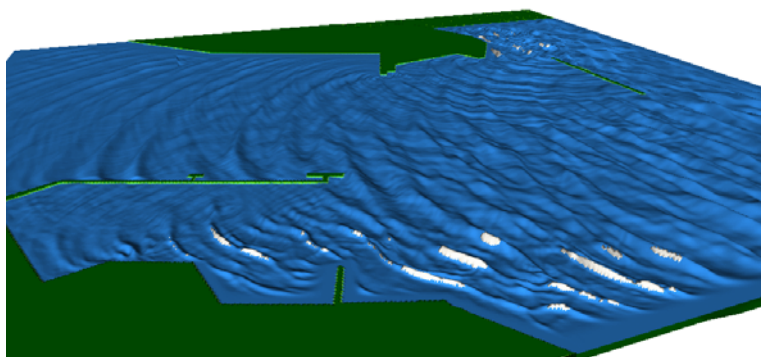
Figure 5.40   Visualisation (3D) of instantaneous surface elevation using MIKE Animator Plus, where input is a dfs2 file including the surface elevation.

P flux
The prognostic variable P flux is the depth-integrated flux density in x-direction.

Q flux
The prognostic variable Q flux is the depth-integrated flux density in y-direction.

Still water depth
The still water depth is identical to the bathymetry and will not change in time. For result presentation of 1DH applications it is often useful to include this parameter in the output.

Auxiliary variable (1DH module only)
This prognostic variable is relevant to save in connection with debugging model results. The auxiliary variable is defined as

$$w = \frac{\partial}{\partial x}\left(d\frac{\partial \eta}{\partial x}\right) \tag{5.9}$$

where η is the surface elevation.

Roller thickness (2DH module only)
The roller thickness can be interpreted as a measure of the intensity of wave breaking. Outside the surf zone the roller thickness is zero.
The definition of the roller thickness is given in Sørensen et al (2004) which is included in the scientific documentation.

Roller angle (2DH module only)
When wave breaking is included you can save the roller angle. The concept of surface rollers and definition of the roller angle is given in the scientific doc-

umentation. The roller angle is set to "initial breaking angle" (default 20º) before breaking and to "final breaking angle" (default 10º) after breaking. Usually you would not save this information.

Roller celerity, c_x (2DH module only)

The roller celerity in the x-direction is calculated as described in the scientific documentation. Usually you would not save this information.

Roller celerity, c_y (2DH module only)

The roller celerity in the y-direction is calculated as described in the scientific documentation. Usually you would not save this information.

Water level (roller) (2DH module only)

The water level including rollers is mainly used in connection with 3D visualisation of the instantaneous surface elevation and surface rollers (see e.g. Figure 5.41) using MIKE Animator Plus. For generation of 3D images and animations you have to save the water level and water level (roller) into two separate dfs2 files. You may use the MIKE Animator Plus setup included in the examples as an template.

Figure 5.41   Visualisation (3D) of instantaneous surface elevation and surface rollers using MIKE Animator Plus, where input is two dfs2 files including the water level and water level (roller), respectively
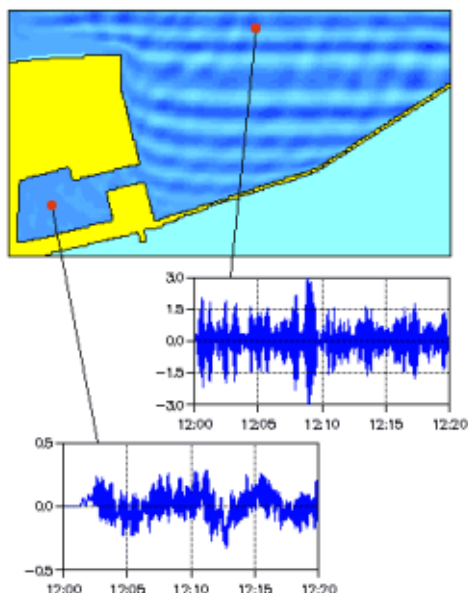
Figure 5.42   Example of 2D map and time series of surface elevation in two selected points

Remarks and hints

When you specify the spatial range of your output data you also have to specify the number of points you would like to save, N_{points}. The output result is interpolated to a grid having the constant spatial resolution of $(J_{last}-J_{first}+1)/N_{points}$.

In case you use a **structured mesh** of the bathymetry we recommend you to use the suggested default value of N_{points} $(=J_{last}-J_{first}+1)$. For **unstructured meshes** (1DH application only) we recommend you to use a larger value of N_{points}, in order to have a high resolution in the surf and swash zone. Further it is recommended (for unstructured meshes) to save the entire profile of data, i.e. keep the default values of J_{first} and J_{last}.
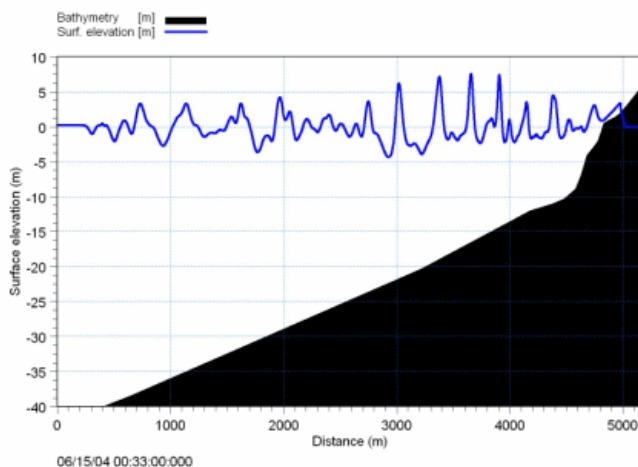
Figure 5.43   Example of line series of surface elevation on a coastal profile modelled by MIKE 21 BW 1DH

5.4.2   Phase-averaged parameters

In this output dialog you define the number of output files. The specification of the individual output files is made subsequently. For each output file you specify which type of data you would like to store, the temporal/spatial range and frequency as will as a name and title of the file.

Figure 5.44   The list of phase-averaged output parameters includes various variables for detailed analysis and presentation

Wave statistics

You can select between following two types of wave statistics

- Cumulative statistics

- Subseries statistics

and whether the phase-averaging should cover the swash zone or not.

If you select "Cumulative statistics" your specified output items will be updated regularly corresponding to the specified update interval. In case of "subseries statistics" your output items will be set to zero at the update interval. In most cases you would select "Cumulative statistics".

NOTE:

- The update interval should cover a reasonable time period (at least one wave period or more) in order to provide meaningful statistics.

- It is important to note that when the phase-averaging is selected to cover the swash zone the number of data used in the averaging processes is varying because of flooding/drying.

- The "Start at wave No" is the number of the first where the computation is initiated. The first wave is counted from when the water level crosses (uncrossing) the still water level first time. Most often you should keep the default value.

Output type

You can choose between following output types:

- type 0: point series data (dfs0 file)
- type 1: line series data (dfs1 file)
- type 2: grid series data (dfs2 file)

For point series data (type 0) you can choose a number of different points in each file.

You can view the results easily using the relevant MIKE Zero tools by clicking on the "View" button.

Output parameters

You can choose following output parameter types:

- Significant wave height
- Maximum wave height
- Maximum surface elevation
- Minimum surface elevation
- Mean surface elevation
- Mean flux, P

- Mean flux, Q

- Mean velocity, u_0

- Mean velocity, v_0

- Mean roller thickness

- Skewness

- Kurtosis

- Atiltness
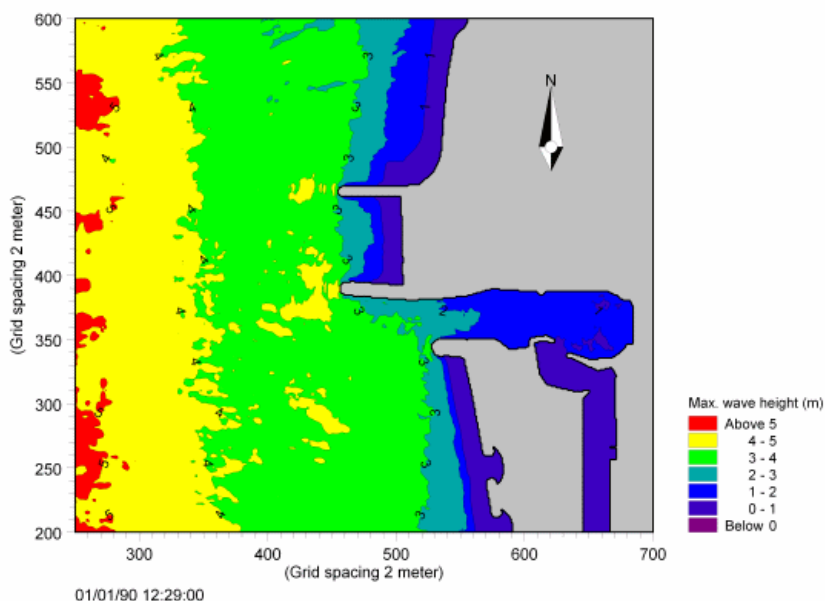
Figure 5.45   Example of phase-averaged output showing the maximum wave height
at a harbour entrance

Significant wave height

The significant wave height H_{m0} is defined by

$$H_{m0} = 4\sigma \tag{5.10}$$

where σ is the standard deviation of the surface elevation. Usually you will include this output item in your model specifications.

Maximum wave height

The maximum wave height H_{max} is defined as the maximum zero-crossing wave height within a specified time period.

Maximum surface elevation

The maximum surface elevation η_{max} is defined as the maximum value of the surface elevation within a specified time period.

Minimum surface elevation

The minimum surface elevation η_{min} is defined as the minimum value of the surface elevation within a specified time period.

Mean surface elevation

The mean surface elevation η_{mean} is defined as the mean value of the surface elevation taken over a specified time period. Usually you will include this output item in your model specifications when wave breaking and moving shoreline is included. The wave set-down and wave setup can be calculated from this item.

Mean flux, P

The mean flux P is defined as time averaged depth-integrated flux density in x-direction

Mean flux, Q

The mean flux Q is defined as time averaged depth-integrated flux density in y-direction.

Mean velocity, u$_0$

The mean velocity u_0 is defined as time mean of the depth-averaged velocity in x-direction. You should include this parameter in your output when studying wave-induced circulation.

Mean velocity, v$_0$

The mean velocity v_0 is defined as time mean of the depth-averaged velocity in y-direction. You should include this parameter in your output when studying wave-induced circulation.

Mean roller thickness

The mean roller thickness can be interpreted as a time-averaged measure of the intensity of wave breaking. Outside the surf zone the mean roller thickness is zero. The definition of the roller thickness is given in Sørensen et al (2004), which is included in the Scientific Documentation.

Skewness

The skewness S (no unit) is defined by

$$S = \frac{E(\overline{\eta^3})}{E(\overline{\eta^2})^{3/2}} \tag{5.11}$$

where E denotes the mean operator. Thus, the skewness may be examined as the mean cube of the time series of the surface elevation normalised by the mean square of the time series of surface elevation to the power 3/2. The

skewness is a higher-order integral measure of the lack of horizontal wave profile symmetry. For further information on this output type please see Kofoed-Hansen and Rasmussen (1998) p. 213ff. Usually you will not include this item in your model output specifications.

Kurtosis

The kurtosis K is defined by

$$K = \frac{E(\eta^4 - \bar{\eta}^4)}{E(\eta^2 - \bar{\eta}^2)^2} - 3 \tag{5.12}$$

where E denotes the mean operator. The kurtosis is a higher-order measure to identify non-Gaussian characteristics of a wave field. For a Gaussian sea state (i.e. linear and random waves) the kurtosis is 0. Usually you will not include this quantity in your model output specifications.

Atiltness

The atiltness A is defined by

$$A = \frac{E\left(\left(\frac{\partial \eta}{\partial t}\right)^3 - \left(\overline{\frac{\partial \eta}{\partial t}}\right)^3\right)}{E\left(\left(\frac{\partial \eta}{\partial t}\right)^2 - \left(\overline{\frac{\partial \eta}{\partial t}}\right)^2\right)^{3/2}} \tag{5.13}$$

where E denotes the mean operator. The atiltness is a higher-order integral measure of the lack of vertical wave profile symmetry. The parameter takes a positive value when the wave profile are tilted forward (like in the shoaling zone) and takes a negative value when tilted backward. For further information on this output type please see Goda and Morinobu (1998) p. 314ff.

Usually you will not include this quantity in your model output specifications.
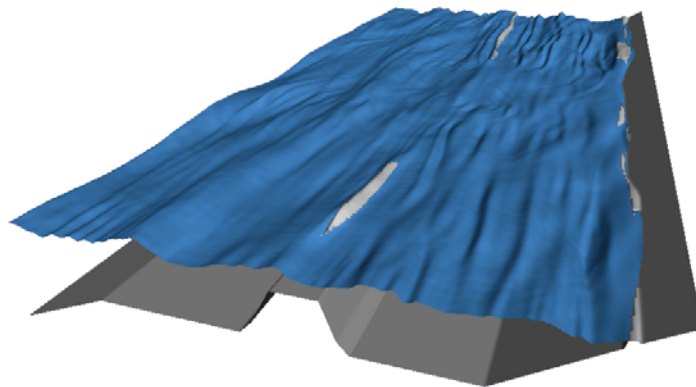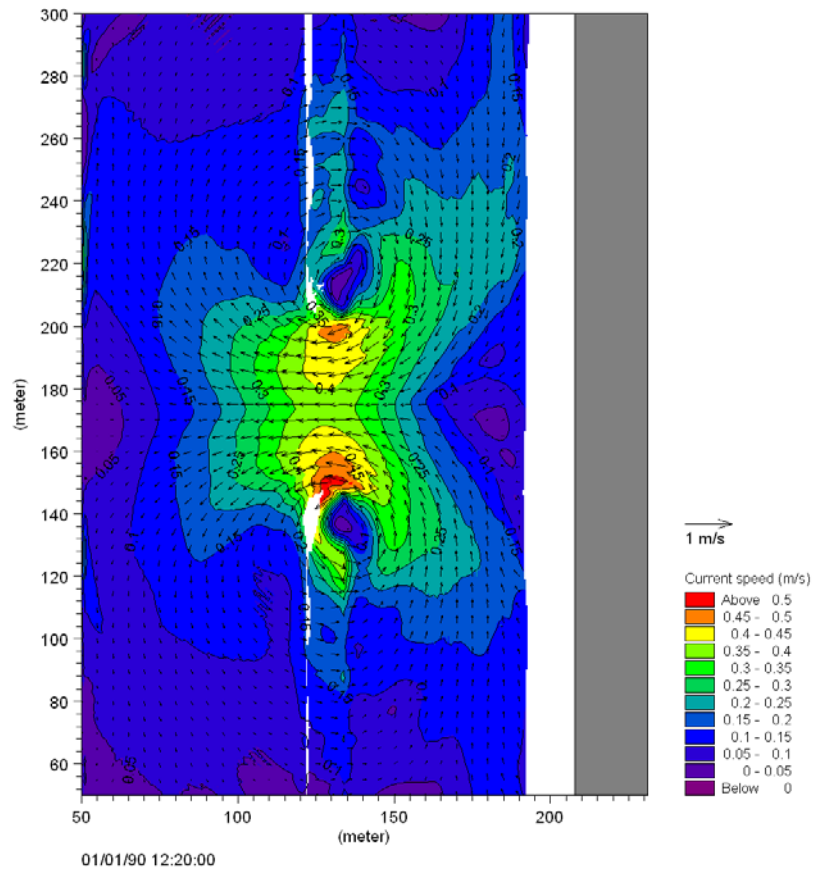
Figure 5.46   Example of phase-averaged output showing the wave-induced current (upper). The lower figure shows the instantaneous surface elevation on top of the bathymetry

Remarks and hints

When you specify the spatial range of your output data (1DH module only) you also have to specify the number of points you would like to save, N_{points}. The output result is interpolated to a grid having the constant spatial resolution of J_{last}-J_{first}+1/N_{points}.

In case you use a **structured mesh** of the bathymetry we recommend you to use the suggested default value of N_{points} (=J_{last}-J_{first}+1). For **unstructured meshes** we recommend you to use a larger value of N_{points}, in order to have a high resolution in the surf and swash zone. Further it is recommended (for unstructured meshes) to save the entire profile of data, i.e. keep the default values of J_{first} and J_{last}.

5.4.3 Wave disturbance parameters

Figure 5.47   A number of wave disturbance parameters can be extracted from a MIKE 21 BW simulation including detailed sub-area statistics

The **wave disturbance coefficient** is defined as the ratio of the significant wave height relative to the incoming significant wave height. If e.g. the wave height at a given position is 0.5 m and the incoming (offshore) wave height is 2.0 m, then the wave disturbance coefficient is 0.5/2.0 = 0.25, or 25%.

Output parameters

The results from the wave disturbance calculation are stored in a data file (.dfs2) with up to 3 items:

- Significant wave height, H_{m0}
- Wave disturbance coefficient, H_{m0}/H_{m0}, incoming
- Time of arrival of the first wave that is used in the calculation
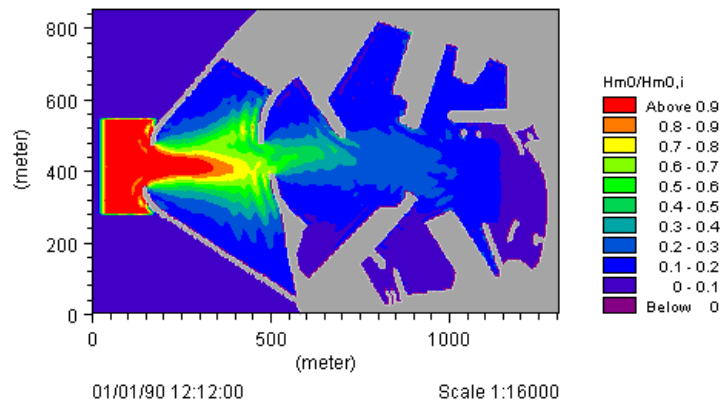
Figure 5.48   An example of a 2D map of wave disturbance coefficients

Type of scaling
You must specify whether the scaling of the coefficients should be relative to a user defined incoming wave height or relative to the wave height in a specific grid point. In the first case you must provide the incoming significant wave height, and in the second case the grid coordinates for the grid point.

Calculation period
The first and last time steps, for which the calculation should be performed.

You must also specify the interval (in number of time steps), for which the 2D map is updated. A typical value is 1-2 minutes.

Area statistics

When "wave dist. coeff." is set in your dialog settings you can include additional statistics for the areas defined in a so-called Area code map file. This Grid data file includes different integer values (larger than zero) for different areas where statistics of wave disturbance coefficient is required. The statistics covers following parameters:

- Maximum
- Minimum
- Mean
- Standard deviation
- Number of data for each defined area

The output is saved in an ASCII text file and can be viewed by common text editor. An example of output is shown in Figure 5.49.
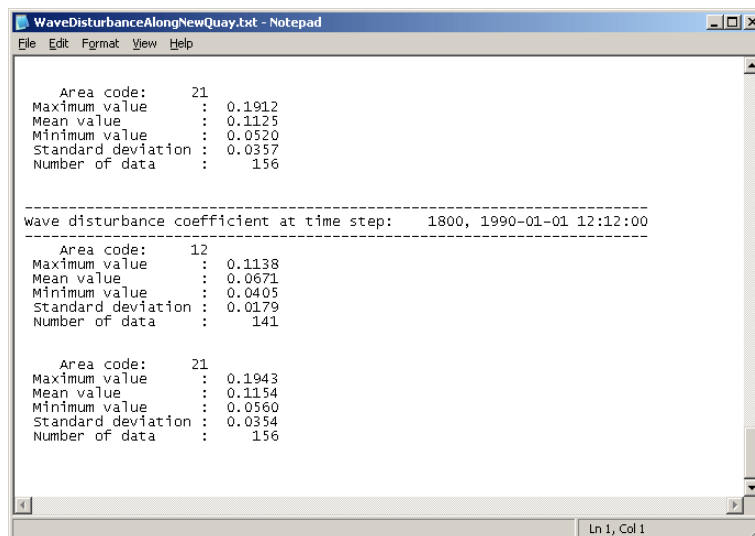
Figure 5.49   Example of output from the Area Statistics calculation. Se also Section 4.2.3

Area code map
Making a copy of the bathymetry and changing all water depths to zero easily generates the Area Code Map file. Then you select your sub-areas of interest (in the Grid Editor) and assign a code (integer number) to each area.

The Rønne Harbour example in the installation includes a setup, which you may use for reference.

Remarks and hints

Start-up period
Regular waves

Start-up period
For irregular waves in ports and harbours it is important to specify the simulation time long enough to allow the wave pattern inside the harbour to build up, including reflections from breakwaters and quays ("start-up period"). The calculation must be initiated after the wave pattern has reached a reasonable quasi-steady state (statistically speaking). If the calculation is started at arrival of the first wave (wave No. 1) the calculated wave disturbance coefficients inside the harbour may be underestimated.
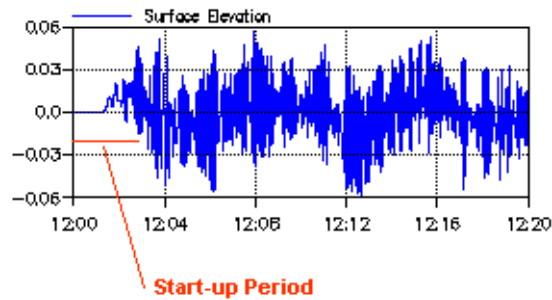
Figure 5.50   Definition of start-up period

The "start-up period" can be estimated using simple linear wave theory. Calculate the wave group velocity and estimate how long the travel time is for the first wave (e.g. the wave with the spectral peak period, T_p) to propagate from the boundary/wave generation line to the innermost part of the harbour. Add to this time e.g. 30 peak wave periods. The total time can be considered as an estimation of the "start-up period". The total duration of the simulation is the "start-up period", plus not less than 15 - 20 min. (which is usually the minimum). The calculation of the wave disturbance coefficients should be started at arrival of e.g. wave No. 30, and the calculation period should normally cover the time step range specified on the Simulation Period dialog.

The "start-up period" should not be confused with the warm-up period.

Regular waves
For regular waves the wave disturbance coefficients can be calculated for a period covering only a few wave periods after a quasi-steady state is reached. In this case the first wave to use in the calculation should be set to zero. Then the calculation will be performed from the first time step of the selected period. Note that for regular waves the significant wave height is a factor $\sqrt{2}$ larger than the wave height.

5.4.4   Moving shoreline parameters

The moving shoreline output MIKE 21 BW 1DH application only consists of time series containing the following three output items:

- Horizontal run-up
- Vertical run-up
- Total run-up

Figure 5.51   Run-up on beach

Horizontal run-up

The horizontal run-up is defined as the instantaneous cross-shore displacement of the still water shoreline. The initial value equals the x co-ordinate of the still water shoreline.

Vertical run-up

The vertical run-up is defined as the vertical displacement of the still water level.

Total run-up

The total run-up is defined as the run-up distance measured along the a swash profile.
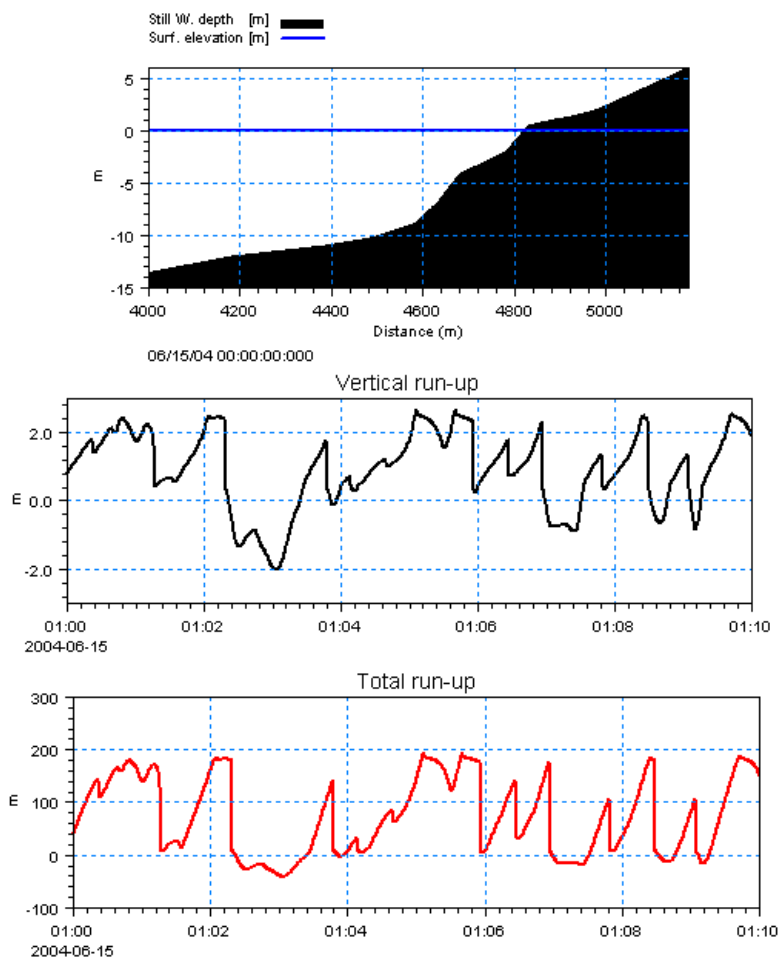
Figure 5.52   Example of moving shoreline output. Upper panel shows the coastal profile, middle the vertical run-up and lower panel the total run-up measured along the profile

5.4.5   Hot start Parameters

The hot start facility allows you to start a simulation as a continuation of an earlier run. For this to be possible, details of the finishing conditions of the earlier simulation must have been saved in a hot data.

The information stored in the hot-data file includes the bathymetry, the initial surface elevation for the continuation, and the basic model specifications, such as model dimensions, grid size, time step, etc. These are used directly by the model and cannot be altered and controlled when setting up the continuation simulation.

The hot-data file does not, however, include the array containing porosity, sponge layer coefficients, or the bed resistance coefficients. These should be specified for the continuation run exactly as in the simulation from which the hot-start data derive.

Remarks and hints

The hot start facility is mainly used for 2DH applications involving wave-current interactions, where the wave simulation is hot started from a run with currents only.

5.5 Entries Arranged Alphabetically

The following subjects are dealt with in this section:

- Application Progress Visualization (APV)
- Artificial land
- Batch mode
- Blow-up
- Boussinesq cross terms
- Courant number
- Deep water terms
- First time model set-up
- Hardware requirements
- High-frequency noise
- Linear dispersion relation
- Mesh generation
- Minimum wave period
- Numerical damping
- Option parameters
- References

5.5.1 Application Progress Visualization (APV)

The progress view includes an Application Progress Visualizer bar showing the progress of the simulation, current simulation time (or time step), computational speed and expected remaining computational time. You may also halt the simulation, change the base priority settings, suspend or set the simulation on idle.

The model simulation log file is shown at all time. The log file is an ASCII file and has the same name as the input PFS-file, but the suffix is ".log". This file

contains a list of all the specifications used for the execution, plus information about the input data and the output data.

If a Blow-up is detected, then the error messages will appear in the log file.

Right clicking on the log file in the progress windows, you may use the "Find…" feature to search in the log file. The "Properties…" page provide you information about the settings for the actual simulation.

When enabling the "Find…" feature the simulation is suspended until canceling the search.

5.5.2   Artificial land

Artificial land is a term used for areas which in the "prototype" are water, but in the model are described as land. Artificial land should only be applied to water areas, which have no influence on the wave conditions in areas of interest.

The idea of applying artificial land is to save computational time.

In the example below, artificial land has been applied on both sides of the harbour entrance and for an area inside the harbour representing a shallow area, where the wave energy is absorbed. In the example the modelled waves approaching the harbour are considered unidirectional. If the wave field had been directional (short crested) a wider spacing between the two artificial land areas would have to be chosen to allow for the directionality.

To be able to separate "real" and "artificial" land in the subsequent plotting it is a good idea to assign the two types of land different values in the bathymetry file e.g. 10 for real land, and 5 for artificial land.
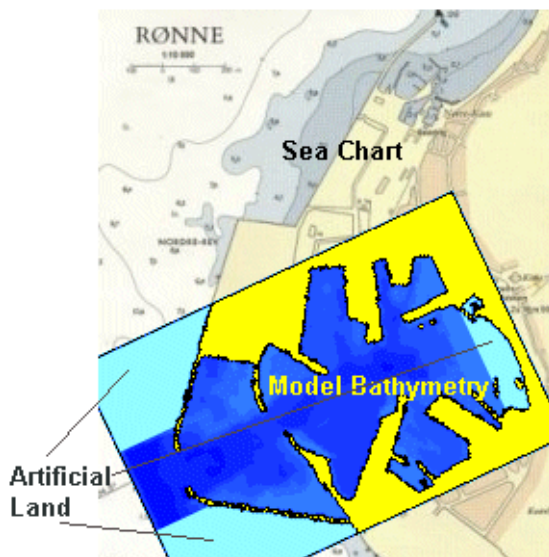
Figure 5.53   Application of artificial land in model setup

5.5.3   Batch mode

If you have a number of runs to be executed in a row you might run the model setup's in batch mode. The procedure is as follows:

- prepare your specification files (PFS-files) for your runs, i.e. files with extension .bw

- prepare a run.bat file (the file name needs to have the extension .bat) in a text editor. The bat-file should include a path to the bin directory and lines for each specification file (default 64bit installation):
path=%path%;"C:\Program Files (x86)\DHI\MIKE Zero\[year]\bin\x64\MzLaunch.exe" [filename.bw]

- execute run.bat in a Command-prompt or simply click on the text file in your windows file manager

Following this procedure your runs will be execute in a row as described in the run.bat file.

Batch execution can also be initiated using the Launch Simulation Manager available in Start -> All Programs -> MIKE Zero [year] -> Launch Simulation Engine.

5.5.4 Blow-up

MIKE 21 BW checks the requests and input from you as much as possible, but in order to minimise the number of restrictions on potential applications, it is allowed to request MIKE 21 BW to perform unrealistic simulations. Also, MIKE 21 BW is an advanced numerical model, which means that the possibility of numerical instabilities ("blow-ups") exists. Finally, no software product, including MIKE 21 BW, is completely error free.

The sections below describe what to do if errors occur during the first few time steps, or after some time steps have been completed. Also help in case of model crash before the first time step is given.

Blow-up after the first few time steps

General description
You have started the execution of a simulation and it stops after the message (in the .log-file "Time step: 1", and possibly "Time step: 2", and a few more time steps have been written.

Normally this type of error is caused by an inconsistency in the simulation setup. Therefore, please check your model specifications carefully.

Near the end of the .log file you get the message:

"-----BLOW-UP detected-----"

In other words, you have encountered a numerical instability. However, as it occurs very early in the simulation it is most likely caused by an inexpedient combination of model input.

What to do?
If one or more time steps have been completed you should make a contour plot of surface elevation of the last one of these time steps. These plots will probably show you in which part of the domain the blow-up occurs. Notice, that first you may need to re-run the simulation storing the results at every time step and in every grid point.

Using the plots as a guideline you should check the following:

1. That the bathymetry has been specified correctly, and that the grid spacing is sufficiently small to adequately resolve the waves specified in the open boundary conditions, or at the internal generation line

2. That the time step is sufficiently small to adequately resolve the waves and surface rollers

3. That the Courant number is less than one (0.5 in the 1DH module)

4. That the water depth is sufficient at all active grid points to ensure that they do not dry out (if moving shoreline is not included)

5. That the correct still water level (SWL) has been specified, and that the boundary conditions and initial surface elevation have been specified relative to the still water level

6. That the boundary/internal generation data are physically realistic

7. That the coefficients used for sponge layers and porosity layers are constant and realistic

If all your efforts fail, you can contact your local DHI Software Support Centre.

Blow-up after some time steps

General description

You have started the execution of a simulation and it stops after a number of time steps (e.g. 100, or 3892).

Near the end of the .log file you get the message:

"-----BLOW-UP detected-----"

In other words, you have encountered a numerical instability.

Instabilities which develop some time into a model simulation may also be the result of inconsistencies in your input specifications. This is because the Boussinesq wave model is relatively robust, and it can take some time before the effects of inconsistencies in your input lead, ultimately, to instability. Other possible causes of late instabilities are an accumulation of reflected wave energy within the model area, and wave breaking.

What to do?

To determine the cause of instability you should make a plot of levels/surface elevation of the last time step stored in the output file. If you have also stored fluxes, you can also make a vector plot. If these plots are too messy, you should plot the results from the previous time step, and so on, until you get a plot which clearly shows where the instability starts (e.g. where unrealistic surface elevations begin to appear). Note, that you may first need to re-run the simulation storing the results at every time step and in every grid point.

The plots not only show you where the instability starts, they may also be used as a guide to determine the cause. What action needs to be taken will depend on the main cause of the instability.

Blow-up near porosity layers

The reason for the blow-up may be application of too small porosity values. Most often the porosity should be larger than 0.2. If you still have a blow-up, you can try to use a larger value or increase the number of porosity layers in front of the structure.

Blow-up near areas with large bathymetry gradients

Strictly speaking, the Boussinesq equations are derived assuming a slowly varying bathymetry (bottom slope less 1:3). If the gradient of the water depth is large (e.g. bottom slope of 1:1 or 1:2) instabilities may appear. You can try reduce the bottom slope by editing the bathymetry.

Alternatively, you can try to use another numerical scheme for the convective terms (see also Section 5.2.4). It is recommended to first apply the "Central differencing with simple upwinding at steep gradients and near land". In case of a new blow-up you can try the quadratic upwinding scheme. If the instability still persists please try the simple upwind scheme.

Blow-up near wave breaking

When there is a build-up of unrealistic waves in shallow water, or in areas where there is a focusing of wave energy, the instability may be caused by breaking waves. In that case you should absorb the breaking wave energy by introducing sponge layers.

You may also reconsider you model setup and include wave breaking. As you have to resolve the individual breaking waves, you would probably need to reduce the grid spacing (to 1-2 m).

Blow-up near open boundaries/generation lines

When the build up of unrealistic waves appear near an open boundary/generation line this may be caused by an inconsistency. Please check the boundary/generation specification, and the input data files.

2DH simulations with wave breaking and moving shoreline

In most 2DH applications including wave breaking and moving shoreline a time extrapolation factor of slightly less than one is recommended for numerical stability. Time extrapolation factors within 0.8-0.9 (for all water depths) have successfully been used in a number of wave breaking applications. If a blow-up occurs (typically seen as high-frequency noise as shown in Figure 5.55) you may try to reduce the time-extrapolation factor to 0.7.

For stability reasons it is recommended to use the scheme 'Simple upwinding at steep gradients and near land' for the space discretisation of the convective terms in connection with simulations including wave breaking (and moving shoreline). If a blow-up occurs in the breaking zone you may try to reduce the time step or use the scheme 'Simple upwinding'. Please note this scheme is quite dissipative if the waves are not resolved properly.

If the blow-up occurs near the moving shoreline (and a low pass filter is used) please increase the slot width and/or slot depth, see also Section 5.3.9 Moving shoreline.

1DH simulations with wave breaking and moving shoreline

When a blow-up occurs in a 1DH application it happens most often near the swash zone. The first step is to reduce the time step and next to increase the slot width.

If a blow-up occurs outside the surf/swash zone please set the so-called supg_coefficient to e.g. 0.1 in an OPTION section (see Section 5.5.15). The default value is 0.05. It is not recommended to use a SUPG coefficient larger than 0.25.

Blow-up using the enhanced equations

Even though the minimum wave period is resolved by approximately 35 time steps, instabilities may appear. An example is shown below in Figure 5.54. Then the time step should be reduced to a temporal resolution of 50 points per wave period. If instabilities still appear, then you may try using a partly linear time-extrapolation of the cross-Boussinesq terms. Excluding the extrapolation will result in some artificial damping of waves propagating with an angle to the grid, and the effect will the enhanced for increasing water depth to deep water wave length. However, partly time-extrapolation applied in a local area will usually not affect the overall results. More information at partly time-extrapolation.

Please note: Exclusion of the time extrapolation (i.e. time extrapolation factor = 0) should only be used as a last resort.
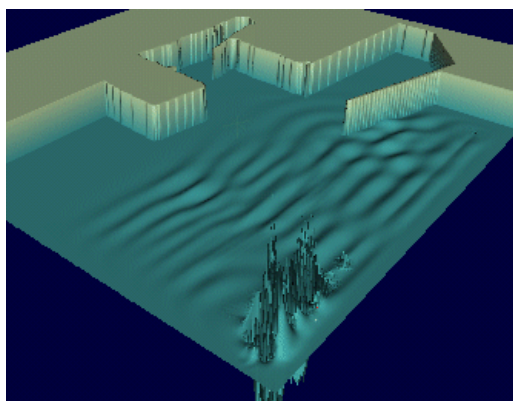
Figure 5.54   Example of blow-up near an internal generation line

Boundary/generation wave data

In order to avoid an instability you may consider generating new open boundary or internal generation data using a different initial random seed. The overall statistics of the data will not change (it is just a different realisation).

Reducing the wave height

As long as the problem you would like to model is not strongly non-linear you may reduce the incident significant wave height.

Crash before the first time step

General description

You have started the execution of a simulation and it stops immediately after. A typical message will be:
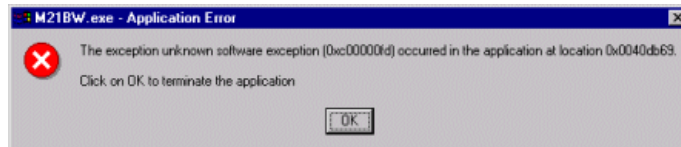
Figure 5.55   Stack overflow problem

This error may occur if the stack size is too small, i.e. a stack overflow problem.

What to do?

You can change the stack size using the MS program EDITBIN.exe. To run EDITBIN, use the following syntax:

editbin /STACK:10000000 m21bw.exe

In this case the stack size is set to 10 MB. You may also try to use a smaller number by trial and error.

If all your efforts fail you can contact you local DHI Software Support Centre.

5.5.5   Boussinesq cross terms

The representation of the Boussinesq cross terms (i.e. Q_{xyt}, Q_{yt} and Q_{xt} in the x-sweep and P_{xyt}, P_{xt} and P_{yt} in the y-sweep (see section 6 Scientific Documentation *(p. 191)*) has required special attention. In order to obtain the correct time-centering, we have used linear time-extrapolation of these terms. A straightforward numerical representation leads to a backward centering (n/2) of these terms and will result in artificial dissipation of waves propagating with an angle to the grid.

Generally, you should always use the linear time-extrapolation when the Deep Water Terms are included. However, excluding the extrapolation makes the simulation more stable, at the cost of some numerical damping.

5.5.6 Courant number

The Courant Number is an expression which describes the number of grid points that wave information will travel in one time step. It is defined as follows:

$$Cr = c\frac{\Delta t}{\Delta x} \tag{5.14}$$

where c is the wave propagation speed (or celerity), Δt is the time step, and Δx is the spatial resolution.

Important: The Courant number should always be equal to, or less than, 1 in 2D applications and less than about 0.5 in 1D applications.

5.5.7 Deep water terms

MIKE 21 BW solves the classical form and an enhanced form of the Boussinesq equations. The enhanced equations are different from the classical formulation as a number of new Boussinesq correction terms (so-called deep water terms) are included.

The major restriction of the classical form of the Boussinesq equations is the shallow water limitation in terms of the water depth to deep water wave length ratio, h/L_0. Celerity errors gradually increase for increasing h/L_0, and 5% is reached for $h/L_0 = 0.22$, which is often taken as the practical deep water limit for these equations.

The new form of the Boussinesq equations incorporates a significant improvement of the dispersion relations, which makes it possible to use the new equations to simulate the propagation of irregular wave trains in shallow to deep water up to a depth to deep water wave length ratio, h/L_0, of 0.5.

5.5.8 First time model set-up

For an efficient determination of the model resolution in space and time you can use the MIKE 21 BW Model Setup Planner.

Alternatively you can follow the short recipe listed below. The procedure is for typical short period wave disturbance study using unidirectional or directional waves.

1. Define the minimum (h_{min}) and maximum (h_{max}) water depth in your model area.

2. Define the minimum wave period in the spectrum which you would like to resolve in the model

$$\frac{h_{max}}{L_{0,min}} \leq \beta \Rightarrow T_{min} \geq \sqrt{\frac{2\pi h_{max}}{g\beta}} \qquad (5.15)$$

where $\beta = 0.22$ for the classical equations, and $\beta = 0.5$ for en enhanced equations (see Minimum wave period p. 183).

3. Find the grid spacing so that the minimum wave length is resolved by at least 7 grid points, i.e.

$$\frac{L_{min}}{\Delta x} > 7 \qquad (5.16)$$

Here L_{min} is the wave length corresponding to T_{min} and h_{min}. The wave length can be calculated from the Linear dispersion relation.

In case of breaking waves and moving shoreline boundary 20-40 grid points are required for the most energetic waves (20 points are assumed in the MIKE 21 BW Model Setup Planner) for the minimum wave period.

4. Find the time step for the numerical time integration. Beside fulfilling the CFL criterion (first expression in bracket), the minimum wave period has to be resolved by a minimum of 7 (classical equations), or 35 (enhanced equations), time steps, i.e.

$$\Delta t < \{\frac{\Delta x}{\sqrt{gh_{max}}}, \frac{T_{min}}{7} \text{ or } \frac{T_{min}}{35}\} \qquad (5.17)$$

In case of breaking waves and moving shoreline boundary the time step is often 0.02-0.1 s in 2DH applications.

In 1DH applications the Courant number needs to be less than about 0.5 and the time step is most often within the range 0.001-0.1s depending on the spatial resolution.

5. Find the simulation duration. Most often the simulation time is longer than 20 minutes, see section 5.2.6 Simulation period.

6. Then you have to decide whether you should apply long crested (unidirectional waves) or short crested (directional) input at the internal generation line.

Please see Setup of internal generation lines (p.128) and Examples of wave fields (p.131).

7. Finally, you should evaluate the CPU, RAM and hard disk requirements.

Evaluation

If you have experienced that your first model set-up takes far to long time to run and requires much more internal memory than available, then you have to reconsider your model set-up. Most often you have to increase the minimum wave period and/or reduce the maximum water depth, reduce the size of the computational domain, etc. In other words, you have to prepare the next iteration starting from point 1 in this recipe.

5.5.9 Hardware requirements

CPU requirements

You can evaluate the required computational time if you know the computational speed (CPS) of your PC running MIKE 21 BW. An estimate is given in the pop-up dialog when you start executing the program. For a 4-500 MHz Pentium II processor the CPS is in order of 100000 points/second for 2D applications and about 15000 points/second for 1D applications (including surf zone dynamics).

When you know the number of grid points in both x- and y-directions (N_x and N_y), the ratio between number of water points and total number of points (α), and the number of time steps N_t, you can evaluate the required CPU time using the formula

$$CPU = \frac{\alpha N_x N_y N_t}{CPS} \tag{5.18}$$

where the unit of *CPU* is seconds. The CPU can be calculated in the MIKE 21 BW Model Setup Planner (Java script) included in the Online Help.

Hard disk requirements.

To keep storage requirements within reasonable bounds, the following suggestions are made:

- for plotting purposes you should not store results from the whole model area too frequently (store e.g. every 10th, 50th or 100th time step, and most often only the levels/surface elevation).

- for time series analysis (including spectral analysis, etc.) you should store results at every time step (e.g. at every 5th or 10th grid point and most often only the levels/surface elevation).

- include the calculation of the wave disturbance coefficients.

Note that an estimate of the required storage for your specified output data is provided through the graphical user interface.

RAM requirements

You can evaluate the required internal memory by

$$RAM \approx 20 \cdot 10^{-4} N_x + 21 \text{ MB} \qquad (1\text{DH})$$

$$RAM \approx 2{,}0 \cdot 10^{-4} N_x N_y + 21 \text{ MB} \quad (2\text{DH})$$

$$RAM \approx 2{,}5 \cdot 10^{-4} N_x N_y + 21 \text{ MB} \quad (2\text{DH, incl. wave breaking/moving shoreline})$$

(5.19)

where the unit of *RAM* is MB.

5.5.10 High-frequency noise

The numerical integration of the enhanced Boussinesq equations may sometimes result in instabilities, which eventually may cause a model blow-up. An example is illustrated in the figure below.
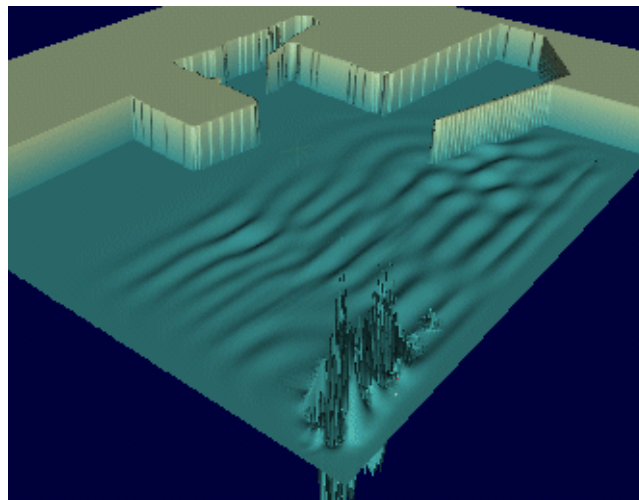
Figure 5.56   High frequency noise

Often the instability (high frequency noise) appears in the computational domain with the highest water depth (typically near an internal generation line or at an open boundary). As explained in "Blow-up after some time steps" (p. 174) the time step should be reduced to avoid this instability. Alternatively, you can use a depth-dependent time-extrapolation factor.

5.5.11 Linear dispersion relation

For small amplitude waves, non-linear effects can be neglected and analytical methods can be used to derive the linear dispersion relation.

The phase celerity using Stokes first order theory is given by

$$c = \sqrt{\frac{g \tanh(kh)}{k}} \qquad (5.20)$$

where c is the celerity (= L/T), k the wave number (= 2π/L), h the water depth, and g is the gravitational acceleration (= 9.81 m/s^2). L and T are the wave length and wave period, respectively.

For the calculation of various wave parameters you can use DHI's linear wave calculator included in the Online Help.

Boussinesq equations

You can use this dispersion relation as a good approximation to the phase celerity of the Boussinesq equations used in MIKE 21 BW as long as you fulfil the requirements:

- Classical Boussinesq equations: $h_{max}/L_{0,min} < 0.22$
- Enhanced Boussinesq equations: $h_{max}/L_{0,min} < 0.50$

Here h_{max} denotes the maximum water depth in the model domain, and $L_{0,min}$ the minimum wave length in deep water (= $gT^2/2\pi$). T is the minimum wave period. A comparison between the phase celerity derived for the Boussinesq equations and Stokes first order theory is shown in Madsen et al (1991) and Madsen and Sørensen (1992), see Scientific Documentation.

5.5.12 Mesh generation

You can use the MIKE 21 BW 1DH module for either structured or unstructured meshes. An unstructured mesh is recommended for detailed modelling of surf and swash zone dynamics as this allows you to use a high spatial resolution in areas of particularly interest. An extracted profile from a e.g. a dfs2 bathymetry file can be used directly in MIKE 21 BW 1DH as a structured mesh having a constant resolution/grid spacing.

If you would like to increase the spatial resolution in local areas (see e.g. the example below) you first have to use the MIKE 21 Toolbox program "Mesh Generation for MIKE 21 BW" for generation of an unstructured mesh. Based on the structured data (dfs1-type data file) this tool generates a new file containing two items; Item 1: co-ordinate (in length unit) of the node in the mesh and Item 2: the corresponding water depth in the node. The second item is used as the bathymetry mesh.

Important note for application of unstructured mesh

For generation of files including sponge, porosity, bed friction and filter coefficients you have to use the original bathymetry/profile file (as used as input to the MIKE 21 Toolbox program "Mesh Generation for MIKE 21 BW"). Linear interpolation is used to obtain the respective values at the non-uniformed distributed node points.
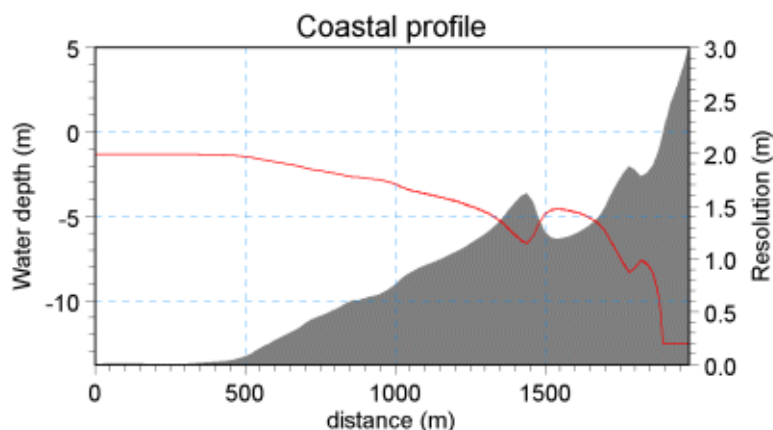
Figure 5.57   Example of spatial resolution for a barred coastal profile. Conditions: Spectral peak period 8 s, 40 nodes per wave length and minimum mesh spacing 0.2 m

5.5.13 Minimum wave period

The minimum wave period that can be resolved in the simulation is governed by the maximum water depth in the model area and by whether deep water terms are included

If the deep water terms are excluded, the minimum wave period can be calculated from $h_{max}/L_0 = 0.22$, where h_{max} is the maximum water depth and L_0 is the deep water wave length ($L_0 = 1.56T^s$)

Including the deep water terms increases the ratio h/L_0 to 0.5.

Example
Maximum water depth: 20 m

The minimum wave period that can be modelled with and without the deep water terms is shown in the table below.

	Including Deep Water Terms	Excluding Deep Water Terms
Max. h/L_0*	0.5	0.22
Min. L_0	40 m	91 m
Min. Wave Period, Tmin	**5.1 s**	**7.6 s**

$$*L_0 = \frac{gT^2}{2\pi} \approx 1.56\,T^2 \tag{5.21}$$

5.5.14 Numerical damping

The figures below illustrate the influence of numerical damping on the significant wave height in case of a practical application involving two generation lines and the deep-water terms (enhanced Boussinesq equation).

The model setup is identical to the Hanstholm Harbour example enclosed in the installation.

The first panel shows the result of a simulation with time-extrapolation (default in MIKE 21 BW, time-extrapolation factor = 1). On the second panel the time-extrapolation has been omitted corresponding to a backward time-centring of the cross-terms (time-extrapolation factor = 0). Finally, the third panel shows the results of using a mixture, i.e. time-extrapolation factor = 0.5. A spatial constant time-extrapolation factor has been used.

The figures clearly illustrate the numerical damping when full or partly backward time-centring of the cross-Boussinesq terms is applied all over the computational domain.
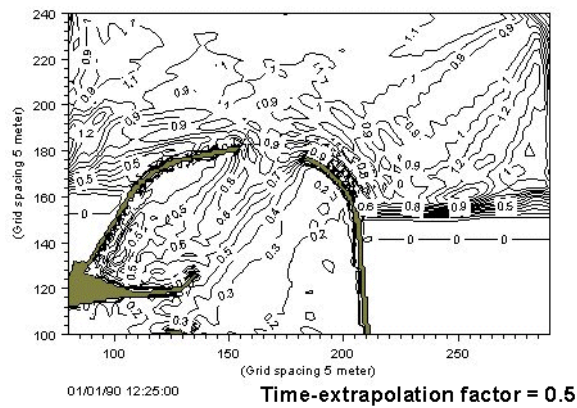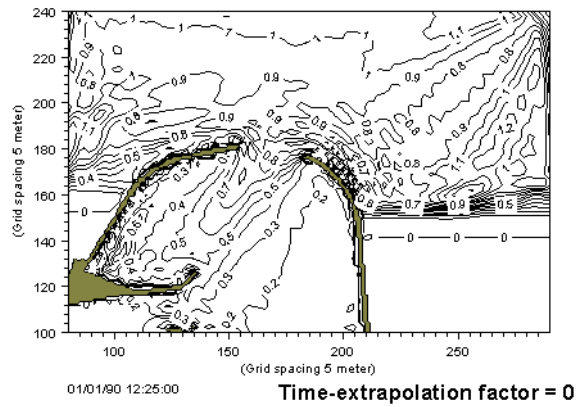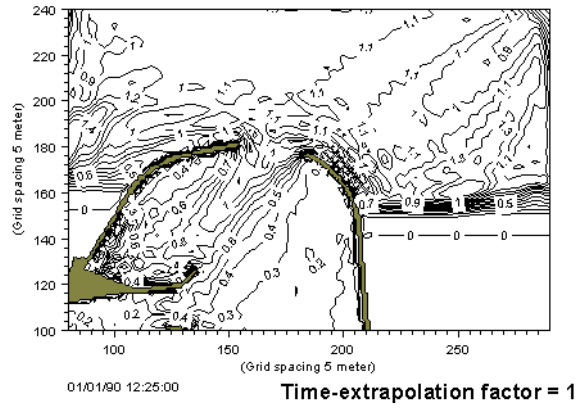
Figure 5.58   Application of different time-extrapolation factors

5.5.15 Option parameters

The option system offers the possibility of specifying input to the model specification (PFS-file) without using the editor. The input to be specified through the option system mostly consists of parameters which normally should not be used.

Example

The format of the option system is very simple and is illustrated below. You add the option parameter as a keyword in the section called OPTION_PARAMETERS in the specification file (file with extension *.BW). In this example the **Name_of_Text_Editor** is set to "Textpad.exe" (see section 5.2.1). The default value is one.

```
[MIKE21_BOUSSINESQ_WAVE]
    [OPTION_PARAMETERS]
        Name_of_Text_Editor = 'Textpad.exe'    ◄──────
    EndSect   // OPTION_PARAMETERS

    [BASIC_MODEL_PARAMETERS]
        NUMBER_OF_DIMENSIONS = 2
        DELETE_FEM_PFS_FILE = 0
        DELETE_FEM_MESH_FILES = 0
        OPEN_LOG_FILE_AFTER_RUN = 1
    EndSect   // BASIC_MODEL_PARAMETERS
```

Figure 5.59   Inclusion of an option parameter in a MIKE 21 BW PFS-file

5.5.16 References

The references listed below provide you with more basic information applicable to the Boussinesq wave module:

Application, verification and practical aspects

Gierlevsen, T, Vargas, B M, Pires, V P L & Acetta, D, J (2003) Numerical and physical modelling of storm waves at Rio de Janeiro Yacht Club. In Proc. COPEDEC VI, Colombo, Sri Lanka, 17 pp.

Gierlevsen, T, Hebsgaard, M & Kirkegaard, J (2001) Wave disturbance modelling in Port of Sines, Portugal - with special emphasis on long period oscillations. In Proc. International Conference on Port and Maritime R&D and Technology, Singapore, 8pp.

Kofoed-Hansen, H, P Slot, O R Sørensen and J Fuchs (2001) Combined numerical and physical modelling of seiching in exposed new marina. In Proc. 27th International Conference of Coastal Engineering, 3600-3614.

Ozsanne, F, Chadwick, A J, Huntley, D A, Simmonds, D J and Lawrence, J (2000) Velocity predictions for shoaling waves with a Boussinesq-type model. Coastal Eng., 41, 361-397.

Kuang-ming, Y, M Rugbjerg and A Kej, (1987) Numerical modelling of harbour disturbance in comparison with physical modelling and field measurements. Proc. Second Int. Conf. on Coastal and Port Eng. in Developing Countries, Beijing, China.

Berenguer, I, Rugbjerg, M, Madsen, P A & Kej, A (1986) Mathematical and physical wave disturbance modelling-complementary tools. Proc. 20th Int. Conf. on Coastal Eng., Taipei, 9-14 November 1986.

Abbott, M B, McCowan, A D & Warren, I R (1984) Accuracy of Short-wave Numerical Models. Journal of Hydraulic Engineering, 110, 10.

Madsen, P A and Warren, I R (1983) Performance of a Short-wave Numerical Model, Coastal Engineering, 8.

Abbott, M B, McCowan, A D & Warren, I R (1981) Numerical Modelling of Free-Surface Flows that are Two-dimensional in Plan. In: Transport Models for Inland and Coastal Waters, editor M B Fischer, Academic Press, New York.

Abbott, M B, Petersen, H M & Skovgaard, O (1978) On the Numerical Modelling of Short-waves in Shallow Water. Journal of Hydraulic Research, 16, 3.

Boussinesq type equations

Kofoed-Hansen, H & Rasmussen, J H (1998) Modelling of nonlinear shoaling based on stochastic evolution equations. Coastal Engineering, 33, 203-232.

Sørensen, O R, Schäffer, H A & Madsen, P A (1998) Surf zone dynamics simulated by a Boussinesq type model III. Wave-induced horizontal near-shore circulations. Coastal Engineering, 33, 155-176.

Schäffer, H A, Madsen, P A & Sørensen, O R (1993) A Boussinesq model for waves breaking in shallow water. Coastal Engineering, 20, 185-202.

Madsen, P A & Sørensen, O R (1993) Bound Waves and Triad Interactions in Shallow Water. Ocean Eng., 2, 4, 359-388.

Madsen, P A & Sørensen, O R (1992) A new form of the Boussinesq equations with improved linear dispersion characteristics. Part 2: A slowly-varying Bathymetry. Coastal Eng., 18, 183-204.

Madsen, P A, Murray, R & Sørensen, O R (1991) A new form of the Boussinesq equations with improved linear dispersion characteristics, (Part 1). Coastal Engineering, 15, No 4.

Peregrine, D M (1967) Long Waves on a Beach. Journal of Fluid Mechanics, 27, Part 4.

Witting, J M (1984) A unified model for the evolution of non-linear water waves. J. Computational Physics 56, pp 203-236.

Wave Generation and Synthesising

Bendat, J L and A G Piersol (1986) Random Data. Analysis and Measurements Procedures. John Wiley & Sons.

Sand, S E (1987) Directional wave generation and analysis. AIRH-Seminar-IAHR Wave analysis and generation in laboratory basins, Lausanne. 203-235.

Porosity and partial reflection

Engelund, F (1953) On the laminar and turbulent flows of groundwater through homogeneous sand. Trans. Danish Academy of Tech. Sc., 3, 4.

Madsen, P A (1983) Wave Reflection from a Vertical Permeable Wave Absorber. Coastal Engineering, 7.

Madsen, O S (1974) Wave Transmission through Porous Structures. Journal of Waterways, Harbours and Coastal Engineering, 100, WW3.

Thompson, E F, H S Chen and L L Hadley (1996): Validation of numerical model for wind waves and swell in harbours. Journal of Waterway, Port, Coastal and Ocean Engineering, 122,5. 245-257.

Sponge Layers and Radiation Boundaries

Larsen, J & Dancy, M (1983) Open Boundaries in Short-wave Simulations - A New Approach. Coastal Engineering, 7.

Wave breaking

Goda, Y & Morinobu, K (1988) Breaking wave heights on horizontal bed affected by approach slope. Coastal Eng. Journal, 40, 307-326.

Ting, F C K & Kirby, J T (1994) Observation of undertow and turbulence in a laboratory surf zone. Coastal Eng., 24, 51-80.

Swash-zone hydrodynamics

Butt, T & Russel, P (2000) Hydrodynamics and cross-shore sediment transport in the swaxh-zone of natural beaches: A review. J Coastal Research, 16, 255-268.

Text books

Abbott, M B & Madsen, P A (1990) Modelling of Wave Agitation in Harbours. The Sea, Ocean Engineering Science, 9, Part B, Chapter 33, pp 1067-1103.

Eagleson, P S & Dean, R G (1966) Small Amplitude Wave Theory. In: Estuary and Coastline Hydrodynamics, editor A T Ippen, McGraw-Hill, New York.

Hamm, L, Madsen, P A & Peregrine, D H (1992) Non-linear wave dynamics in the near-shore zone. A review paper on MAST Morphodynamics.

Larsen, J, Madsen, P A & Abbott, M B (1984) Numerical modelling of directional seas in deepwater. Proc. Symp. Description and Modelling of Directional Seas, Lyngby, Paper No C-2, p 11.

Mei, C C (1983) The applied dynamics of ocean surface waves. John Wiley & Sons, New York.

Svendsen, I A and Jonsson, I G (1980) Hydrodynamics of Coastal Regions, Technical University of Denmark.

U S Army. Coastal Engineering Research Center (1984) Shore Protection Manual.

Whitham, G B (1974) Linear and non-linear waves. John Wiley & Sons, New York.

Wiegel, R L (1964) Oceanographical Engineering, Prentice-Hall, Englewood Cliffs.

6 Scientific Documentation

Scientific documentation for MIKE 21 BW can be accessed online via the Documentation index. Likewise the following papers are available:

Abbott, M B, McCowan, A D & Warren, I R (1984): "Accuracy of Short-wave Numerical" *Models. J. Hydr. Eng.*, **110**, 173-197.

Madsen, P A (1983), "Wave reflection from a vertical permeable wave absorber". *Coastal Eng.*, **7**, 381-396.

Madsen, P A, Murray, R & Sørensen, O R (1991), "A new form of the Boussinesq equations with improved linear dispersion characteristics, (Part 1)", *Coastal Eng.*, **15**, 371-388.

Madsen, P A & Sørensen, O R (1992), "A new form of the Boussinesq equations with improved linear dispersion characteristics. Part 2: A slowly-varying Bathymetry". *Coastal Eng.*, **18**, 183-204.

Madsen, P A, Sørensen, O R and Schäffer, H A (1997a), "Surf zone dynamics simulated by a Boussinesq type model. Part I: Model description and cross-shore motion of regular waves." *Coastal Eng.*, **32**, 255-288.

Madsen, P A, Sørensen, O R and Schäffer, H A (1997b), "Surf zone dynamics simulated by a Boussinesq type model. Part II: Surf beat and swash zone oscillations for wave groups and irregular waves." *Coastal Eng.*, **32**, 289-320.

Sørensen, O R, Schäffer, H A and Madsen P A (1998), "Surf zone dynamics simulated by a Boussinesq type model. Part III: Wave-induced horizontal nearshore circulations." *Coastal Eng.*, **33**, 155-176.

Sørensen, O R, Schäffer, H A and Sørensen. L S (2004), "Boussinesq-type modelling using an unstructured finite element technique". *Coastal Eng.*, **50**, 181-198.

For further description of the physical, mathematical and numerical background related to wave modelling using Boussinesq type equations, see References in section 5.5.16.

INDEX