BREAKWAT 3.1.1, June 2005

User & Technical Manual
Version 3.1.02

wl | delft hydraulics

Contents

1 Introduction

1.1 BREAKWAT 3

BREAKWAT 3 is a tool to aid the designer in the preparation of a conceptual design of breakwater structures. The design tools given in this manual are based on tests of schematised structures. Structures in prototype may differ substantially from the tested sections. Results, based on these design tools, can therefore only be used in a conceptual design. The confidence bands given for most formulas support the fact that reality may differ from the mean curve. It is advised to perform physical model investigations for detailed design of all important breakwater structures.

Version 3 of BREAKWAT is an upgrade of Version 2.02, dating from 1993. BREAKWAT 2.02 has been widely used for the design of rubble-mound structures. It was based on studies of wave runup, transmission and structural stability, mostly performed in the 1980's. In the past 10 years new developments in the technical aspects of breakwater design as well as Input/Output and graphical presentation of results have been made. The development of a Version 3 of BREAKWAT incorporates many of those new improvements. BREAKWAT 3 includes most of the options currently in Version 2.02 as well as the following aspects:

- Windows (95/98/NT/2000/XP) based operation
- Possibility to read from input files and to write results to output files,
- Possibility of more than one scenario to be calculated at the same time (sensitivity analysis),
- Improved graphical presentation of results,
- Improved digital and printable user and technical manual,
- New formulae for wave runup, transmission and overtopping of sloped structures,
- Calculation of the failure probability of rock armour layers,
- Possibility to compare various stability formulae for toplayers of rubble mound breakwaters,
- Calculation of longshore transport for berm breakwaters,
- Calculation of rear side stability of rubble mound and berm breakwaters,
- Calculation of stability of near bed structures,
- Calculation of stability and overtopping of caisson breakwaters.

1.2 Reader's guide

This manual basically consists of two parts (see also the Contents of this document):

1. a first part with a description of the functional usage of the program; the User Manual, and
2. a second part with a technical description of the applied formulae; the Technical Manual.

1.3 Additional information

For additional information on BREAKWAT or on other (software) products developed by WL | Delft Hydraulics, please contact: delftchess.info@wldelft.nl, or visit the website: www.wldelft.nl/soft/chess/breakwat/index.html.

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2 User Manual

2.1 Introduction in User Manual

The User Manual provides a description of how to use the program. In this part of the manual some program-specific terms are defined, as is the program structure, the general screen layout, the graphical capabilities and the I/O procedures. At the end of this part of the manual an example is given for a specific calculation, including a tailor-made graphical presentation of the results.

2.2 General setup and definitions

BREAKWAT 3 can be used as a design aid to calculate a number of aspects of breakwater structures. The software is Windows based and can run on the platforms 95/98/NT and XP. This manual does not describe the standard Windows applications which are included in BREAKWAT 3. Other program-specific applications are described in detail. This includes the following aspects:

- Screen layout
- Program specific terms defined
- Program structure
- Cases and Projects

At the end of this chapter an example is provided for a single calculation scenario with graphical output.

2.2.1 Program specific terms defined

Calculation scenario

These terms pertain to a single aspect to be calculated and includes the structure type, the selected item to be computed, the input parameters and the formula to be used.

Structure type

Structure types included in the program are:

- Conventional rubble mound breakwaters
- Berm breakwaters
- Reef-type structures
- Near-bed structures
- Vertical (caisson) breakwater

The structure type can be chosen in two ways. The first way is by selecting ‘New Case’ (see Cases and Projects) in which the list of available structure types is presented directly. After making a selection a ‘Case’ window is opened containing all relevant information. The second way of selecting/changing the structure type is from the ‘Case’ window, by selecting the field ‘Structure type’. A pull-down menu will appear with a list of available structure types the desired option can be selected

Then, for each structure type a number of response factors can be calculated.

Response factors

Once a structure type has been selected you can choose which response factor you want to calculate. Response factors are subdivided into 2 categories, hydraulic response factors and structural response factors.

Hydraulic response factors include:

- wave height distribution,
- wave runup,
- wave overtopping,
- wave transmission, and
- wave reflection.

Structural response factors include:

- stability of rock or concrete armour layers,
- toe berm stability,
- profile development of berm breakwaters,
- longshore transport of berm breakwater armour,
- rear side stability of berm breakwaters,
- pressures and forces on a vertical caission,
- safety factors against sliding and overturning of a vertical caisson, and
- toe berm stability in front of a vertical caisson.

The response factor can also be chosen in two ways. The first way is by selecting 'New Case' (see Cases and Projects) in which the list of available structure types and response factors is presented directly. After making a selection a 'Case' window is opened containing all relevant information. The second way of selecting/changing the response factor is from the 'Case' window, by selecting the field 'Response factor'. A pull-down menu will appear with a list of available response factors and the desired option can be selected.

Formula

For each response factor 1 or more formulae can be applied to make the calculation. After opening a 'Case' window the formula to be applied can be chosen/changed by selecting the field 'Formula'. A pull-down menu will appear with a list of available formulae and you can select the desired option.

Output parameter

For some formulae the user has the option of choosing which parameter you want calculated. The list of input and output parameters is therefore dependent on that choice. After opening a 'Case' window the formula to be applied can be chosen/changed by selecting the field 'Output parameter'. A pull-down menu will appear with a list of available parameters and the desired option can be selected.

2.2.2 Program structure

The technical contents and structure of BREAKWAT 3 are shown in Overview of structure types and response factors. Each item in this structure is identified by a number, for example:

Each item in the program structure is identified by a number. The main classifications are:

1 Rubble mound structure and
2 Vertical (caisson) structure.

For both classifications various structure types are defined, for example:

1.1 Conventional breakwater
1.2 Berm breakwater.

For each structure type either structural or hydraulic properties can be calculated, for example:

1.1.1 Hydraulic response.

Then, for each property a selection from a number of options (response factors) can be made. For some of the response factors more than one formula can be applied. The desired formula can be chosen from the list of available options. Lastly, for the selected formula, the desired output parameter can be selected.

This structure is summarised as follows:

1 Rubble mound = Main structure classification
1.1 Conventional breakwater = Structure type
1.1.1 Hydraulic response = Property (hydraulic/structural)
1.1.1.1 Wave height distribution = Response factor to be calculated

For some of the different structure types the items under 'Hydraulic response' are identical.

2.2.3 Cases and Projects

With BREAKWAT 3 data can be stored in files for future use. For this purpose the file structure makes a distinction between a 'Case' and a 'Project'.

A 'Case' is a single calculation scenario (eg. Rubble mound breakwater, Armour stability with the formula of Hudson).

A 'Project' is a group of calculation scenarios, usually related to a certain project, (eg. armour stability, overtopping and transmission calculations).

A 'Case'-file is stored with file extension filename.BWC.
A 'Project'-file is stored with file extension filename.BWP.

The user defines the filename.

2.3 Getting started

After installation of BREAKWAT 3 on the user's PC the program can be started by clicking the breakwat.EXE file in the program subdirectory or by defining and activating a short-cut icon on the desktop. After confirming authorisation from the licence file the start-up screen will appear.

2.3.1 Screen lay-out

Upon starting BREAKWAT 3 the start-up screen is displayed. From this screen you have a number of options as to how to proceed. For example, you can start a new calculation for a 'Case' or a 'Project' or you can open an existing data file for a 'Case' or 'Project' (see Cases and Projects for more information on 'Cases' and 'Projects').

The start-up screen contains the following fields, the title bar, the menu bar and the contents window, in which the list all active cases and projects are displayed see Figure 2. Beside the contents window are 4 control buttons for managing the cases and projects. The title bar shows the program name and version number.

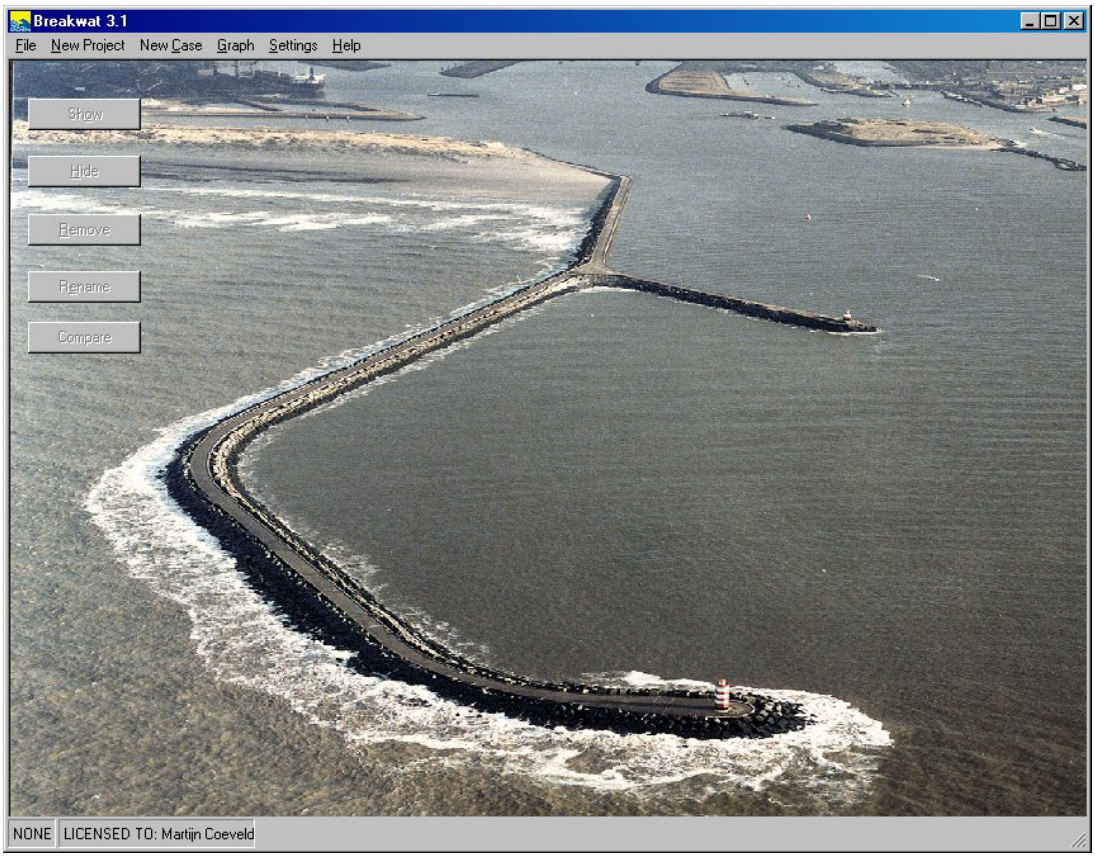


Figure 2 Start-up screen

The menu bar at the top of the screen has a number of items. These items are:

- File
- New Project
- New Case
- Graph
- Settings
- Help

2.3.2 File

Under this option you can perform the following operations:

Open
To open an existing 'Project' or 'Case'.

Close project
To close a current 'Project'.

Save
To save the active 'Case' or 'Project'. The output file will have the name of the active window and extension .BWC or .BWP for 'Cases' and 'Projects', respectively. The file is written to the directory defined in the settings option (see Settings).

Save as
To save a 'Case' or a 'Project'. You will be asked to specify the filename and can choose which directory to save it in. A 'Case' is saved with extension .BWC and a 'Project' with extension .BWP.

Save all
To save all open 'Cases' and 'Projects'. Each 'Case' or 'Project' will be saved with the filename of the Window name, with file extension .BWC or .BWP for 'Cases' and 'Projects', respectively. The files will be written to the directory defined in the settings option (see Settings).

Print to Editor
To print the contents of the active 'Case' to a text file. The text file will be opened with the editor defined in the settings option (see Settings).

2.3.3 New Project

A 'Project' is a group of 'Cases', usually related to a certain project, (eg. armour stability, overtopping and transmission calculations).

Under this option you can define a new 'Project'. With New Project all the input and output of all separate cases defined under the 'Project' can be saved in the output file:

filename.BWP.

The 'Project' can contain 1 (a 'Project' contains after creation default 1 'Case') or more 'Cases', which the user can later define (see New Case). After clicking on New Project at least 1 new 'Case' must be opened in order to perform a calculation. To view this 'Case' double click the 'Project'.

To save the data from the calculation scenarios in the 'Project', choose File from the menu bar and then either:

Save, or
Save as > Project

By later opening the 'Project'-file all of the data can be retrieved.

Note: 'Case'-files that are not opened within a 'Project' cannot be later saved under a 'Project'-file.

2.3.4 New Case

A 'Case' is a single calculation scenario (eg. Rubble Mound Breakwater, Armour stability with the formula of Hudson). The 'Case' can be defined either as a stand-alone calculation scenario or as part of a 'Project' (see New Projectfor 'Project' definition).

Under this option the user can select a new calculation scenario. Upon clicking on New Case a list of structure types is presented, from which a selection can be made. The list of available structure types is:

- Conventional rubble mound breakwaters
- Berm breakwaters
- Reef-type structures
- Near-bed structures
- Vertical (caisson) breakwater

For each of these structure types a number of different response factors can be calculated. Each new 'Case' defined is stored in a separate window. Each of these case windows has a name, the default name is 'Breakwat #1', 'Breakwat #2', etc., which is shown on the title bar of each window. Each 'Case' name is also displayed in the Contents Window on the screen. If more than one 'Case' is created, the user can activate any given 'Case' by clicking on the desired 'Case' description in the Contents Window.

After selecting the desired 'Case' the desired 'Formula' to be applied (for situations where more than 1 formula is available) and the desired 'Output parameter' (for situations where more than 1 output parameter can be selected) can the be chosen by clicking on the relevant fields (see Input fields for more details).

With New Case all the input and output of the defined 'Case' can be saved in the output file:

filename.BWC

By later opening the 'Case'-file all of the data can be retrieved.

Note: 'Case'-files that are not opened within a 'Project' cannot be later saved under a 'Project'-file.

2.3.5 Graph

After a calculation has been performed and at least 1 input parameter has been entered as an array (see Section Input arrays for information on input arrays) it is then possible to create a tailor-made graph of the results. This can be accomplished by clicking on the 'Graph' option of the menu bar. Upon clicking this option a schematic graph will appear on the screen, see Figure 3. The user can then select (confirm or change) the desired parameters for the x and y axis. If 2 input parameters have been defined as arrays, then the choice of the z parameter can also be made. If more than one calculation scenario ('Case') is active then the user must also select the appropriate case window for the graph (see Example calculation). Default is the active 'Case' at the time of selecting the 'Graph' option. For some calculation scenarios the user may desire to plot the x- axis as a Rayleigh distribution. In this case check the box marked 'Rayleigh distribution' on the lower left corner of the graph definition screen.

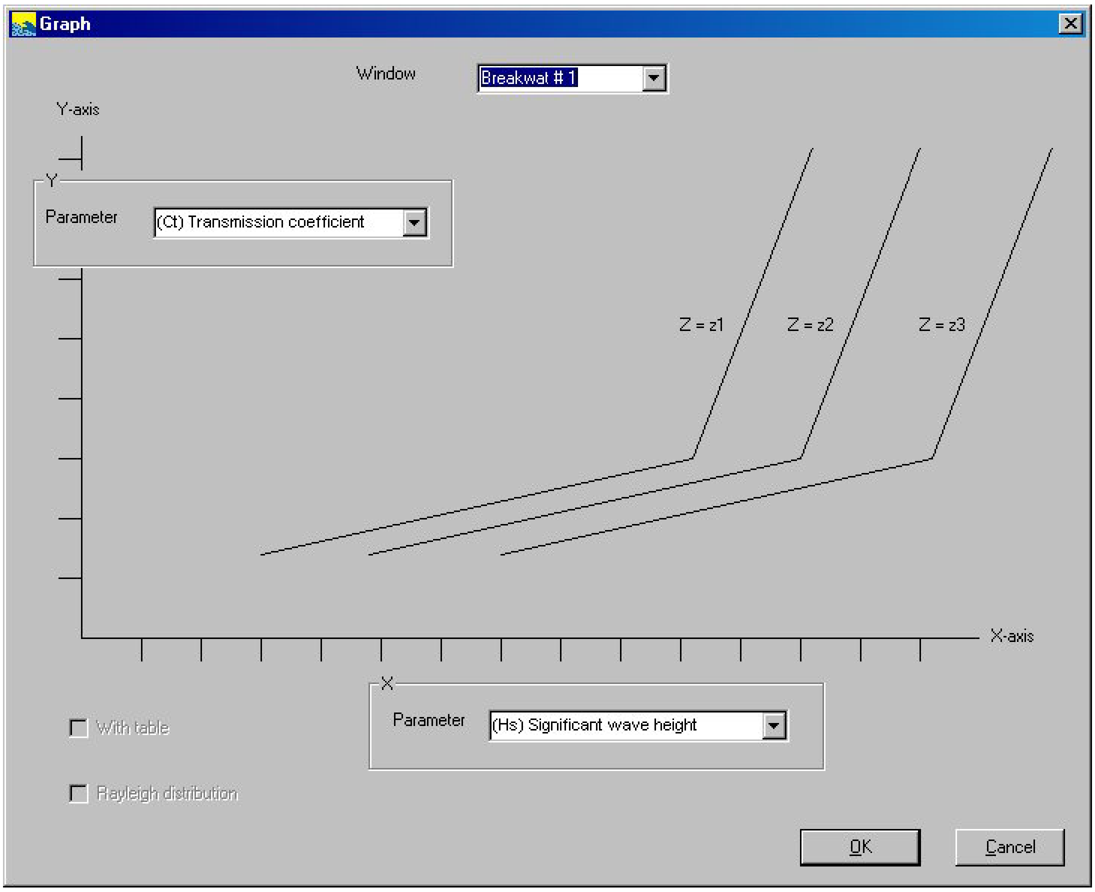


Figure 3 Graph definition screen

The results will then be displayed in a separate graphical window. Upon closing that window the graphical data is stored and shown as an extra button at the bottom of the output parameter list and the graph can be recalled at any time. When any of the input parameters are changed the output parameters and the graph are automatically updated.

When the calculation scenario is saved as either a 'Case' or a 'Project' the graphical data is also saved.

2.3.5.1 Embellishment of graphs

It is possible to change the appearance of almost all aspects of a graph created by the Graphical Editor (by doubleclicking on the graph an Edit window will appear or select File and then Chart Properties), see Figure 4. Among other things, the following aspects can be changed:

- series titles
- point markers - shape, size, colour
- lines between points - colours, thickness, style
- axis labels and titles - text, font style, size, colour
- legend - format, position
- data labels
- x, y axis types - linear, logarithmic, inverted
- plot area - layout, colour.

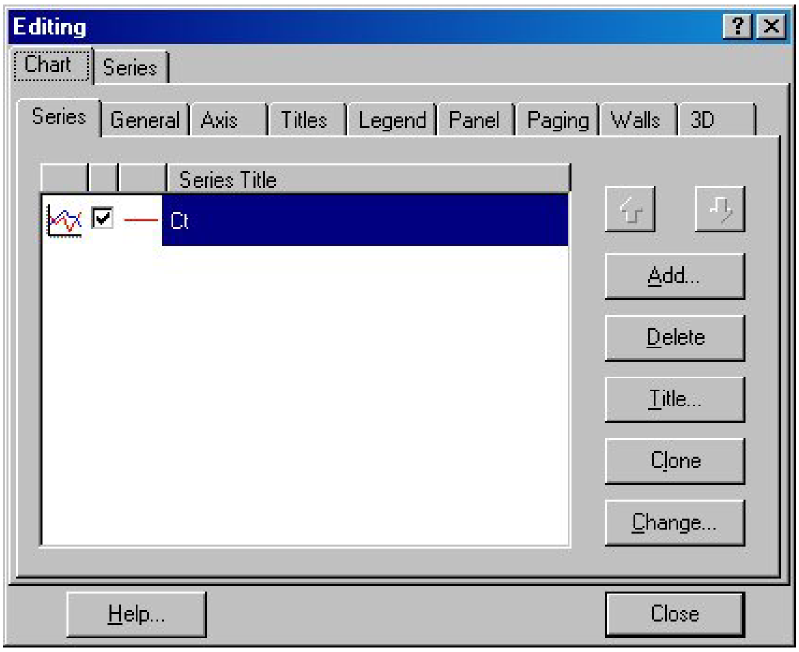


Figure 4 Graphical Editor command window

This window has two main tabs:

Chart
Series

Under the Chart tab, the general properties of the graph layout can be edited:

Series
Shows the series which the graph contains.

General
General chart features, such as 'Print preview', 'Chart export', 'Margin settings' and 'Zoom settings'.

Axis
Axis settings including 'Scales', 'Log. or inverted axis', 'Titles', 'Labels', 'Tick mark positions'. Axes can be defined on the left, bottom, top or right side of the graph, as well as 'Depth' (for 3D views).

Titles
Graph 'Title' and/or 'Footnotes' can be inserted, including the 'Font type', 'Colour', 'Background colour' and 'Alignment'.

Legend
Graph 'Legend' can be inserted, including the 'Position', 'Font type', 'Colour', 'Background colour' and 'Alignment'.

Panel
Plot area parameters, 'Colour', 'Gradient' (between 2 colours can be inserted), 'Frame' ('Bevel Outer/Inner'), or an 'Image' can be inserted as the 'Background' to the chart.

Paging
The graph can be displayed over more than 1 page, each page containing a specified number of data points. Entering 0 points per page (default) means that the entire graph is displayed on 1 page.

Walls Left, bottom and backwall size and colour definitions.

3D Enable/disable and modify 3D display parameters ('Zoom', 'Rotation', 'Elevation', 'Offsets', 'Perspective' and 'Orthogonal' view toggle.

Under the Series tab (see Figure 5), the general properties of the graph layout can be edited:

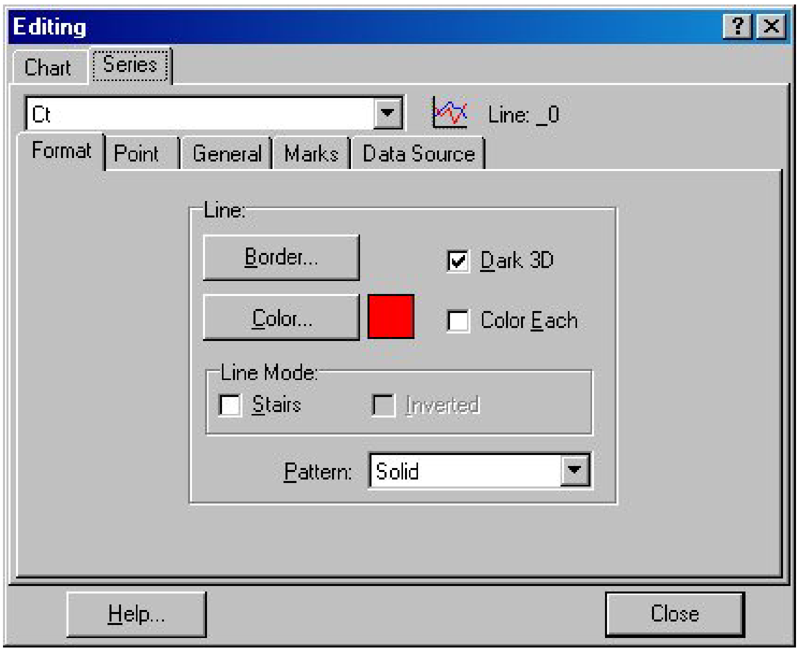


Figure 5 Series tab in the Graphical Editor

Format
Defines the format for the line type, thickness and colour for the selected Series (displayed in the property window with pull-down menu in the upper left corner). The 'Line type', 'Line colour' and 'Line thickness' are defined under the Border button. Thick lines can also be defined as a 'Pattern'.

Point
Defines the format for the data point markers. To show or remove data point markers, check the 'Visible' field. The 'Style', 'Background', 'Colour', 'Height' and 'Width' of the point markers can be defined.

General
Defines general layout aspects for the selected series (displayed in the property window with pull-down menu in the upper left corner). Whether the series is to be shown in the legend, the axis on which the series data is to be plotted and value formats can be defined in this window.

Marks
'Data labels' can be plotted alongside the data points. To show or remove data labels, check the 'Visible' field. The 'Background colour', 'Font' and 'Border type' can be specified, as can the 'Length' and 'Colour'of the 'Arrows' which connect the data labels to the data point.

Data Source
Random Values must be specified in the input field for all Series. The other options in the input field are not applicable to BREAKWAT 3 applications.

2.3.5.2 Copying other data to an existing graph

In addition, it is possible to copy data from other output parameters from the same ‘Case’ and from other ‘Cases’ to the graph. To do this simply click on the output button of the desired parameter and drag it to the graph. This does not work if the x-axis is drawn as a Rayleigh distribution. In that case, however, it is possible to drag data from a separate graph with a Rayleigh x-axis to the current graph by clicking and dragging the ‘graph’ button to the current graph.

2.3.5.3 Copying data to and from the clipboard

It is also possible to add data from an external spreadsheet to the graph. To do this select the range of data points to be plotted from the spreadsheet. Two or more adjacent columns must be selected, having the x values in the first column and the y values in the following columns. The first row of data is taken to be a title and is not plotted as data. Therefore, ensure that the first real data point is in the 2nd row of the array. The data is, however, pasted as separate values and is not linked to the spreadsheet.

Graphical data can also be copied to the clipboard, to be pasted into a spreadsheet for example. To do this choose File on the graph menu bar and then copy data to Clipboard (Paste in Excel).

The data can then be pasted into a spreadsheet. Similarly, the entire graph can be pasted into an (Excel) spreadsheet by choosing File on the graph menu bar and then copy chart to Clipboard.

The entire graph is then inserted as a picture into the spreadsheet.

2.3.5.4 Graphical templates

It is possible to save the graph appearance (colours, series definitions, etc.) in a template file, to be applied to other graphs or at a later time. This option is accessed by selecting the

Template
option in the graph Menu Bar. A list of available templates is presented and the template which is currently applied is checked. The user can either select a different template from the list or you can define a new template by choosing the option

Template Manager
By selecting this option the layout of the current graph can be saved as a named template. The user therefore has to enter the desired name for the new template in this option. To make the new template directly also the default template click the button

Set as default.

2.3.5.5 General Graph Information

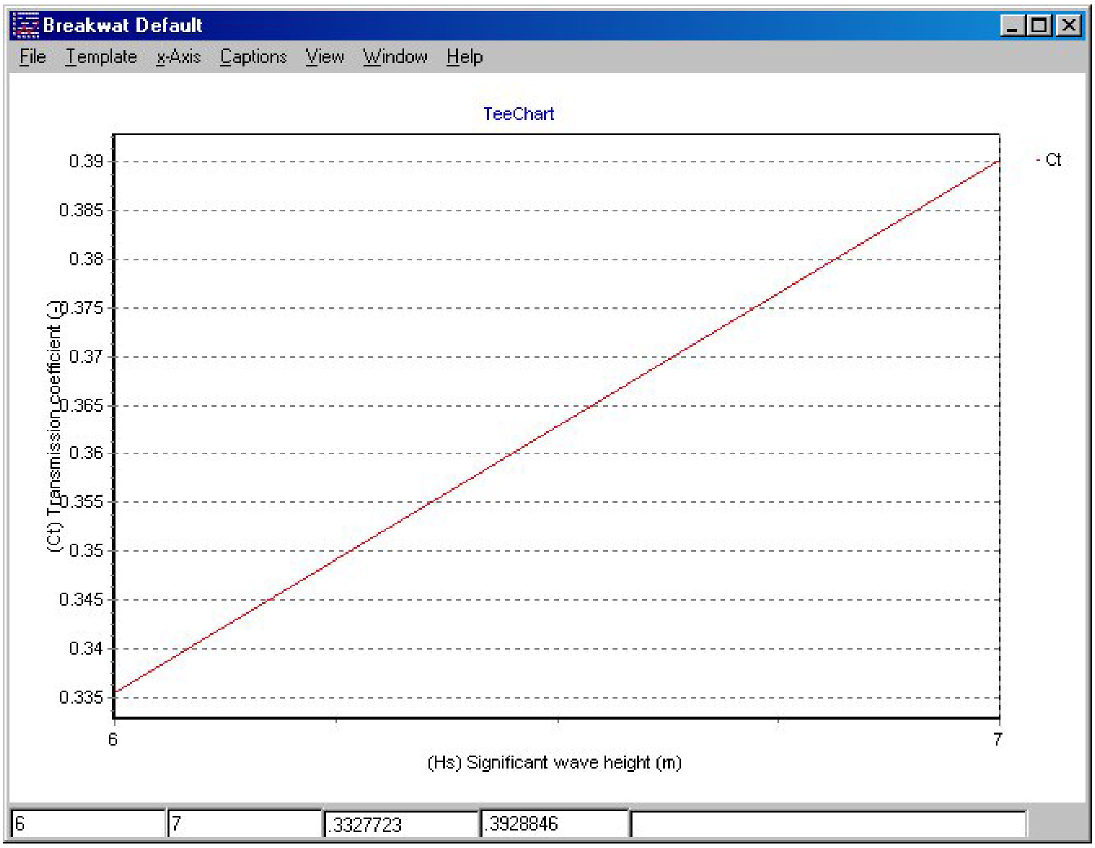
Navigating through the graph

Zoom-in
Move your mouse in the graph area. Press the left mouse button and keep it pressed while your drawing a rectangle to indicate the area of interest. Release the mouse button when you're finished. The system will zoom in on the section of the graph you specified.

Zoom-out
To zoom out select `UNZOOM` from the `VIEW` menu, or use the shortcut `<CTRL>-U`. The graph will be displayed in its original form.

Scrolling the graph
You can scroll through your graph. To do so place the mouse pointer inside the graph area. Press the right mouse button and keep it pressed while you move your mouse. To stop scrolling release the mouse button.
This can be useful when you are zoomed in and want to see more of the graph at the same zoom level.

Zoom navigation bar
You can edit the axis-values by means of the Chart Properties menu. However, there is an easier way to do this. Selecting `ZOOM NAVIGATION BAR` from the `VIEW` menu (shortcut `<CTRL>-N`) will display the Zoom Navigation Bar at the bottom of your Graph Window.



- ***Minimum value of bottom axis***

Here the minimum value of the bottom axis is displayed. If the number is in black it is the minimum value calculated by automatic scaling. Select the number and type your own value to change the minimum value of the bottom axis. Note that this number is displayed in blue. To change the value to the minimum value calculated by automatic scaling, simply double click the number in the box.

- ***Maximum value of bottom axis***

Here the maximum value of the bottom axis is displayed. If the number is in black it is the maximum value calculated by automatic scaling. Select the number and type your own value to change the maximum value of the bottom axis. Note that this number is displayed in blue. To change the value to the maximum value calculated by automatic scaling, simply double click the number in the box.

- ***Minimum value of left axis***

Here the minimum value of the left axis is displayed. If the number is in black it is the minimum value calculated by automatic scaling. Select the number and type your own value to change the minimum value of the left axis. Note that this number is displayed in blue. To change the value to the minimum value calculated by automatic scaling, simply double click the number in the box.

- ***Maximum value of left axis***

Here the maximum value of the left axis is displayed. If the number is in black it is the maximum value calculated by automatic scaling. Select the number and type your own value to change the maximum value of the left axis. Note that this number is displayed in blue. To change the value to the maximum value calculated by automatic scaling, simply double click the number in the box.

- ***Change series title***

In this box you can change the title of a series in your graph. Click with your mouse in the box to make it ‘active’. Now move to the series you want to change the title for. You can move to the series in the graph or in the legend. Press <SHIFT> and move your mouse over the series. Now the default title will be displayed in the box. Now you can change the title by typing a new title in the box.

The Caption - menu

Introduction

The captions menu determines how the legend and axes titles will be displayed. The following sub menus are available:

- Relative/Absolute
- Reset to default

Relative/Absolute

Determines whether to display relative or absolute labels. As a user you’re normally interested in relative labels. Only people designing templates will use absolute labels.

Reset to default

This option will reset the labels to the default values.

The File - menu

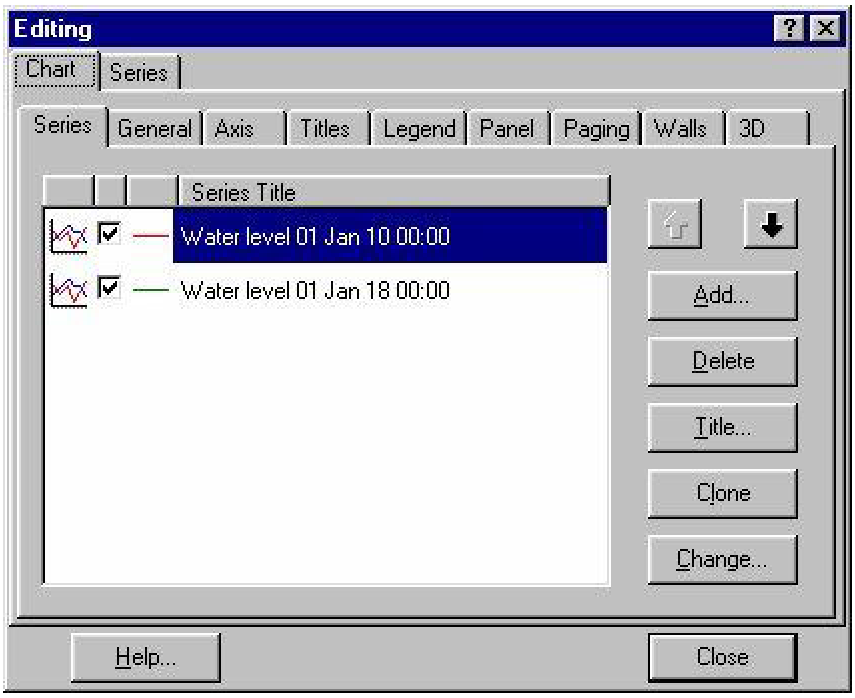
Introduction

The `FILE` menu of Delft Graph Server consists of the following sub-menus:

- Chart Properties
- Export
- Copy to Clipboard
- Refresh
- Cancel
- Print
- Exit

Chart Properties

The `CHART PROPERTIES` menu will display the chart editor window, as given in the figure below.



The Chart Editor is designed to help you quickly create and modify Charts.

- ***Editor design***

There are 2 principal sections to the Chart editor, Chart parameters and the Series parameters, which are separated as 2 tabs of the Chart Editor.

- ***Chart pages***

You may define overall Chart display parameters at any time before, during or after adding Series to the Chart. Chart parameters are divided into the following sections:

- ***Series page***

You may add a mixture of different Series types to the Chart to define the specific Chart of your choice. Note here that you are not limited to predefined Chart types. Most Series types are compatible with other Series types on the same Chart, those Series types not available are greyed out. To add a new Series to a Chart select the Add button on this page which will display the Chart Gallery. Select the Series type of choice from the gallery and it will display on the Series page of the Chart Editor.

- ***General Page***

Margins, Zoom and Scroll, Print Preview and Export.

- ***Axis Page***

All Axes definition. Some parameters depend upon the Series associated with the axis, for example, Datetime depends on whether the Series data has date time definition, this can be configured on the Series 'General' page of the Series concerned.

- ***Titles Page***

Teechart Header and Footer

- ***Legend Page***

Legend display. Formatted displays work in conjunction with the Chart Series. See also the 'General' page of the Series.

- ***Panel Page***

Chart Panel display properties. Colours, Bevels, Backimages, Colour Gradient and Border.

- ***Paging Page***

Definition of number of points per chart page. May be used to browse at design time, too, if your data is sourced from an ODBC datasource.

- ***Walls Page***

Left, Bottom and Backwall size and Colour definitions.

- ***3D Page***

Enable/disable and modify 3D display parameters.

- ***Series Pages***

Series pages will contain parameters dependant on the series type concerned. The Combobox at the top of the Series tab page shows which series you are editing.

- ***Format Page***

Contains Series type specific parameters.

- ***Point Page***

Contains settings (visibility, size, style, colour) of series Points.[only available for certain chart types, e.g. line]

- ***General Page***

Series value format, Axis association

- ***Marks Page***

Series Mark format, text, frame and back colour and positioning.

- ***Data Source Page***

Access to Function definition and ODBC data sourcing

Export

The `EXPORT` menu will give the export window as given in the figure below.

- ***Copy to clipboard***

This will place the current graph in the windows clipboard (see also section 2.4).

- ***Save to File***

This will save the current graph as a file which name has to be specified by the user. The format of the file is determined by the checked option under "format".

- ***Close***

This will close the export window.

Copy to clipboard

The `COPY TO CLIPBOARD` menu copies the present graph to the windows clipboard. You can paste it in any other document by using <<ctrl-V>>.

Refresh

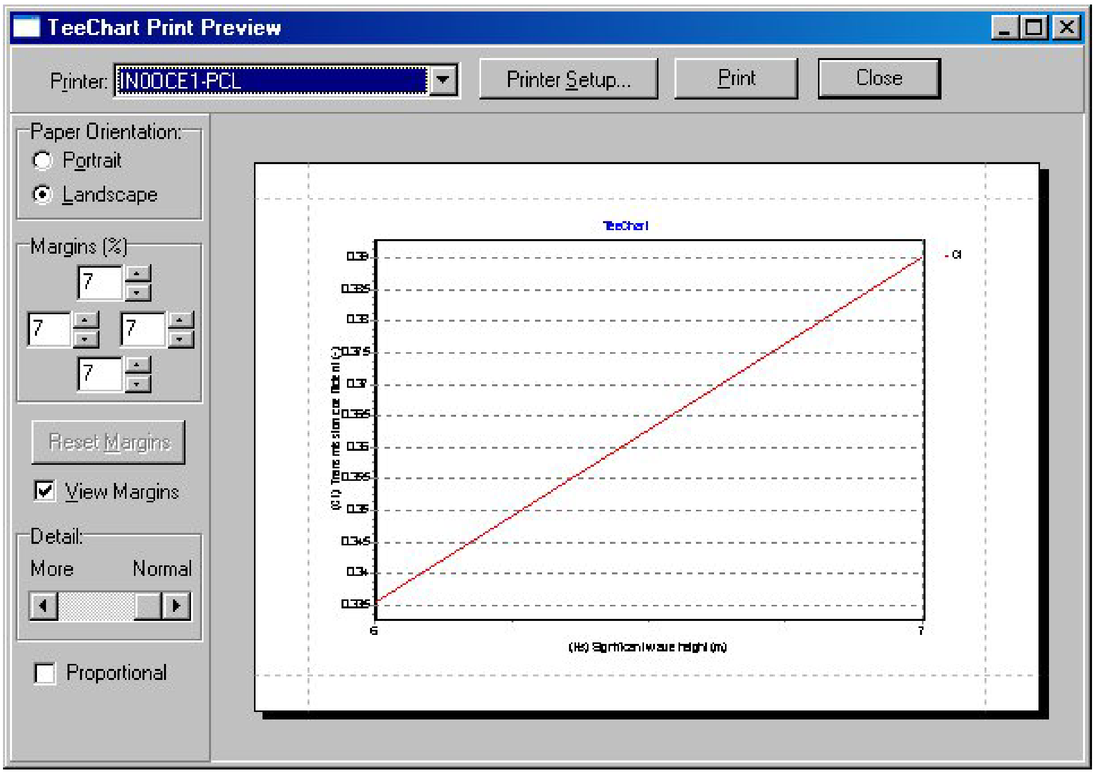
The `REFRESH` menu will refresh the screen.

Cancel

The `CANCEL` menu will stop drawing the graph series.

Print

The print menu will display the print window as given below.



- ***Printer Setup***

Will display properties of the printer chosen.

- ***Print***

Will print the actual lay-out

- ***Close***

Will close the printer menu.

Exit

The `EXIT` menu will stop the Graph Server and return control to the program that called the Graph Server.

The Template -menu

Introduction

The template menu of the Graph Server gives access to the following sub-menus:

- Template Manager
- Save
- Reload

Template Manager

- ***Name***

Changes name of selected template.

- ***Description***

Changes description of selected template.

- ***Visibility***

Determines whether a template should be global (available in every program that calls the Graph Server) or local.

- ***Add New***

Adds a new template.

- ***Remove***

Removes the selected template

- ***Copy***

Copies the selected template to a new template

- ***Set as Default***

Set the selected template as default template

- ***Load***

Loads the selected template and applies its settings to the present graph. The loaded template becomes the active template.

- ***Save***

Saves the present settings of the graph to the selected template. The selected template becomes the active template.

- ***Import***

Imports a teechart template file (*.tee).

- ***Export***

Exports the selected template to a teechart template file (*.tee).

- ***Cancel***

Quits the template manager without saving the changes made to the templates.

- ***OK***

Quits the template manager after saving all the changes made to the templates.

Save

Saves the present settings of the graph to the active template.

Reload

The active template will be reloaded.

The Window - menu

Introduction

The Window menu consists of two sub menu:

- Save Window Position
- Autosave Window Position

Save window position

This option will save the current position of the window. Next time the graphserver will pop up at the same position.

If the sub menu `AUTOSAVE WINDOW POSITION` is activated, the position settings will be overruled at the moment you close the graphserver

Autosave window position

This option automatically save the window position at the moment the program will shut down. Next time the graph server is start, the window will pop up at the same position.

The X-axis -menu

Introduction

The X-axis menu enable the user to manipulate the x-axis of the graph. It consists of the following sub menus:

- Time / Location / Parameter / Auto
- Label / Values
- Dynamic Date Time Format

Time/Location/Parameter/Auto

This sub menu determines which element will be displayed on the x-axis. This will be either Time, Location or Parameter. Auto let the template decide which element to display on the x-axis.

Labels/Values

This sub menu determines whether labels or values will be displayed as x-axis units.

Dynamic Date Time Format

Use this option for long time series. It will label the timesteps with intelligence. If you have for example a time series with a 6 hour interval starting at March 1, 12.00 hour. The first label will be *12.00*; the second *18.00*; the third *March 2*; the fourth *06.00* etc.

The View -menu

Introduction

The VIEW menu contains some shortcuts to manipulate the way the graph is displayed. The following sub menus are available:

- Unzoom
- Show all series
- Remove all series

Unzoom

Will return to the original form of the graph.

- ***The Zoom Navigation Bar***

You can edit the axis-values by means of the Chart Properties menu. However, there is an easier way to do this. Selecting ZOOM NAVIGATION BAR from the VIEW menu (shortcut <CTRL>-N) will display the Zoom Navigation Bar at the bottom of your Graph Window.

- ***Minimum value of bottom axis***

Here the minimum value of the bottom axis is displayed. If the number is in black it is the minimum value calculated by automatic scaling. Select the number and type your own value to change the minimum value of the bottom axis. Note that this number is displayed in blue. To change the value to the minimum value calculated by automatic scaling, simply double click the number in the box.

- ***Maximum value of bottom axis***

Here the maximum value of the bottom axis is displayed. If the number is in black it is the maximum value calculated by automatic scaling. Select the number and type your own value to change the maximum value of the bottom axis. Note that this number is displayed in blue. To change the value to the maximum value calculated by automatic scaling, simply double click the number in the box.

- ***Minimum value of left axis***

Here the minimum value of the left axis is displayed. If the number is in black it is the minimum value calculated by automatic scaling. Select the number and type your own value to change the minimum value of the left axis. Note that this number is displayed in blue. To change the value to the minimum value calculated by automatic scaling, simply double click the number in the box.

- ***Maximum value of left axis***

Here the maximum value of the left axis is displayed. If the number is in black it is the maximum value calculated by automatic scaling. Select the number and type your own value to change the maximum value of the left axis. Note that this number is displayed in blue. To change the value to the maximum value calculated by automatic scaling, simply double click the number in the box.

- ***Change series title***

In this box you can change the title of a series in your graph. Click with your mouse in the box to make it 'active'. Now move to the series you want to change the title for. You can move to the series in the graph or in the legend. Press `<SHIFT>` and move your mouse over the series. Now the default title will be displayed in the box. Now you can change the title by typing a new title in the box.

Show all series

This option will show all available series in the graph.

Remove all series

This option will remove all series from the graph.

2.3.6 Settings

Under the Settings button you will see 3 tabs:

Breakwat

Controls the toggle setting for showing the 'Property Tooltips' or not. 'Property Tooltips' are the messages given on the screen over a particular parameter when you place the cursor over the parameter field. These messages give information on the allowed and recommended values of each parameter

Editor

Gives the possibility of using the built in editor or to define the user's own editor.

Environment

Gives the possibility of using the default path or to define a new path for which all input and output files will be stored in.

2.3.7 Help

Under this option you can perform the following operations:

Contents: to startup the help file

About Breakwat: general information.

2.4 Input/output fields

When a certain response factor has been selected a new window is opened with the list of input and output parameters. Also a figure is displayed in the lower left corner of the screen showing the definition of the most important parameters.

- Structure type
- Response factor
- Formula
- Output parameter

2.4.1 Default values

For each input parameter for each formula a default value has been entered. These default values can be changed by placing the cursor over the desired field and entering a new value.

2.4.2 Input fields

The fields for the necessary input parameters for the chosen calculation are shown in white. In some cases (if more than 1 formula exists for the selected computation) some input parameter fields are shown in grey. These grey fields are for parameters associated with another formula and are irrelevant for the chosen formula. The user cannot input values for the grey fields.

2.4.3 Input parameter ranges

When the cursor is placed over a particular input parameter field a message is shown stating the allowed and for the recommended minimum and maximum values for the given parameter. For each input parameter there are thus 2 limits.

Hard limits
Maximum and minimum allowed values. If the value entered is not within these limits, it is not accepted. A window appears showing the maximum or minimum acceptable values and the user has to re-enter the value before a calculation is performed, see Figure 6.

Soft limits
Maximum and minimum recommended values. These are the limits within which the chosen formula has been validated for the chosen parameter. If a value is entered beyond these limits a warning message is issued at the bottom of the 'Case' Window, showing the parameter which is out of range of the recommended limits and its value. This is also the case for derived parameters, calculated based on the given input. Output values are still calculated in this situation. **However, the user must be aware that in this case the results are beyond the tested limits of the chosen formula and may therefore not be reliable.**

Figure 6 Example of warning message for a situation where the input parameter P is outside of the validated limits (hard limits) of the formula

2.4.4 Output parameters

Upon entering any input parameter the formula is recalculated and the results are shown immediately in the output parameter field.

2.4.5 Input arrays

For most of the input parameters the user has the option of entering an array of values instead of a single value. Output parameters are accordingly calculated for each value of the input array. A limitation of this option is that a maximum of 2 input parameters can be entered as arrays. The input array option is activated by checking the white square field to the right of the input parameter field for the chosen parameter, see Figure 7.

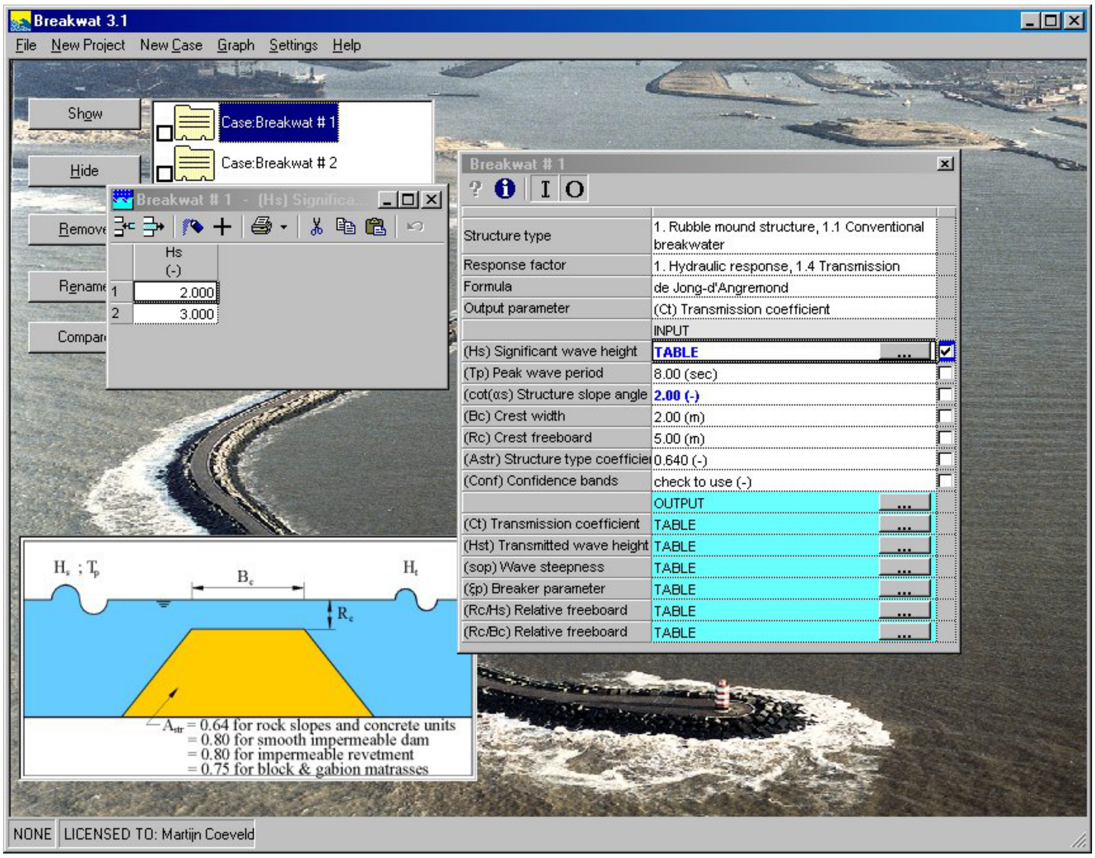


Figure 7 Data array input window appears after clicking on the grey button of the checked input parameter

A new button then appears in the input parameter field. By clicking on this button the input table appears. A maximum of 40 values per array can be entered.

A number of buttons to aid in the data entry are available at the top of the window. Most of these buttons are self explanatory but the first four are explained below.

'Insert Row' button
To add a new value (row), can also be used to add a row in between existing values.

'Delete Row' button
To delete a value (row).

'Fill Range' button
To facilitate the data entry you can enter a value at Position 1 (or any position for that matter) and then press 'Enter' for up to 39 additional places and then an end value can be entered. By then pressing the 'Fill Range' button the intermediate values are filled in automatically by linear interpolation between the first value and the end value entered.

'Operation' button
The value in the selected cell can be changed performing an arithmetic operation. The value can be changed by addition, subtraction, multiplication and division of a given input value. This operation can also be performed to a range of cells, if so selected.

2.4.6 Confidence bands

For some formulae there is an option to compute the values of the confidence bands. This is seen as an extra input parameter, 'confidence bands' array. If this option is clicked then only 1 other input parameter can be entered as an array.

2.4.7 Output arrays

When an input parameter is entered as an array of values, the output parameters are accordingly listed as arrays. The values can be viewed by clicking on the grey 'Table' button at the desired output parameter field. By doing this both the table of results as well as a simple chart of the results are shown. The chart can be removed by clicking on the 'Chart' toggle key at the top of the output window.

If 2 different input parameters are entered as arrays the output is presented as a 2 dimensional array by clicking the desired parameter. The results are presented with the first input array values shown as rows and the second input array values shown as columns. The results are also shown in chart form with the first input array plotted on the x axis with a separate series for each value in the second input array.

2.4.8 Compare data in different Cases

When 2 or more 'Cases' have been defined it is possible to make a comparison of all the data between these 'Cases'. This facilitates the comparison of values when, for example, more than 1 formula is available to a given option. The 'Compare' option makes it possible to display all parameters from the selected ‘Cases’ together in a single table, comparable to a spreadsheet. For example, if 2 ‘Cases’ have been defined: Rock Armour stability for Conventional breakwaters; the first ‘Case’ using the Hudson formula and the second ‘Case’ using the Van der Meer formula, the display is as shown in Figure 8. It is desired to make a direct comparison of all the input and output values.

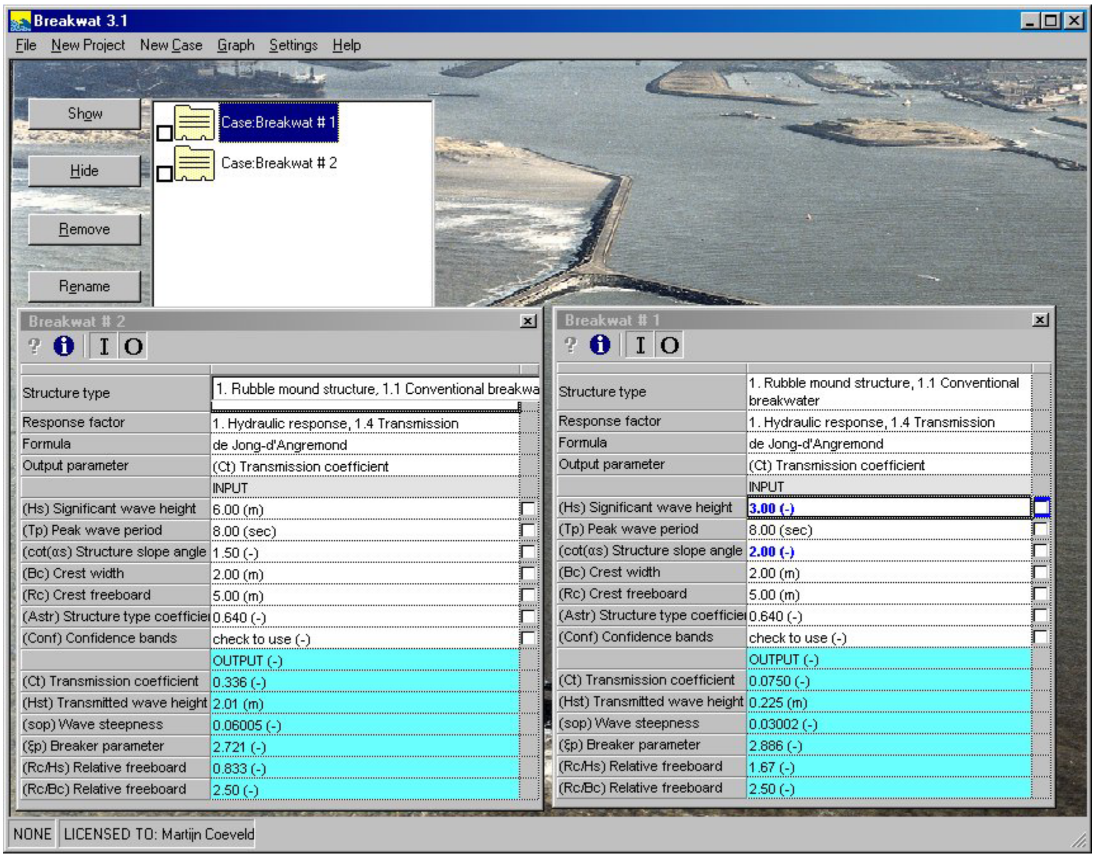


Figure 8 Two ‘Cases’ are compared using the ‘Compare’ option

To make such a comparison, the white square fields to the left of the ‘Case’ names to be compared have to be checked. Then click on Compare beside the Contents Window. A new window is displayed with the name of the active ‘Case’ followed by a definition of the number of cases being compared: Van der Meer (2/2), see Figure 9.

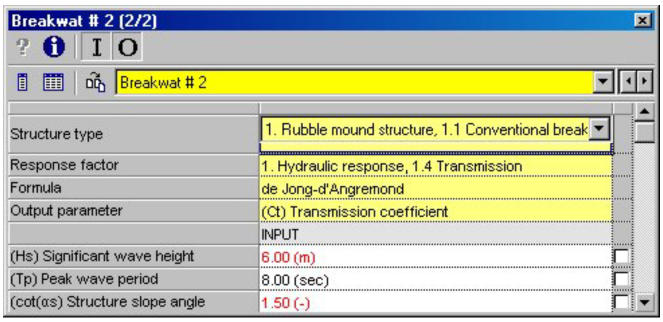
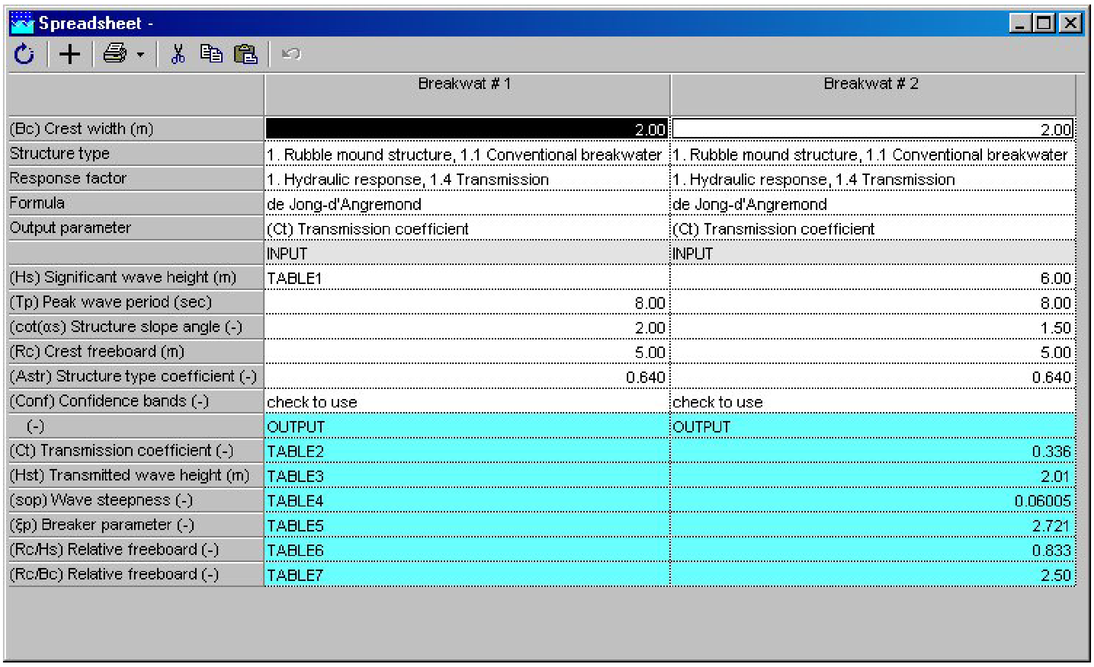


Figure 9 ‘Compare’ window definition screen

All the fields in this window are highlighted in yellow. The user can now choose to either compare individual fields or to compare all fields in the window. To compare all the fields the user has to click on the small rectangular field above the 'Structure type' field. The cursor will be displayed as a cross. After making this selection, click on the 'Spreadsheet' icon. The following window, now entitled 'Spreadsheet -' is displayed, see Figure 10.



	Breakwat # 1	Breakwat # 2
(Bc) Crest width (m)	2.00	2.00
Structure type	1. Rubble mound structure, 1.1 Conventional breakwater	1. Rubble mound structure, 1.1 Conventional breakwater
Response factor	1. Hydraulic response, 1.4 Transmission	1. Hydraulic response, 1.4 Transmission
Formula	de Jong-d'Angremond	de Jong-d'Angremond
Output parameter	(Ct) Transmission coefficient	(Ct) Transmission coefficient
	INPUT	INPUT
(Hs) Significant wave height (m)	TABLE1	6.00
(Tp) Peak wave period (sec)	8.00	8.00
(cot(αs) Structure slope angle (-)	2.00	1.50
(Rc) Crest freeboard (m)	5.00	5.00
(Astr) Structure type coefficient (-)	0.640	0.640
(Conf) Confidence bands (-)	check to use	check to use
(-)	OUTPUT	OUTPUT
(Ct) Transmission coefficient (-)	TABLE2	0.336
(Hst) Transmitted wave height (m)	TABLE3	2.01
(sop) Wave steepness (-)	TABLE4	0.06005
(ξp) Breaker parameter (-)	TABLE5	2.721
(Rc/Hs) Relative freeboard (-)	TABLE6	0.833
(Rc/Bc) Relative freeboard (-)	TABLE7	2.50

Figure 10 Spreadsheet window comparing data from 2 cases

All the information of both cases is presented in the new table. Furthermore, the input values can still be changed in either table and the results will be automatically updated. It is, however, not possible to edit input arrays from the 'Spreadsheet' window.

2.5 Example calculation

This example shows how you can define a ‘Case’, perform a calculation using 1 input parameter as an array, inspect the results and prepare a tailor-made graph of the results.

To define a ‘Case’ click on New Case (for more info on new ‘Cases’ see New Case) in the Menu bar.

Choose Rubble mound 1.1 Conventional breakwater and then Hydraulic response 1.1.1.1 Wave height distribution.

Then the input/output screen for this option is displayed, see Figure 11. The default name of the associated window is ‘Breakwat #1’, but the user can change this name if desired by clicking on Rename.

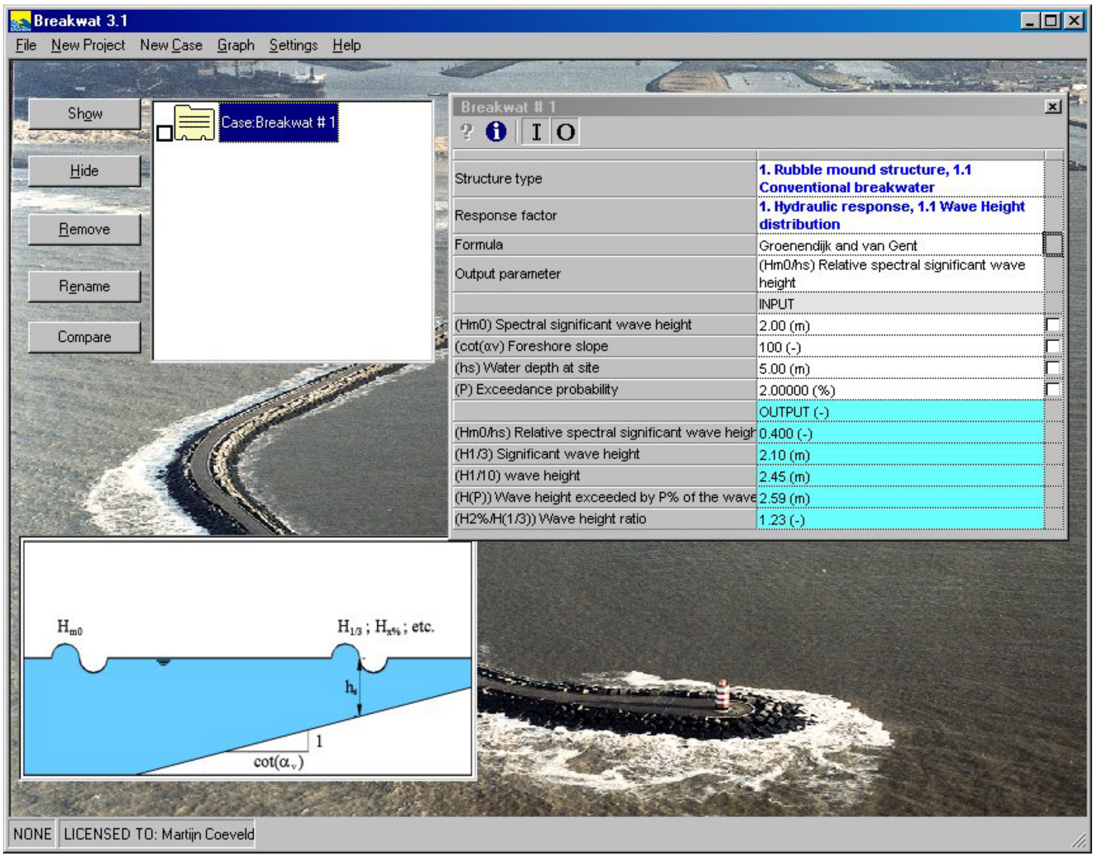


Figure 11 Example calculation for wave height distribution

For this particular case there is only 1 formula and 1 output parameter available, so no other options are listed when the fields ‘Formula’ and ‘Output parameter’ are clicked.

The default values for all input parameters are listed, as are the values of the corresponding output parameters. In this case the output values correspond to a 2% exceedance level. In order to view the entire wave height distribution, check the white square field at the right of the input parameter ‘Exceedance probability’. A default input array of values is read in and the output values are now also displayed in arrays. Click on any of the buttons to view the results for any given output parameter or click on the button in the ‘Output’ field to view all output arrays together.

By clicking on the button for output parameter ‘(H(P)) Wave height exceeded by P% of the waves’ the output values are shown along with a simple graph, see Figure 12.

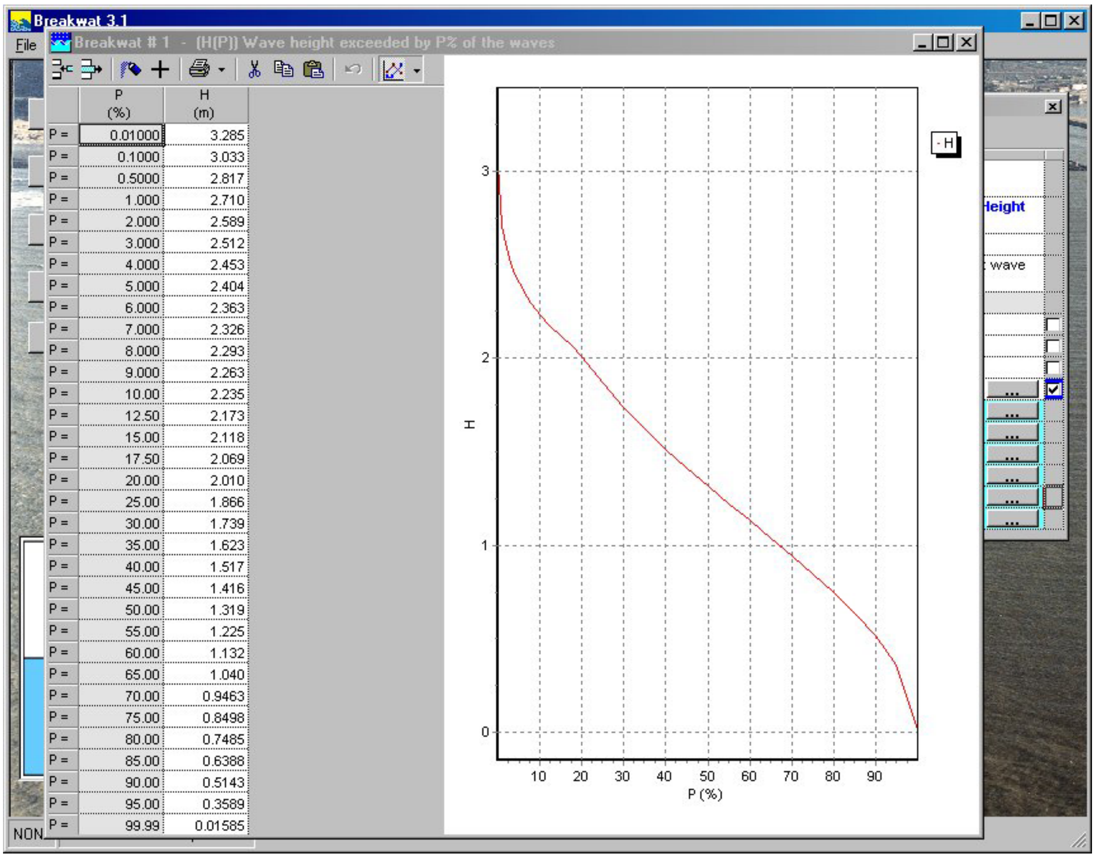


Figure 12 Output values included with a simple graph

All the values are plotted on a linear axis system.

However, the user may wish to prepare a more detailed graph. For example, it is common to plot exceedance probabilities on a Rayleigh axis. This can be done by clicking on the Graph button in the Menu bar at the top of the screen. The schematic graph definition window will appear. The user can now select the desired parameters for the x- and y-axis. Select ‘(H(P)) Wave height exceeded by P% of the waves’ for the y-axis parameter and ‘(P) Exceedance probability’ for the x-axis parameter.
(Because only 1 ‘Case’ has been created, only 1 window (Breakwat #1) can be selected.) Lastly, you can choose whether to define the x-axis values as being Rayleigh distributed by checking the white square box on the left of the window (default setting for this case. If this option is not checked the x-axis values will be linearly distributed).

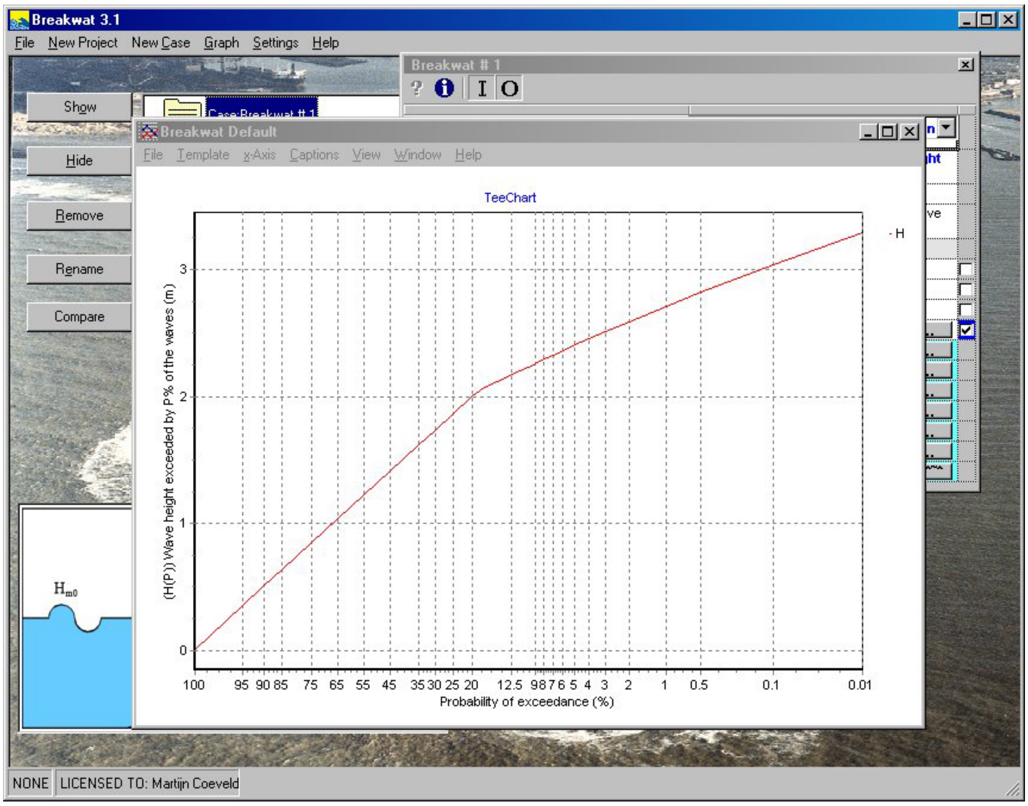


Figure 13 Tailor-made graph using the Graph button in the Menu bar

In Figure 13 the graph is now displayed and a new output parameter 'Graph' has been added to the output parameter field. The graph window can therefore be closed and recalled again at any time by selecting the grey button beside the 'Graph' output field. The user can change the appearance of most aspects of a graph (for more information see [Embellishment of graphs](#)). The graphical editor is activated by double-clicking anywhere on the graph window. The following window is then displayed (see Figure 14):

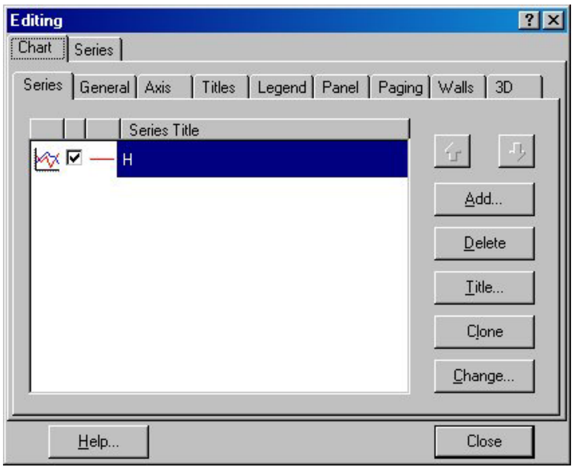


Figure 14 Options in graphical editor

2.6 Overview of structure types and response factors

Table 1 gives an overview of the structure types included in BREAKWAT 3, together with the response factors and formulae.

Rubble mound 1.1 Conventional breakwater

Hydraulic response	1.1.1.1	Wave height distribution	Groenendijk and Van Gent (1999)
Hydraulic response	1.1.1.2	Runup	Runup distribution on plane rock slope (Van der Meer, 1993) Bermed slopes (TAW, 2002) Bermed slopes (Van Gent, 2001)
Hydraulic response	1.1.1.3	Overtopping	Bermed slopes (TAW 2002)
Hydraulic response	1.1.1.4	Transmission	De Jong and D'Angremond (1996)
Hydraulic response	1.1.1.5	Reflection	Postma (1989)
Structural response	1.1.2.1	Rock armour	Top layer stability (Van der Meer, 1993) Top layer stability (Hudson, SPM, 1984) Top layer stability (Van Gent et al, 2003) Top layer stability (Modified Van der Meer - Van Gent et al 2003) Cumulative damage (Van der Meer, 1993) Probability of damage to armour layer (Van der Meer, 1993)
Structural response	1.1.2.2	Concrete armour	Dolosse (Holthausen and Zwamborn, 1992) Accropodes (Van der Meer, 1999) Cubes (Van der Meer, 1999) Tetrapods (Van der Meer, 1999)

			Core-Loc (US Army Corps of Engineers, 1997)
Structural response	1.1.2.3	Toe berm	Stability Van der Meer (1998)
Structural response	1.1.2.4	Rear side	Stability Van Gent & Pozueta (2004)
Rubble mound 1.2 Berm breakwater			
Hydraulic response	1.2.1.4	Transmission	De Jong and D'Angremond (1996)
Structural response	1.2.2.1	Profile development	Stability Van der Meer (1993)
Structural response	1.2.2.2	Longshore transport	
Structural response	1.2.2.3	Rear side stability	
Rubble mound 1.4 Reef type structure			
Structural response	1.4.2.1	Rock armour	Stability, final crest elevation Van der Meer (1993)
Rubble mound 1.5 Near-bed structure			
Structural response	1.5.2.1	Rock armour	Stability Wallast and Van Gent (2002)
Vertical (caisson) structure 2.1			
Hydraulic response	2.1.1.2	Overtopping	Franco et. al. (1994)
	2.1.1.3	Transmission	Goda (1969)
	2.1.2.1	Stability	Pressures, forces and safety factors, (Goda, 1985)
Structural response	2.1.2.3	Toe berm	Stability (Goda, 1985, Tanimoto formula)

Table 1 Overview of structure types and response factors

3 Technical Manual

3.1 Introduction in Technical Manual

The Technical Manual describes the technical background of all the items included in BREAKWAT 3. With BREAKWAT 3 the structural and hydraulic response factors for a number of different structure types can be calculated. The formulae used for each item to be computed, along with a list of all input and output parameters and the limits on their ranges are all specified in the following sections. The technical contents and structure of BREAKWAT 3 is shown in Overview of structure types and response factors.

The hydraulic and structural response factors included in BREAKWAT 3 are described for different types of structures:

- Conventional rubble mound breakwaters
- Berm breakwaters
- Reef-type structures
- Near-bed structures
- Vertical (caisson) breakwater

The processes involved with stability of breakwater structures under wave (possibly combined with current) attack are given in a basic scheme in Figure 15.

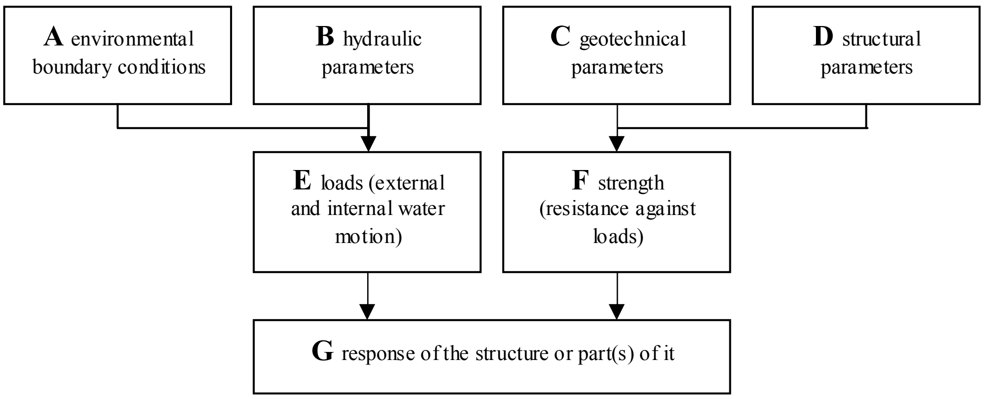


Figure 15 Processes involved in the stability of a breakwater structure

The environmental conditions (wave, current and geotechnical characteristics) lead to a number of parameters which describe the boundary conditions at or in front of the structure (A in Figure 15).

These parameters are not affected by the structure itself, and generally, the designer of a structure has no influence on them. Wave height, wave height distribution, wave breaking, wave period, spectral shape, wave angle, currents, foreshore geometry, water depth, setup and water levels are the main hydraulic environmental parameters. These environmental parameters are not described in this manual. A specific geotechnical environmental condition is an earthquake.

Governing parameters can be divided into parameters related to hydraulics (B), related to geotechnics (C) and to the structure (D). Hydraulic parameters are related to the description of the wave action on the structure (hydraulic response). The main hydraulic responses are wave runup, rundown, wave overtopping, wave transmission and reflection. Geotechnical parameters are related to, for instance, liquefaction, dynamic gradients and excessive pore pressures. They are not described in this manual.

The structure can be described by a large number of structural parameters (D) and some important ones are the slope of the structure, the mass and mass density of the rock, rock or grain shape, surface smoothness, cohesion, porosity, permeability, shear and bulk moduli and the dimensions and cross-section of the structure.

The loads on the structure or on structural elements are given by the environmental, hydraulic, geotechnical and structural parameters together (E). These loads can be divided into those due to external water motion on the slope, loads generated by internal water motion in the structure and earthquakes. The external water motion is affected amongst others things by the deformation of the wave (breaking or not breaking), the runup and run-down, transmission, overtopping and reflection. The internal water motion describes the penetration or dissipation of water into the structure, the variation of pore pressures and the variation of the phreatic line. These topics are not dealt within this manual.

Almost all structural parameters may have some or large influence on the loads. Size, shape and grading of armour rocks have influence on the roughness of the slope, and therefore on runup and rundown. Filter size and grading, together with the above mentioned characteristics of the armour rocks, have an influence on the permeability of the structure, and hence on the internal water motion.

The resistance against the loads (waves, earthquakes) can be called the strength of the structure (F). Structural parameters are essential in the formulation of the strength of the structure. Most of them also influence the loads, as described above.

Finally, the comparison of the strength with the loads leads to a description of the response of the structure or elements of the structure (G), the description of the so-called failure mechanisms. The failure mechanism may be treated in a deterministic or probabilistic way.

Hydraulic and structural responses such as stability of armour layers, filter layers, crest and rear, toe berms and stability of crest walls and dynamically stable slopes and geotechnical responses or interactions such as slip failure, settlement, liquefaction, dynamic response, internal erosion and impacts, are not described in this manual.

Figure 15 can also be used in order to describe the various ways of physical and numerical modelling of the stability of coastal and shoreline structures. A black box method is used if the environmental parameters (A) and the hydraulic (B) and structural (D) parameters are modelled physically, and the responses (G) are given in graphs or formulas. A description of water motion (E) and strength (F) is not considered.

A grey box method is used if parts of the loads (E) are described by theoretical formulations or numerical models which are related to the strength (F) of the structure by means of a failure criterion or reliability function. The theoretical derivation of a stability formula might be the simplest example of this.

Finally, a white box is used if all relevant loads and failure criterions can be described by theoretical/physical formulations or numerical models, without empirical constants. It is obvious that it will take a long time and a lengthy research effort before coastal and shoreline structures can be designed by means of a white box.

The colours black, grey and white, used for the methods described above do not suggest a preference. Each method can be useful in a design procedure.

This part of the manual will deal with physical processes and design tools, which means that design tools should be described in such a way that:

- they are easily applicable,
- the range of application should be as wide as possible, and
- research data from various investigations should, wherever possible, be combined and compared, rather than giving the data of different investigations separately.

3.2 Conventional rubble mound breakwaters

The hydraulic response factors which can be computed for conventional rubble mound breakwaters are:

- Wave height distribution for RMB
- Wave runup on RMB
- Wave overtopping of RMB
- Wave transmission over RMB
- Wave reflections of RMB

The structural response factors which can be computed for conventional rubble mound breakwaters are:

- Rock armour layers (Van der Meer (1993), Hudson (1975), Modified Van der Meer – Van Gent et al (2003), Van Gent et al (2003))
    - Cumulative damage after a sequence of storms
    - Probability of damage to a rock armour layer
    - Toe berm stability of rubble mound breakwater
    - Rear side stability
- Concrete armour units

Rules of thumb are described only in this manual for the following structural aspects for conventional rubble mound breakwaters are:

- Underlayers and filters
- Breakwater head

3.2.1 Hydraulic response factors

3.2.1.1 Wave height distribution for RMB

In deep water the wave heights are distributed according to the Rayleigh distribution, which is a specific case of a Weibull distribution. In shallower waters depth decreases which causes the higher waves in a wave field to break without disturbing the distribution of the smaller waves, therefore introducing a change in shape of the wave height distribution. This change in shape is not well described with a wave height distribution containing one shape parameter, such as the Rayleigh distribution. Therefore a composite Weibull distribution is used. It consists of two Weibull distributions, separated by a transitional wave height, H_{tr}. This module computes the wave heights in water depth h assuming a "composite Weibull" distribution.

Input

H_{m0}	Offshore spectral significant wave height	(m)
$\cot(\alpha_v)$	Cotangent of foreshore slope	(-)
h_s	Water depth at the site	(-)

Limits on input parameters

0.1	<	H_{m0} / h_s	<	0.6
20	<	$\cot(\alpha_v)$	<	250
0.01	=	H_{m0}	=	20 m
		h_s	>	0 m

(foreshore straight and depth parallel contours)

Technical background

The Composite Weibull distribution consists of two Weibull distributions, separated by a transitional wave height, H_{tr}.

$$F(H) \equiv \Pr\{\underline{H} \leq H\} = \begin{cases} F_1(H) = 1 - \exp\left[-\left(\dfrac{H}{H_1}\right)^{k_1}\right] & H \leq H_{tr} \\ F_2(H) = 1 - \exp\left[-\left(\dfrac{H}{H_2}\right)^{k_2}\right] & H > H_{tr} \end{cases}$$

Waves smaller than the transitional wave height, which are not influenced by the depth, obey the first part of the distribution $F_1(H)$. Waves exceeding the transitional wave height, which are subjected to depth induced breaking, obey the second part of the Composite Weibull distribution $F_2(H)$. The exponents k_1 and k_2 determine the shape of the distributions and two scale parameters, H_1 and H_2, are used to scale the distributions to the wave field of concern. The Composite Weibull wave height distribution is valid for $0 < H < \infty$. In order to obtain continuity the distribution must satisfy $F_1(H_{tr}) = F_2(H_{tr})$. In Figure 16the Composite Weibull distribution is presented on Rayleigh scale.

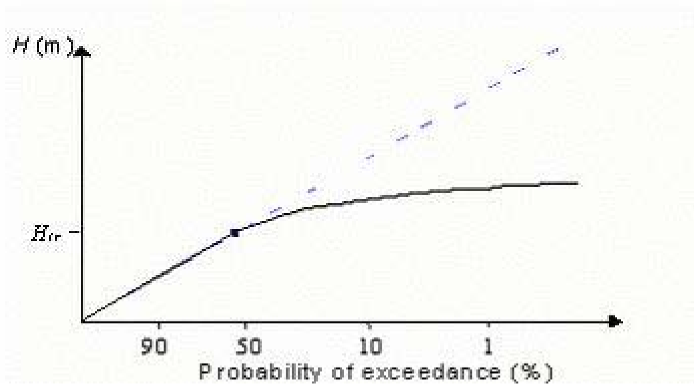


Figure 16 The Composite Weibull distribution

In the calculations all wave heights are normalised with *Hrms*:

$$\widetilde{H}_x = \frac{H_x}{H_{rms}}$$

in which H_x denotes a characteristic wave height, like H_{tr}, $H_{1/3}$ or $H_{1\%}$. Doing so, a Composite Weibull distribution, containing the parameters k_1, k_2, $\hat{H}_1$ and $\hat{H}_2$ are obtained. For further details on the determination of these parameters, the user is referred to Battjes and Groenendijk (2000) and Groenendijk and Van Gent (1999).

Output

Tabular and graphical output is given. In tabular form the following quantities are shown:

$\cot(\alpha_v)$	Cotangent of foreshore slope	(-)
H_{m0}/h	Local relative wave height	(-)
$H_{1/3}$	Significant wave height	(m)
$H_{10\%}$	Wave height exceeded by 10% of the waves	(m)
$H_{2\%}$	Wave height exceeded by 2.0% of the waves	(m)
$H_{1\%}$	Wave height exceeded by 1.0% of the waves	(m)
$H_{0.1}\%$	Wave height exceeded by 0.1% of the waves	(m)

In graphical form a comparison of these output values with the Rayleigh wave height distribution is shown by plotting $H_{p\%}$ values on the y-axis and the exceedance probability values on the x-axis. The Rayleigh wave height distribution is defined by:

$$H_{p\%} = \frac{H_s}{1.41}\sqrt{\left[-\ln\left(p\%/100\right)\right]}$$

H_s is the H_{m0} value used from above output.

For the Rayleigh distribution the x-values are plotted as a linear series defined by:

$$x = \sqrt{-\log_{10}\left(\frac{p\%}{100}\right)}$$

3.2.1.2 Wave overtopping of RMB

To predict the amount of wave overtopping of coastal structures two design methods have been implemented in BREAKWAT:

- TAW (2002), and
- Neural Network.

TAW (2002)

This formula is presently accepted as the official formula for computing the wave overtopping of dike slopes in The Netherlands (TAW, 2002a). As breakwater slopes are generally rougher and more permeable than dikes, the suggested values especially for the influence of roughness factor for roughness may not be applicable to breakwaters. For such an application the roughness factor γ_f has to be set to a low value (0.5). However, the formula has not been validated for values in this range.

The official formula has been somewhat simplified for the use in BREAKWAT. The structure slope may include one single, horizontal berm of width B with a lower slope (α_{s1}) and upper slope (α_{s2}).

Technical background

If extreme runup levels exceed the crest level the structure will be overtopped. This may occur for relatively few waves under the design event, and a low overtopping rate may often be accepted without severe consequences for the structure or the area protected by it. Sea walls and breakwaters are often designed on the basis that some (small) overtopping discharge is to be expected under extreme wave conditions. The main design problem therefore reduces to dimensioning the cross-section geometry such that the mean overtopping discharge under design conditions remains, q, below acceptable limits.

The most simple dimensionless parameter, Q, for the mean overtopping discharge, q, can be defined by:

$$Q = \frac{q}{\sqrt{gH_s^3}}$$

Sometimes the wave steepness and the slope angle have also influence on the overtopping and in that case the definition of dimensionless overtopping discharge may be extended by including s_{0m} or s_{0p} and/or $\cot(\alpha)^2$. Various definitions can be found in Owen (1980), Bradbury *et al.* (1988) and De Waal and Van der Meer (1992).

Input

H_{m0}	Spectral significant wave height at the toe of the structure	(m)
T_m	Mean wave period	(s)
$T_{m-1,0}$	Spectral wave period ($T_p = 1.1 T_{m-1,0}$)	(s)
$\cot(\alpha_{s1})$	Lower slope angle of the structure	(-)
$\cot(\alpha_{s2})$	Upper slope angle of the structure	(-)
B	Berm width	(m)
d_h	Depth of berm below SWL (negative if above SWL)	(m)
N	Number of incoming waves	(-)
R_c	Crest freeboard above still water level	(m)
γ_f	Reduction factor for roughness	(-)
γ_V	Reduction factor for a vertical wall on top of the slope	(-)
β	Angle of wave attack with respect to structure	(deg)

Formula

For deterministic design purposes, the following formula is applied for the dimensionless mean overtopping discharge:

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{\tan\left(\alpha_{rep}\right)}}\gamma_b \xi_0 \exp\left(-4.3\frac{R_c}{H_{m0}}\frac{1}{\xi_0 \gamma_b \gamma_f \gamma_\beta \gamma_V}\right)$$

with a maximum value of:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2\exp\left(-2.3\frac{R_c}{H_{m0}}\frac{1}{\gamma_f \gamma_\beta}\right)$$

in which the relative crest freeboard parameters can be computed as:

$$R = \frac{R_c}{H_{m0}}$$

$$R_n = \frac{R_c}{H_{m0}}\frac{1}{\gamma_f \gamma_\beta}$$

and the dimensionless overtopping rate as:

$$Q = \frac{q}{\sqrt{gH_{m0}^3}}$$

The values of γ_b, γ_f, and γ_V are identical to those computed with the TAW runup formula for wave runup on a dike slope (Runup on bermed slopes - TAW formula), except for the factor γ_β . In the formulae to determine the influence of the angle of wave attack in the formula for wave runup, the factor 0.0022 has to be replaced for 0.0033 in order to calculate the influence of the angle of wave attack on wave overtopping. The breaker parameter is also calculated in the same way as for the TAW formula for wave runup:

$$\xi_0 = \frac{\tan \alpha_{rep}}{\sqrt{s_0}}$$

and

$$s_0 = \frac{2\pi H_{m0}}{g T_{m-1,0}^2}$$

where

q	Mean overtopping rate per m running length of dike	(l/s/m)
g	Acceleration due to gravity	(m/s^2)
cot(α_{rep})	Representative slope angle of the structure	(-)
γ_f, γ_V, γ_b, γ_β	Reduction factors for roughness, vertical wall on top of the slope, berm and angle of wave attack. The minimum value for a combination of influence factors is 0.4	(-)

The slope of the dike may include a horizontal berm of width B and/or a lower slope (α_{s1}) and upper slope (α_{s1}).

Overtopping percentages and volumes per wave

The percentage of overtopping waves is calculated as:

$$P_{ov} = \exp\left[-\left(\sqrt{-\ln 0.02} \frac{R_c}{R_{u2\%}} \right)^2 \right]$$

The volume of overtopping water in one given wave (V_{OL} in l/m) for a given exceedance probability is defined as:

$$V_{OL} = 1000a\left[-\ln\left(1-p_v\right)\right]^{4/3}$$

with

$$a = \frac{0.84 T_m q / 1000}{P_{ov}}$$

and p_v is the exceedance probability.

V_{OL} should be calculated (and plotted) for values of p_v of 100%, 50%, 10%, 5%, 2%, 1%, 0.5% and 0.1%.

The maximum volume in one wave (in l/m) is computed as:

$$V_{MAX} = 1000a\left[\ln\left(NP_{ov}\right)\right]^{4/3}$$

Limits

0 m	<	H_{m0}	<	20 m
0 s	<	$T_{m-1,0}$	<	20 s
0 s	<	T_m	<	20 s
0	<	s_0	<	0.07
0	<	ξ_0	<	7.0
1.0	<	$\cot(\alpha_{s1})$	<	7.0
1.0	<	$\cot(\alpha_{s2})$	<	7.0
0.01 m	≤	B	<	$0.25(gT^2_{m-1,0})/(2\pi)$
0.5	<	$\gamma_b\ \xi_0$	<	10
		$\gamma_f, \gamma_V, \gamma_b, \gamma_\beta$	≥	0.4
		β	<	90 deg

Overtopping prediction by Neural Network

The prediction tool for the estimation of mean overtopping discharges at various types of coastal structures is developed by WL | Delft Hydraulics. Details of the methodology followed for the development of the prediction tool are described in Pozueta *et al.* (2004) and Van Gent *et al.* (2004). The output of this program tool includes the Neural Network prediction of the mean overtopping discharge at a coastal structure and several other parameters indicating the uncertainty of the prediction.

The study on which this tool is based was co-sponsored by the Commission of the European Communities within the framework of the CLASH project ('Crest Level Assessment of Coastal Structures by full scale monitoring, neural network prediction and Hazard analysis on permissible wave overtopping').

The predictions based on the Neural Network can be used for the conceptual design of coastal structures; they may not be used in the final design stage, since the results should be verified based on dedicated physical model tests for the particular wave conditions and structure geometry of the structure to be built. The predictions are based on a data-set based on small-scale physical model tests; the predictions are to some extent affected by model effects, scale effects (see deliverable D40 from the CLASH project), limited accuracy of measurement equipment, limited accuracy of wave generation techniques (compared to nowadays state-of-the-art techniques), inconsistencies in the data-set, and lack of data in certain fields of application. Although reliability levels are given in addition to the predictions, these reliability levels do not account for most of these influences. Therefore, the Neural Network predictions may only be used as first estimates of mean overtopping discharges.

Background

For the prediction of the mean overtopping discharge a Neural Network model is used. This model was derived by WL | Delft Hydraulics from 8372 input-output combinations obtained from measurements performed in hydraulic scale models at several institutes (Aalborg University, Denmark; Danish Hydraulic Institute, Denmark; WL | Delft Hydraulics, The Netherlands; Hydraulic Research Wallingford, UK; Leichtweiss Institute für Wasserbau, WKS+GWK, Germany; Modimar, Italy; University of Edinburgh, United Kingdom; Universidad Politécnica de Valencia, Spain; and others in Iceland, Japan, Norway and U.S.A).

Input

The input consists of the following 15 parameters. A more detailed description of these parameters is given hereafter.

$H_{m0,toe}$	Spectral significant wave height at the toe of the structure obtained from spectral analysis, $H_{m0,toe} = 4m_0^{1/2}$	(m)
$T_{m-1,0\text{-}toe}$	Mean wave period at the toe of the structure obtained from spectral analysis, $T_{m-1,0\ toe} = m_{-1}/m_0$	(s)
β	Direction of wave attack w.r.t. to the normal of the structure	(°)
h	Water depth in front of the structure	(m)
h_t	Water depth on the toe of the structure	(m)
B_t	Width of the toe of the structure	(m)
γ_f	Roughness/permeability of the structure	(-)
$\cot(\alpha_d)$	Slope of the structure downward of the berm	(-)
$\cot(\alpha_u)$	Slope of the structure upward of the berm	(-)
B	Berm width	(m)
h_b	Water depth on the berm	(m)
$tan\ \alpha_B$	Slope of the berm	(-)
R_c	Crest freeboard of the structure	(m)
A_c	Armour crest freeboard of the structure	(m)
G_c	Crest width of the structure	(m)

Figure 16a shows a graphical illustration of the meaning of the fifteen input parameters.

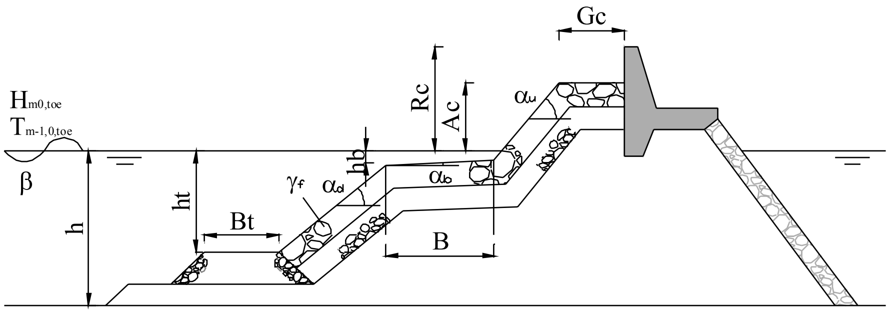


Figure 16a Parameters used for the NN modelling of wave overtopping discharge at coastal structures

For each user-supplied set of input parameters [H_{m0}, $T_{m-1,0}$, β, h, h_t, B_t, γ_f, $cot\ \alpha_d$, $cot\ \alpha_u$, B, h_b, $tan\ \alpha_b$, R_c, A_c, G_c], the output includes the mean wave overtopping discharge (q), and 7 other output values indicating the quantiles of several orders, $q_{2,5\%}$, $q_{5\%}$, $q_{25\%}$, $q_{50\%}$, $q_{75\%}$, $q_{95\%}$ and $q_{97,5\%}$.. The 95% confidence interval is, for instance, given by the quantiles $q_{2.5\%}$ and $q_{97.5\%}$.

Determination of input parameters

This section briefly describes how to estimate some of the hydraulic parameters required by the program when these are not available, and how to determine the required structure parameters for any arbitrary structure and for rather complicated coastal structures. A more extensive description can be found in Van der Meer *et al.* (2004).

Often some of the **hydraulic parameters** might not be directly available. In these cases, the following estimations or calculations can be applied:

- If only the deep water wave characteristics are available and not the wave characteristics at the toe of the structure, the calculation of the wave characteristics at the toe of the structure can be performed with the model 'SWAN'.
- If only the wave height from time domain analysis, H_s, is available, the wave height from spectral domain analysis, H_{m0}, can be determined with Battjes and Groenendijk (2000).
- If only the wave period from time domain analysis is available, the estimation of the wave period from spectral analysis can be made by means of empirically determined proportions of wave periods available in literature, such as the following (Goda and Nagai, 1974; Goda, 1985), which can be used for single-peaked wave energy spectra with a spectral shape similar to Jonswap spectra.

$$T_p \approx 1.05\ T_{1/3}$$
$$T_p = 1.2\ T_m$$
$$T_{m-1,0} \approx T_p / 1.1$$

Most coastal structures can be relatively well schematised by means of the 12 **structure parameters**. For the correct use of this prediction tool it is important that all parameters are determined in the same way. In the following, a brief description indicating how to determine these 12 structure parameters for an arbitrary structure is given.

Three parts can be distinguished in an average coastal structure, the lower part (or **toe**), the centre part (eventually with a **berm),** and the upper part (or **crest)**. The separation of these three parts of the structure is not always that clear and depends on the hydraulic conditions and structure shape. In this way, the same structure could have a different schematisation for a different water level and different wave attack. Figure 16b shows the three parts of a typical coastal structure, where the Centre part corresponds to the area within the vertical distance 1.5*$H_{m0,toe}$ above and below the sea water level, and the Upper and Lower parts correspond to the areas above and below the Centre part, respectively.

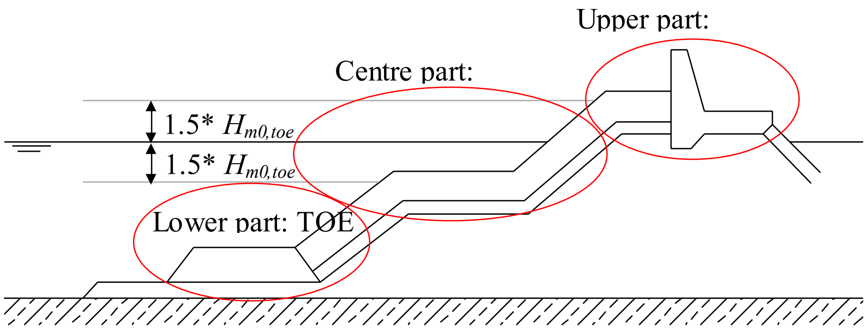


Figure 16b Parts of a coastal structure

The toe of the structure is normally situated in the lower area of the structure. Nevertheless, in some cases the toe can also be located at the centre part of the structure (e.g. structure with quite large toe situated in relatively shallow water). In this case, the toe can be taken into account as a berm, as shown in Figure 16c.

A berm is always located in the centre area of the structure. If the 'berm' is situated lower, then it is considered as a toe (as shown in Figure 16c), while if it is situated higher, then it is considered as a crest. Some further restrictions exist regarding the definition of a berm. These are described in TAW (2002a).

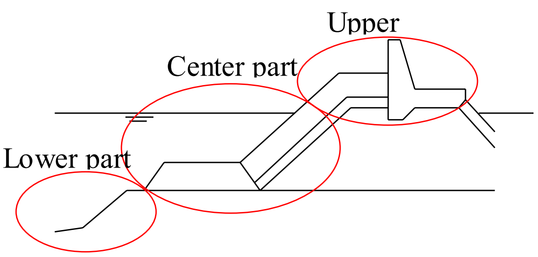


Figure 16c Toe schematised as a berm

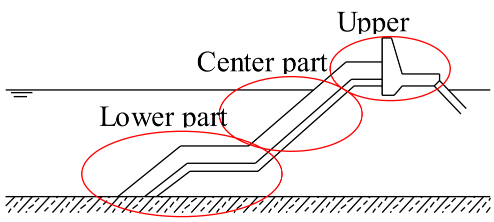


Figure 16d Berm schematised as a toe

A crest of a structure is situated normally in the upper area of a structure. However, there are also exceptions in this case (e.g. in very low structures the crest can be a part of the centre area).

In the following, a detailed explanation regarding the determination of each of the 12 **structure parameters** is given.

- h [m]: This is the water depth at the toe of the structure, more precisely the water depth just before the structure.

- h_t [m], B_t [m]: These are the water depth on the toe and the width of the toe. The width of the toe is measured on top of the toe. If there is no toe, the value of the water depth on the toe is the same as the water depth in front of the structure $h_t = h$. In this case the width of the berm B_t is equal to zero.

- B [m], h_b [m], $tan\ \alpha_B$ [-]: These parameters describe the berm of the structure. B is the berm width, measured horizontally. h_b is the water depth on the berm, measured in the middle of the berm. If the berm is situated above swl, h_b is negative. $Tan\ \alpha_B$ is the tangent of the slope of the berm. If the berm is horizontally, $tan\ \alpha_B = 0$.

- R_c [m], A_c [m], G_c [m]: These parameters describe the upper part of the structure. R_c is the crest freeboard of the structure; that is, the distance, measured vertically from the still water level (swl) to the highest impermeable point of the structure. This means that at this point, waves are stopped by the structure. A_c is the armour crest freeboard of the structure; that is, the distance, measured vertically from the swl to the highest point of the armour on the structure. G_c is the crest width. In the case that a crest is constructed on the structure, G_c is the width of the armour in front of the crest element.

- $cot\ \alpha_d$ [-], $cot\ \alpha_u$ [-]: These parameters are used to describe the slope(s) of the structure. The toe and the crest of the structure are already described in other parameters, therefore they are not included in these two parameters. $cot\ \alpha_d$ and $cot\ \alpha_u$ are the cotangents of the mean slopes in the centre area of the structure under (cot $\alpha_{\mathbf{down}}$) and above (cot $\alpha_{\mathbf{up}}$) the berm respectively. The upper slope α_u can be determined by taking the point of the structure at a level of $1.5*H_{m0\ toe}$ above the swl and connecting it with the point of the berm farthest from the sea. If the crest of the structure is situated in the centre area of the structure (at a distance less than $1.5*H_{m0\ toe}$ above swl), then the point of the crest nearest to the sea has to be used to determine α_u. The lower slope α_d can be determined by taking the point of the structure at a level of $1.5*H_{m0\ toe}$ under the swl and connecting it with the point of the berm nearest to the sea. If the toe of the structure is situated in the centre area of the structure (at a distance less than $1.5*H_{m0\ to}$ under swl), then the point of the toe farthest of the sea has to be used to determine α_d.

- γ_f [-]: This parameter gives an indication of the roughness and the permeability of the structure. The rougher and more permeable a structure is, the lower the overtopping will be since more energy is dissipated on a rough surface and more energy will disappear into a permeable structure. The factor γ_f was originally introduced as a reduction factor for the roughness and permeability for run-up on a structure. Values of γ_f for run-up on dikes with different top-layers are given in TAW (2002a). Overtopping structures are often constructed of rubble mound, artificial armour units or a combination of both. As no extensive research has been performed before regarding the roughness of different armour units, some assumptions can be made. Table 1a gives some suggestions for the roughness-factors of the most common armour layers.

Type of armour layer	γ_f	**Type of armour layer**	γ_f
smooth impermeable surface	1	accropods (2 layers)	0.4
rock (single layer)	0.7	core-locs (1 layer)	0.4
rock (double layer)	0.55	sheds (1 layer)	0.5
basalt revetment	0.9	seabeas (1 layer)	0.7
cubes (2 layers, random)	0.4	berm breakwater	0.5
tetrapods (2 layers)	0.4	Icelandic berm breakwater	0.4
dolosse (2 layers)	0.4		

Table 1a Roughness/permeability of common armour layers

Schematisation of complex structures

In some cases, it is not possible to describe very complicated coastal structures with the above-mentioned parameters. For these cases, it is then required to make rough schematisations to come to an approximate description of the section with these parameters. Often, several possibilities exist to make such approximations; therefore it is the task of the user to choose the best solution for the schematisation of the structure. In the following, some examples of the schematisation of several structures are given.

Example 1: Structure with several slopes

In the case that a structure consists of more than two slopes (e.g. lower, mid and upper slopes), it should be taken into account that a mid-slope of 1:5 is too steep to be a berm. This slope should be therefore included in the slope of the structure downward of the berm (parameter $cot\ \alpha_d$). A possible schematisation could be to consider the lower slope as the toe of the structure with toe width $B_t = 0$, the mid-slope as the slope downward the berm ($cot\ \alpha_d$), the upper slope as the slope upward the berm ($cot\ \alpha_u$) and the intersection between the last slopes as a berm with berm width $B = 0$.

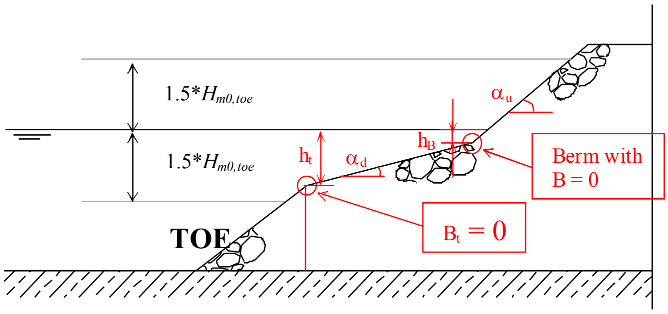


Figure 16e Example 1: Structure with several slopes

Example 2: Structure with more than one berm

In the case that a structure consists of several sloping and horizontal parts (e.g. two horizontal parts or horizontal berms with not a too big difference in level between both berms), a rougher schematisation is required. The schematisation advised in this case is to combine the two horizontal berms in one mean berm at the average level of both berms. The berm width can be determined here by lengthen the upper and lower slope up to the level of the mean berm.

If the width of the two berms is very different, a weighted average level of the mean berm is preferable (so the berm with the largest width has most influence on the level of the mean berm).

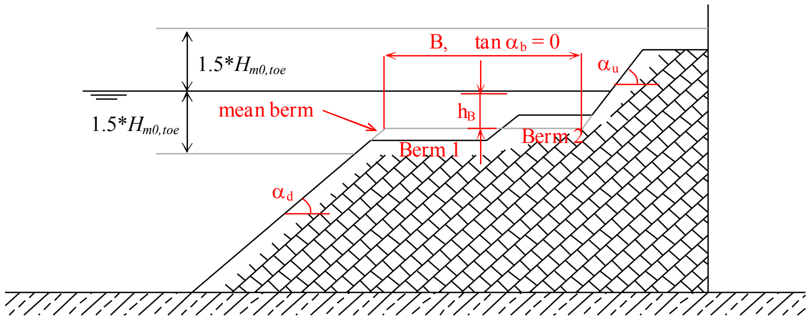


Figure 16f Example 2: Structure with more than one berm

Example 3: Sloping crest

The schematisation advised in the case that the crest of the structure is a sloping crest, is to consider the crest as a horizontal crest at a level corresponding to the middle point of the crest. The value of the crest width, G_c, in this case results from extending the slope of the structure up to the level of the horizontal crest.

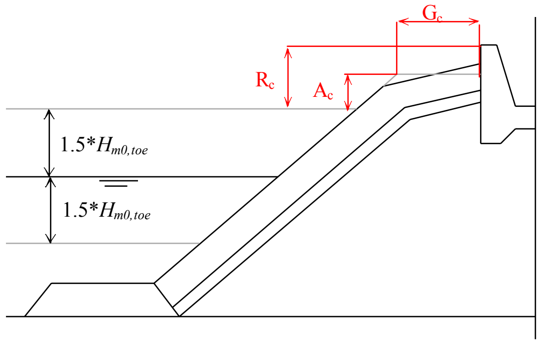


Figure 16g Example 3: Sloping crest

Output

As described in previous sections, the main part of the output is the mean overtopping discharge (q) predicted by the NN model in l/s/m. Next to that, the output consists of the quantiles $q_{2,5\%}$, $q_{5\%}$, $q_{25\%}$, $q_{50\%}$, $q_{75\%}$, $q_{95\%}$ and $q_{97,5\%}$.

Apart from numerical output, also remarks and warnings are generated for specific cases. If that is the case, they can be found displayed in red in the output window just below the output parameters.

– 'WARNING> Parameter <parameter name> out of range of validity. No prediction is given'

  This warning indicates that because the parameter <parameter name> is out of the range of validity of the NN model, no prediction is given. This warning can appear for any of the input parameters. For the mean overtopping discharge (q) the value -1000 is given.

– 'WARNING> For $10^{-6} < Q < 10^{-5}$, the NN prediction is less reliable (indicative)'

  This remark indicates that the dimensionless value of the mean overtopping discharge predicted by the NN lies in the region $10^{-6} < Q < 10^{-5}$, (with Q = dimensionless overtopping discharge, $Q = q/\sqrt{gH_s^3}$), and therefore, the NN prediction should be considered as less reliable, or indicative.

– 'WARNING> For $Q < 10^{-6}$, no prediction is given'

  This remark indicates that the dimensionless value of the mean overtopping discharge predicted by the NN is in the region $Q < 10^{-6}$ (with Q = dimensionless overtopping discharge, $Q = q/\sqrt{gH_s^3}$), and therefore, the NN prediction does not give any prediction. The default value given in this case is -1000.

– 'REMARK > For prototype, rough-sloping structures, a correction factor is applied: q = '

With this remark the user is given the possibility of choosing between the direct output of the NN model or a corrected value to account for model effects, scale effects and wind effects in prototype situations (H_{m0} > 0.5 m), for rough-sloping structures (γ_f < 0.9 and cot α > 1).

– 'REMARK > For prototype, rough-sloping structures, a correction factor is applied: q = , Since 10^{-6} < Q < 10^{-5}, the NN prediction is less reliable (indicative)'

With this remark the user is given the possibility of choosing between the direct output of the NN model or a corrected value to account for model effects, scale effects and wind effects in prototype situations (H_{m0} > 0.5 m), for rough-sloping structures (γ_f < 0.9 and cot α > 1). The user is also warned that the dimensionless value of the mean overtopping discharge predicted by the NN before the correction is applied lies in the region 10^{-6} < Q < 10^{-5}, and therefore the NN prediction should be considered as less reliable, or indicative.

– 'REMARK > For prototype, smooth vertical structures, a correction factor is applied: q = '

With this remark the user is given the possibility of choosing between the direct output of the NN model or a corrected value to account for wind effects in prototype situations (H_{m0} > 0.5 m), for smooth ($\gamma_f \geq 0.9$) and vertical structures (cot $\alpha \leq 1$).

– 'REMARK > For prototype, smooth vertical structures, a correction factor is applied: q = , Since 10^{-6} < Q < 10^{-5}, the NN prediction is less reliable (indicative)'

With this remark the user is given the possibility of choosing between the direct output of the NN model or a corrected value to account for wind effects in prototype situations (H_{m0} > 0.5 m), for smooth ($\gamma_f \geq 0.9$) and vertical structures (cot $\alpha \leq 1$). The user is also warned that the dimensionless value of the mean overtopping discharge predicted by the NN before the correction is applied lies in the region 10^{-6} < Q < 10^{-5}, and therefore the NN prediction should be considered as less reliable, or indicative.

Limits

The range of validity of the possible input parameters is given below referring to a significant wave height of H_{m0}=1 m.

1 m	≤	H_{m0}	≤	1 m
0.005	≤	$s_{m-1,0}$	≤	0.07
0	≤	β	≤	80
0.9 m	≤	h	≤	20 m
0.5 m	≤	h_t	≤	20 m
0	≤	B_t	≤	10 m
0.3	<	γ_f	≤	1
0	≤	$cot\ \alpha_d$	≤	10
-1	≤	$cot\ \alpha_u$	≤	10
0	≤	B	≤	15 m
-1 m	≤	h_b	≤	5 m
0	≤	$tan\ \alpha_B$	≤	0.1
0.5 m	≤	R_c	≤	5 m
0	≤	A_c	≤	5 m
0	≤	G_c	≤	10 m

3.2.1.3 Wave transmission over RMB

Breakwaters with relatively low crest levels may be overtopped with sufficient severity to excite wave action behind. When a breakwater has been constructed with relatively permeable materials, long wave periods may lead to transmission of wave energy through the structure. In some cases the two different responses will be combined.

The quantification of wave transmission is important in the design of low-crested breakwaters intended to protect beaches or shorelines. It is also important in the design of harbour breakwaters where long wave periods transmitted through the breakwater could cause movement of ships or other floating bodies.

The severity of wave transmission is described by the coefficient of transmission, C_t, defined in terms of the incident and transmitted wave heights, H_i and H_t respectively.

The transmission is computed according to the formula by De Jong and d'Angremond (1996). This formula includes the effect of structure slope on the transmission characteristics by means of the surf similarity parameter. The parameter A_{str} is used in the formula to account for the effect of the structure type.

Input

H_s	Incident significant wave height	(m)
T_p	Peak wave period	(s)
B	Crest width	(m)
R_c	Crest freeboard above still water level	(m)
$\cot(\alpha_s)$	Slope angle of the structure	(-)
A_{str}	A coefficient depending on the structure type 0.64 for rock slopes and concrete units 0.80 for smooth impermeable dam 0.80 for impermeable smooth block revetment 0.75 for block mattresses 0.70 for gabion mattresses	(-)

Technical background

$$C_t = a - \left[0.4\left(R_c / H_s\right)\right]$$

where

$$a = \left(B / H_s\right)^{-0.31}\left[1 - \exp\left(-0.5\xi_{0p}\right)\right]A_{str}$$

$$\xi_{0p} = 1.0 / \left[\left(\cot\alpha_s\right)\sqrt{2\pi H_s / \left(gT_p^2\right)}\right]$$

Limits

0.075	$\leq$	C_t	$\leq$	0.8
0.01 m	<	H_s	<	20 m
0.5 s	<	T_p	<	30 s
1.1	<	$\cot(\alpha_s)$	<	7.0
0.5	<	ξ_{0p}	<	10.0
0.005	$\leq$	$2\pi H_s / gT_p^2$	$\leq$	0.07

3.2.1.4 Wave reflections of RMB

Wave reflections are of importance on the open coast, and at commercial and small boat harbours. The interaction of incident and reflected waves often lead to a confused sea in front of the structure, with occasional steep and unstable waves of considerable hazard to small boats. Reflected waves can also propagate into areas of a harbour previously sheltered from wave action. They will lead to increased peak orbital velocities, increasing the likelihood of movement of beach material. Under oblique waves, reflection will increase littoral currents and hence local sediment transport.

All coastal structures reflect some proportion of the incident wave energy. This is often described by a reflection coefficient, C_r, defined in terms of the incident and reflected wave heights, H_i and H_r respectively.

The amount of wave reflection is computed using the formula of Postma (1989).

Input

P	Van der Meer's permeability coefficient	(-)
H_s	Incident significant wave height	(m)
T_p	Peak wave period	(s)
$\cot(\alpha_s)$	Slope angle of the structure	(-)

Formula

$$C_r = 0.071 P^{-0.082} \cot \alpha_s^{-0.62} s_{0p}^{-0.46}$$

where

C_r	Coefficient of reflection	(-)
s_{0p}	Wave steepness: $2\pi H_s/gT_p^2$	(-)

Limits

0	$\leq$	C_r	$\leq$	1.0
0.01 m	<	H_s	<	20 m
0.5 s	<	T_p	<	30 s
1.1	<	$\cot(\alpha_s)$	<	7.0
0.5	<	ξ_{0p}	<	10.0
0.005	$\leq$	$2\pi H_s/gT^2_p$	$\leq$	0.07

3.2.1.5 Wave runup on RMB

Wave action on a rubble mound structure will cause the water surface to oscillate over a vertical range generally greater than the incident wave height. The extreme levels reached in each wave, termed runup and rundown, R_u and R_d respectively, and defined relative to the still water level (SWL), constitute important design parameters. The design runup level can be used to determine the level of the structure crest, the upper limit of protection or other structural elements, or as an indicator of possible overtopping or wave transmission. Runup is often given in a dimensionless form R_{ux}/H_s, where the subscript x describes the level considered (for instance 2%) or significant (s).

Runup levels on smooth, impermeable dike slopes as well as rubble slopes armoured with rock armour or rip- rap have been measured in laboratory tests. In the case of rubble mound structures, the rubble core has, been frequently reproduced as fairly permeable, except for those particular cases where an impermeable core has been used therefor test results often span a range within which the designer must interpolate. Analysis of test data from measurements by Van der Meer and Stam (1992) has given prediction formulas for rock slopes with an impermeable core, described by a notional permeability factor P = 0.1, and porous mounds of relatively high permeability given by P = 0.4 - 0.6. The notional permeability factor P is described in Stability formulae rock - Van der Meer (1993).

Three different types of formulae can be compared:

- Runup distribution on plane rock slopes - Weibull
- Runup on bermed slopes - TAW formula
- Runup on bermed slopes - Van Gent formula

Runup distribution on plane rock slopes - Weibull

Various applied fundamental research studies using physical scale models have been directed towards the analysis of wave run-up and overtopping on various structures. In some of these studies the run- up has been extensively measured on rock slopes (De Waal and Van der Meer, 1992). The influence of berms, roughness on the slope (also one layer of rock) and shallow water on run-up and overtopping, has been measured for smooth slopes. Analysis of test data from measurements by Van der Meer and Stam (1992) has given prediction formulas for rock slopes with an impermeable core, described by a notional permeability factor P = 0.1, and porous mounds of relatively high permeability given by P = 0.4 - 0.6. The notional permeability factor P is described in Stability formulae rock - Van der Meer (1993). For those measurements the runup has been described as a Weibull distribution with which the runup level associated with a given exceedance probability can be evaluated.

Input

The formula makes use of the following parameters:

H_s	Significant wave height at the toe of the structure	(m)
T_m	Mean wave period	(s)
P	Notional permeability factor defined by Van der Meer	(-)
$\cot(\alpha_s)$	Slope angle of the structure	(-)
Prob	Exceedence probability value of the runup level	(%)

Technical background

$$R_{up} = b\left(-\ln \Pr ob\right)^{1/c}$$

in which R_{up} (m) is the runup level above the still water level exceeded by '(Prob)' percent of the incoming waves, b (-) a scale parameter and c (-) a shape parameter.

The parameters b and c are calculated as follows:

$$b = 0.4 H_s \cot\left(\alpha_s\right)^{-0.2} s_{om}^{-0.25}$$

$$s_{om} = \frac{2\pi H_s}{g T_m^2}$$

$$\begin{cases} c = 3.0\xi_{om}^{-0.75} & \xi_{om} \le \xi_c \\ c = 0.52 P^{-0.3} \xi_{om}^{P} \sqrt{\cot \alpha_s} & \xi_{om} > \xi_c \end{cases}$$

$$\xi_{om} = 1/\left[\left(\cot \alpha_s\right)\sqrt{2\pi H_s / g T_m^2}\right]$$

$$\xi_{mc} = \left[5.77 P^{0.3} \sqrt{10/\cot \alpha_s}\right]^{1/(P+0.75)}$$

Confidence bands

An indication of the reliability of the formula can be obtained by plotting the 90% confidence bands. The confidence bands are computed by adjusting the factor b in the above equations.

Limits

0.01 %	<	*Prob*	<	99.99 %
2.0	<	cot(αs)	<	7.0
0.1	=	P	=	0.6
0.01 m	=	H_s	=	20 m
0.5 s	=	T_m	=	30 s
0.005	=	s_{om}	=	0.07
0.5	=	ξ_{0m}	=	10

Runup on bermed slopes - TAW formula

This formula is presently accepted as the official formula for computing the 2% wave runup level on dike slopes in the Netherlands (TAW, 2002a). As breakwater slopes are generally rougher and more permeable than dikes, the suggested values especially for the influence of roughness factor may not be applicable to breakwaters. For such an application the roughness factor γ_f has to be set to a low value (0.5). However, the formula has not been validated for values in this range. If this formula is used for breakwaters, it should be compared to results from the Weibull distribution.

The official formula has been somewhat simplified for the use in BREAKWAT 3. The structure slope may include one single, horizontal berm of width B with a lower slope (α_{s1}) and upper slope (α_{s2}).

The formula requires an iterative solution for $R_{u2\%}$,. The procedure is described after the general formula is given.

Input

The formula makes use of the following parameters:

H_{m0}	Spectral significant wave height at the toe of the structure	(m)
$T_{m-1,0}$	Spectral wave period ($T_p = 1.1T_{m-1,0}$)	(s)
$\cot(\alpha_{s1})$	Lower slope angle of the structure	(-)
$\cot(\alpha_{s2})$	Upper slope angle of the structure	(-)
B	Berm width	(m)
d_h	Depth of berm below SWL (negative if above SWL)	(m)
γ_f	Reduction factor for roughness	(-)
β	Angle of wave attack with respect to structure	(deg)

Technical background

$$\frac{R_{u2\%}}{H_{m0}} = 1.75\gamma_b\gamma_f\gamma_\beta\xi_0$$

with a maximum of:

$$\frac{R_{u2\%}}{H_{m0}} = \gamma_f\gamma_\beta\left(4.3 - \frac{1.6}{\sqrt{\xi 0}}\right)$$

$$\xi_0 = \frac{\tan\alpha_{rep}}{\sqrt{s_0}}$$

$$s_0 = \frac{2\pi H_{m0}}{g T_{m-1,0}^2}$$

where ξ_0 (-) is the breaker parameter using representative slope and period ($T_{m-1,0}$), γ_f (-) is the reduction factor for roughness on the slope, γ_b (-) is the reduction factor for a bermed slope and γ_β (-) is the reduction factor for oblique wave attack.

These parameters are defined as:

Reduction factor for a bermed slope γ_b

$$\gamma_b = 1 - r_B\left(1 - r_{dh}\right)$$
$$0.6 \le \gamma_b \le 1.0$$
$$r_B = \frac{B}{L_{berm}}$$

where B is the berm width and L_{berm} is the horizontal distance between a point on the dike face 1.0 H_{m0} below the mid point of the berm and a point on the dike face 1.0 H_{m0} above this mid point.

$$L_{berm} = \frac{H_{m0}}{\tan\left(\alpha_1\right)} + \frac{H_{m0}}{\tan\left(\alpha_2\right)} + B$$

$$r_{dh} = 0.5 - 0.5\cos\left(\pi \frac{d_h}{x}\right)$$

The parameter d_h is positive if the berm is below the still water level (SWL) and negative if it is above SWL. The parameter x is defined as:

$$\begin{cases} \dfrac{-R_{u2\%}}{H_{m0}} \le \dfrac{d_h}{H_{m0}} < 0 & x = R_{u2\%} \\ 0 \le \dfrac{d_h}{H_{m0}} \le 2 & x = 2H_{m0} \\ \dfrac{-d_h}{H_{m0}} \ge \dfrac{R_{u2\%}}{H_{m0}} \quad or \quad \dfrac{d_h}{H_{m0}} > 2 & r_{dh} = 1 \end{cases}$$

The above equation has to be solved iteratively as $R_{u2\%}$ is not known beforehand.

Computation of the representative slope α_{rep}

If the structure has a different lower and upper slope a representative slope must be computed. The definition of the representative slope is given by taking the average slope between a point 1.5 H_{m0} under the still water level and the 2% runup level $R_{u2\%}$ above the still water level (this horizontal distance is defined as L_{slope}) and ***excluding*** the berm width B. Therefore, the average slope to be used requires an iterative solution as $R_{u2\%}$ is not known beforehand.

$$d_h > 1.5H_{m0} \qquad \tan\left(\alpha_{rep}\right) = \tan\left(\alpha_{s2}\right)$$

$$d_h < -R_{u2\%} \qquad \tan\left(\alpha_{rep}\right) = \tan\left(\alpha_{s1}\right)$$

$$\begin{cases} -R_{u2\%} \le d_h \le 1.5H_{m0} \\ \tan\left(\alpha_{rep}\right) = \dfrac{1.5H_{m0} + R_{u2\%}}{\left(L_{slope} - B\right)} \\ L_{slope} = L_1 + L_2 + B \\ L_1 = MAX\left\{\dfrac{1.5H_{m0} - d_h}{\tan\left(\alpha_{s1}\right)};0\right\} \\ L_2 = MAX\left\{\dfrac{R_{u2\%} - d_h}{\tan\left(\alpha_{s2}\right)};0\right\} \end{cases}$$

Reduction factor for roughness on the slope γ_f (input parameter)

For dikes the following values of γ_f are recommended:

γ_f = 1.0, for smooth impermeable slopes,
γ_f = 1.0 for grass;
γ_f = 0.7 for rock slopes with one layer, and
γ_f = 0.55 for rock slopes with 2 or more layers.

The roughness of a slope has less effect on the runup in case of longer waves. Therefore, the roughness reduction factor increases linearly up to a value of 1.0 for $1.8 < \xi_0 < 10$ dependent on ξ_0. If $\xi_0 < 1.8$, the roughness reduction factor is not adjusted.

Reduction factor for oblique wave attack γ_β

The reduction factor for oblique wave attack is:

$$\begin{cases} \gamma_\beta = 1 - \left(0.0022 \times |\beta|\right) & for\ 0^o \le |\beta| \le 80^o \\ \gamma_\beta = 1 - \left(0.0022 \times 80\right) & for\ 80^o \le |\beta| \le 90^o \end{cases}$$

where the angle β is defined as the angle between the wave direction and the line normal to the dike (β= 0 means perpendicular wave attack).

Limits

0.001	<	s_0	<	0.10
0.5	<	ξ_0	<	10
1.0	<	$\cot(\alpha_{s1})$	<	8.0
1.0	<	$\cot(\alpha_{s2})$	<	8.0
0 m	≤	B	<	$0.25(gT^2_{m-1,0})/(2\pi)$
0.5	<	$\gamma_b\ \xi_0$	<	10
		$\gamma_b, \gamma_f, \gamma_\beta$	≥	0.4
		γ_f, γ_β	≥	0.4
		β	<	90 deg

Runup on bermed slopes - Van Gent formula

This is an alternative formula for computing the 2% runup values on smooth or rough slopes, by Van Gent (1999). The computation procedure is less complicated than the TAW (2002a) formula as it does not involve an iteration process and the results are therefore insensitive to numerical solution techniques. The formula has been developed based on numerical and physical model tests and was the first runup formula to be based on the spectral period $T_{m-1,0}$.

Input

The formula makes use of the following parameters:

H_{m0}	Spectral significant wave height at the toe of the structure	(m)
$T_{m-1,0}$	Spectral wave period ($T_p = 1.1T_{m-1,0}$)	(s)
$\cot(\alpha_{s1})$	Lower slope angle of the structure	(-)
$\cot(\alpha_{s2})$	Upper slope angle of the structure	(-)
B	Berm width	(m)
d_h	Depth of berm below SWL (negative if above SWL)	(m)
γ_f	Reduction factor for roughness	(-)
β	Angle of wave attack with respect to structure	(deg)
c_0	1.35 (recommended value in Van Gent formula)	(-)
c_1	4.7 (recommended value in Van Gent formula)	(-)

Technical background

$$\begin{cases} R_{u2\%} = (\gamma H_{m0})\, c_0 \xi_{s,-1} & \text{for } \xi_{s,-1} < p \\ R_{u2\%} = (\gamma H_{m0})\left[c_1 - (c_2 / \xi_{s,-1})\right] & \text{for } \xi_{s,-1} \geq p \end{cases}$$

in which $p = 0.5c_1/c_0$, $c_2 = 0.25c_1^2/c_0$ and $\gamma = \gamma_f\gamma_\beta$.

The reduction factor for oblique wave attack is:

$$\begin{cases} \gamma_\beta = 1-\left(0.0022\times|\beta|\right) & for\ 0^o \le |\beta| \le 80^o \\ \gamma_\beta = 1-\left(0.0022\times 80\right) & for\ 80^o \le |\beta| \le 90^o \end{cases}$$

where the angle β is defined as the angle between the wave direction and the line normal to the dike (β= 0 means perpendicular wave attack).

$$\xi_{s,-1} = \frac{1/\cot\alpha_{rep}}{\sqrt{2\pi H_{m0}/\left(gT_{m-1,0}^2\right)}}$$

in which $cot(\alpha_{rep})$ is the representative structure slope angle.

Computation of the representative slope α_{rep}

If the dike has a different lower and upper slope a representative slope must be computed. The definition of the representative slope is given by taking the average slope between a point $2H_{m0}$ under the still water level and $2H_{m0}$ above the still water level. This average slope can be used only in this formula and is then defined as:

$$d_h > 2H_{m0} \qquad \cot\left(\alpha_{rep}\right) = \cot\left(\alpha_{s2}\right)$$

$$d_h < -2H_{m0} \qquad \cot\left(\alpha_{rep}\right) = \cot\left(\alpha_{s1}\right)$$

$$\begin{cases} -2H_{m0} \le d_h \le 2H_{m0} \\ \cot\left(\alpha_{rep}\right) = \dfrac{L_1 + L_2 + B}{4H_{m0}} \\ L_1 = \left(2H_{m0} - d_h\right)\cot\left(\alpha_{s1}\right) \\ L_2 = \left(2H_{m0} - d_h\right)\cot\left(\alpha_{s2}\right) \end{cases}$$

Limits on input parameters

0.5	<	$\xi_{s,-1}$	≤	50
1.5	≤	$\cot(\alpha_{s1})$	≤	8.0
1.5	≤	$\cot(\alpha_{s2})$	≤	8.0
0.01 m	≤	H_{m0}	≤	20 m
0.5 s	≤	$T_{m-1,0}$	≤	30 s
0.005	≤	$2\pi H_{m0}/gT^2_{m-1,0}$	≤	0.07

3.2.2 Structural response factors

3.2.2.1 Rock armour layers

The most important parameter forming relationship between the structure and the wave conditions is the stability parameter $H/\Delta D$. For the design of rubble mound structures this parameter can vary between 1 (statically stable breakwater) and 6 (berm breakwater).

$$\Delta = \rho_a / \rho_w - 1$$

in which ρ_a (kg/m^3) is the mass density of the rock armour and ρ_w (kg/m^3) is the mass density of water. The wave height H is usually the significant wave height H_s at the toe of the structure, defined either by the statistical definition, the average of the highest one third of the waves or by the spectral definition $H_{m0} = 4m_0^{0.5}$, where m_0 is the area under the energy density spectrum and therefore gives a representative value of the total wave energy. For deep water both definitions give more or less the same wave height. For shallow water conditions substantial differences may be present due to the process of wave breaking.

The diameter D used in the definition is related to the average mass of the rock and is called the nominal diameter:

$$D_{n50} = \left(\frac{M_{50}}{\rho_a} \right)^{1/3}$$

where

D_{n50}	Nominal diameter	(m)
M_{50}	Median mass of unit given by 50% on mass distribution curve	(kg)

With these definitions the parameter $H/\Delta D$ becomes $H_s / \Delta D_{n50}$.

Another important structural parameter is the surf similarity parameter, which relates the slope angle to the wave period or wave steepness, and which gives a classification of different types of wave breaking.

The surf similarity parameter or breaker parameter is defined as $\xi = \tan\alpha / s^{0.5}$, where α is the slope angle of the structure and s is the wave steepness H/L. If the wave length L is based on the peak wave period T_p then s is denoted by s_p; if L is based on the mean wave period T_m then s is denoted by s_m, etc.

3.2.2.2 Stability formula rock - Hudson (1975)

The original Hudson formula is written as follows:

$$M_{50} = \frac{\rho_a H^3}{K_D \Delta^3 \cot \alpha_s}$$

In which K_D is a stability coefficient that takes into account all other variables. K_D-values suggested for design correspond to a "no damage" condition where up to 5% of the armour units may be displaced. In the 1973 edition of the Shore Protection Manual (SPM, 1984), the values given for K_D for rough, angular stone in two layers on a breakwater trunk were:

- $K_D = 3.5$, for breaking waves,
- $K_D = 4.0$, for non-breaking waves.

The definition of breaking and non-breaking waves is different from plunging and surging waves, which were described in Stability formulae rock - Van der Meer (1993) and Cumulative damage after a sequence of storms. A breaking wave means that the wave breaks due to the foreshore in front of the structure directly on the armour layer. It does not describe the type of breaking due to the slope of the structure itself.

No tests with random waves had been conducted and it was suggested to use H_s in the equation below. However, in the 1984 version of the SPM the advice given was more cautious and recommended to use $H = H_{1/10}$, the average of the highest 10 percent of all waves. For the case considered above the value of K_D for breaking waves was revised and lowered from 3.5 to 2.0 (non-breaking waves it remained 4.0).

The main advantages of the Hudson formula are its simplicity, and the wide range of armour units and configurations for which values of K_D have been derived. The use of $K_D\cot(\alpha_s)$ does not always describe the effect of the slope angle in a sufficient way. It may therefore be convenient to define a single stability number without this $K_D\cot(\alpha_s)$. Moreover, it may often be more helpful to work in terms of a linear armour size, such as a typical or nominal diameter. The Hudson formula can be rearranged to:

$$\frac{H_s}{\Delta D_{n50}} = \left(K_D \cot \alpha_s\right)^{1/3}$$

The equation above shows that the Hudson formula can be written in terms of the structural parameter $H_s/\Delta D_{n50}$ which was discussed in Rock armour layers.

Input

[H_s	Incident significant wave height	(m)]
[M_{50}	Armour unit mass	(kg)]
[$\%D$	Damage level	(%)]
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
$\cot(\alpha_s)$	Slope angle of the structure	(-)
K_D	Stability coefficient	(-)
α_{Hs}	Wave height factor	(-)

Parameters between brackets [..] indicate which particular parameter may be selected as an input parameter or as an output parameter.

The wave height factor α_{Hs} is used in the Hudson formula to compute the design wave height H from the significant wave height H_s. As described above, according to the Shore Protection Manual (1984) the design wave height of $H_{1/10} = 1.27\ H_s$ may be applied for breakwaters subject to breaking waves. The default value of α_{Hs} should be 1.27.

With the Hudson formula it is also possible to define both H_s and M_{50} as input parameters. In this case the output will be the damage level $\%D$.

Formula

The armour size for the "no damage" criterion, given the design wave height is:

$$M_{50} = \frac{\rho_a \left(\alpha_{Hs} H_s\right)^3}{K_D \Delta^3 \cot\left(\alpha_s\right)}$$

and

$$D_{n50} = \left(\frac{M_{50}}{\rho_a}\right)^{1/3}$$

Alternatively, the "no damage" wave height $H_{s,D=0}$ is:

$$H_{s,D=0} = \frac{\Delta}{\alpha_{Hs}} \left(\frac{M_{50} K_D \cot\left(\alpha_s\right)}{\rho_a}\right)^{1/3}$$

If both a wave height and an armour size are given as input parameters, the output is the amount of damage expressed as a percentage. If the input wave height is higher than the "no damage" wave height the amount of damage is calculated based on the coefficients in the following table (SPM, 1984):

Armour	**Relative**	**Damage D in percent**[1]						
type	**wave ht**	0 – 5%	5-10%	10-15%	15-20%	20-30%	30-40%	40-50%
Rough rock	$H_s/H_{s,D=0}$	1.00	1.08	1.14	1.20	1.29	1.41	1.54
Smooth rock	$H_s/H_{s,D=0}$	1.00	1.08	1.19	1.27	1.37	1.47	<u>1.56</u>[2]
Tetrapods and Quadripods	$H_s/H_{s,D=0}$	1.00	1.09	1.17[3]	1.24[3]	1.32[3]	1.41[3]	1.50[3]
Tribar	$H_s/H_{s,D=0}$	1.00	1.11	1.25[3]	1.36[3]	1.50[3]	1.59[3]	<u>1.64</u>[3]
Dolosse	$H_s/H_{s,D=0}$	1.00	1.10	<u>1.14</u>[3]	<u>1.17</u>[3]	<u>1.20</u>[3]	<u>1.24</u>[3]	<u>1.27</u>[3]

[1] all values for breakwater trunk, n=2, randomly placed armour and non-breaking waves
[2] underlined values are interpolated or extrapolated
[3] effects of unit breakage <u>NOT</u> included - actual damage may be significantly higher

Table 1 Amount of damage according to Hudson formula

3.2.2.3 Stability formulae rock - Van der Meer (1993)

This option computes the stability of rock slopes under random wave attack for statically stable conditions. It uses the Van der Meer stability formulae for plunging and surging waves. The main basic assumptions for the formulae are:

- a rubble mound structure with an armour layer consisting of rock;
- little or no overtopping (less than 10% - 15% of the waves); and
- the slope of the structure should be generally uniform.

Technical background

The damage to the armour layer can be given as a percentage of displaced rocks related to a certain area (the whole or a part of the layer). In this case, however, it is difficult to compare various structures as the damage figures are related to different totals for each structure. Another possibility is to describe the damage by the erosion area around still-water level. When this erosion area is related to the size of the rocks, a dimensionless damage level is presented which is independent of the size (slope angle and height) of the structure. For the Van der Meer formulae this damage level is defined by:

$$S = \frac{A_e}{D_{n50}^2}$$

where

S	Damage level	(-)
A_e	Erosion area around still-water level	(m^2)

A plot of a damaged structure, where both settlement and displacement are taking into account is shown in Figure 17. A physical description of the damage, S, is the number of squares with a side D_{n50} which fit into the eroded area. Another description of S is the number of cubic stones with a side of D_{n50} eroded within a D_{n50} wide strip of the structure. The actual number of stones eroded within this strip can be more or less than S, depending on the porosity, the grading of the armour rocks and the shape of the rocks. Generally the actual number of rocks eroded in a D_{n50} wide strip is equal to 0.7 to 1 times the damage S.

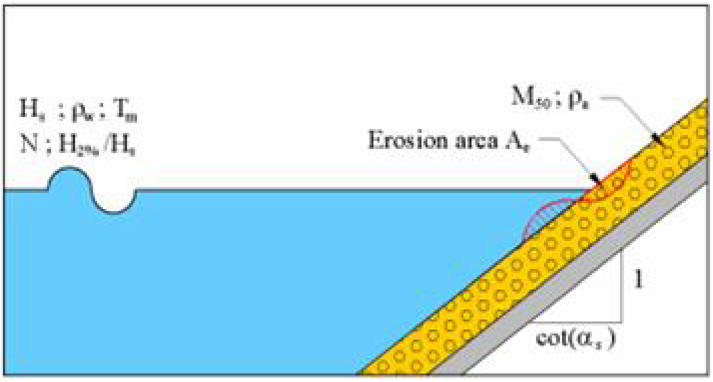


Figure 17 Damage level S based on erosion area A_e.

The limits of S depend mainly on the slope angle of the structure. For a two diameter thick armour layer the values in Table 2 can be used. The initial damage of S = 2-3, according to the criterion of the Hudson formula gives 0-5% damage. Failure is defined as exposure of the filter layer. For values of S higher than 15-20 the deformation of the structure results in a S-shaped profile and should be called dynamically stable.

Slope	Initial damage	Intermediate damage	Failure
1:1.5	2	3 – 5	8
1:2	2	4 – 6	8
1:3	2	6 – 9	12
1:4 - 1:6	3	8 – 12	17

Table 2 Design values of S for a two diameter thick armour layer

In BREAKWAT the user can calculate one of the following items:

- the required armour size M_{50} for a given damage level and storm condition, or
- the maximum wave height for a given structure and damage level, or
- the damage level S for a given structure and storm condition.

These choices are summarised as follows:

Output	Required input
S	H_s and M_{50}
M_{50} and D_{n50}	H_s and S
H_s	M and S

Table 3 Rock stability - output parameter options and required input parameters

The 'start of damage' values for S in Table 2 are similar to the damage percentage obtained from the Hudson formula.

Input parameters

[H_s	Incident significant wave height	(m)]
[M_{50}	Armour unit mass	(kg)]
[S	Damage level	(-)]
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
T_m	Mean wave period	(s)
N	Number of incoming waves	(-)
P	Notional permeability	(-)
$H_{2\%}/H_s$*	Wave height ratio	(-)

Parameters between brackets [..] indicate which particular parameter may be selected as an input parameter or as an output parameter.

*Recent studies (Smith *et al.*, 2002) indicate that the use of a reduction factor for wave breaking on a shallow foreshore (1:100) will lead to an underestimation of the damage. It is therefore recommended to apply the factor $H_{2\%}$ / H_s = 1.4 to such situations, even if the actual value is less than 1.4.

The notional permeability factor P is an empirical constant which is dependent on the structure type. It can vary between 0.1 for a relatively impermeable structure to 0.6 for a permeable structure (see figure below).

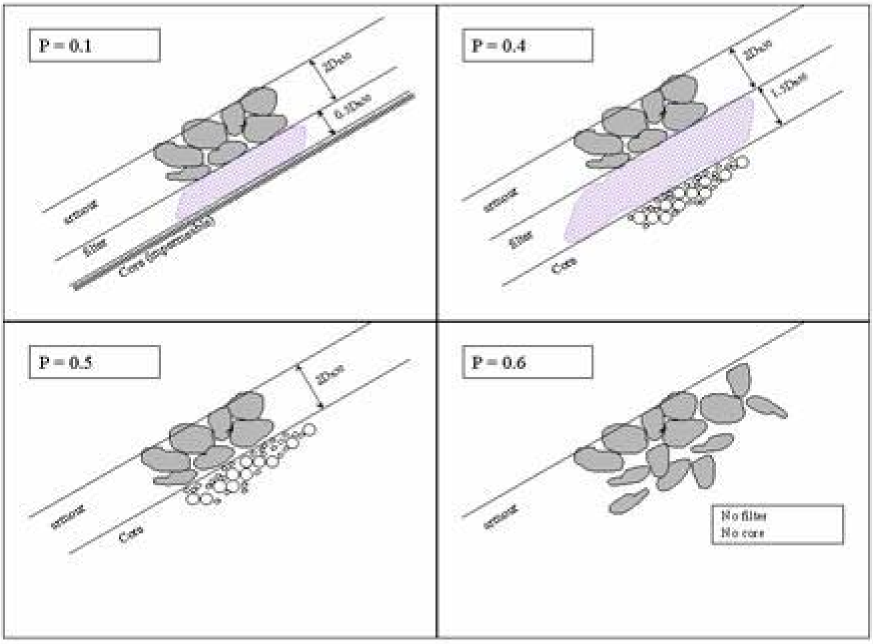


Figure 18 Notional permeability factor P for the Van der Meer stability formulae

The permeability of the structure has an influence on the stability of the armour layer. This depends on the size of filter layers and core. The lower limit of P is an armour layer with a thickness of two diameters on an impermeable core (sand or clay) and with only a thin filter layer. This lower boundary is given by P = 0.1. The upper limit of P is given by a homogeneous structure which consists only of armour rocks. In that case P = 0.6. Two other values are shown in Figure 18 and each particular structure should be compared with the given structures in order to make an estimation of the P factor. It should be noted that P is not a measure of porosity. The estimation of P from Figure 18 for a particular structure must, more or less, be based on engineering judgement. Although the exact value may not precisely be determined, a variation of P around the estimated value may well give an idea about the importance of the permeability.

Limits

0.1	$\leq$	P	$\leq$	0.6
0.01 m	$<$	H_s	$<$	20 m
0.5 s	$<$	T_p	$<$	30 s
1.1	$<$	$\cot(\alpha_s)$	$<$	7.0
0.005	$\leq$	$2\pi H_s / gT^2_p$	$\leq$	0.06
0	$<$	N	$<$	7500
1.10	$<$	$H_{2\%}/H_s$	$<$	1.40
2000	$<$	ρ_a	$<$	3100 kg/m^3

Formulae

The formula is dependent on the type of wave breaking on the structure. The waves can be classified as either "plunging" or "surging". This classification is made based on the value of the breaker parameter, ξ_m.

The stability formula for plunging waves, if $\xi_m < \xi_{mc}$ OR $\cot(\alpha_s) \geq 3.5$:

$$\frac{H_s}{\Delta D_{n50}} = \left(\frac{H_{2\%}}{H_s}\right)^{-1} 8.68 P^{0.18} \left(\frac{bS}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$$

where the term b is a correction term for N < 1000 or N > 5000 waves.

The stability formula for surging waves, if $\xi_m > \xi_{mc}$ AND $\cot(\alpha_s) < 3.5$:

$$\frac{H_s}{\Delta D_{n50}} = \left(\frac{H_{2\%}}{H_s}\right)^{-1} 1.4 P^{-0.13} \left(\frac{bS}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\alpha_s}\,\xi_m^P$$

where the transition from plunging to surging waves occurs at the critical value of the breaker parameter, ξ_{mc}:

$$\xi_{mc} = \left[6.2 P^{0.31} \sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}}$$

It should be noted that for relatively small wave steepnesses dicontinuities can occur in the formulae described above around $\cot(\alpha_s) = 3.5$.

3.2.2.4 Stability formulae rock – Modified Van der Meer (1988a) by Van Gent et al (2003)

This option computes the stability of rock slopes under random wave attack for statically stable conditions. The main basic assumptions for the formulae are:

- a rubble mound structure with an armour layer consisting of rock;
- little or no overtopping, and
- the slope of the structure should be generally uniform.

Technical background

Based on analysis of the stability of rock slopes for conditions including situations with shallow foreshores, it was proposed by Van Gent *et al* (2003) to modify the formulae of Van der Meer (1988a), see also Stability formulae rock - Van der Meer (1993), to extend its field of applications. The modified formulae are considered valid for **both deep water and shallow water conditions**. This concerns the following modifications:

- A different wave period is used to take the influence of the shape of the wave energy spectra into account (*i.e.* the use of the spectral wave period $T_{m-1,0}$ instead of the mean wave period from time-domain analysis T_m).
- The coefficients are re-calibrated.
- The confidence levels are adapted.

These modifications were based on the formulae by Van der Meer (1988a) for shallow water conditions in which the ratio $H_{2\%}/H_s$ is used. The method by Battjes and Groenendijk (2000) can be used to obtain estimates of $H_{2\%}$. The modified formulae read for plunging waves ($\xi_{s,-1} < \xi_c$):

$$\frac{S}{\sqrt{N}} = \left(\frac{1}{c_{plunging}} P^{-0.18} \xi_{s,-1}^{0.5} \frac{H_s}{\Delta D_{n50}} \left(\frac{H_{2\%}}{H_s} \right) \right)^5$$

and for surging waves ($\xi_{s,-1} \geq \xi_c$):

$$\frac{S}{\sqrt{N}} = \left(\frac{1}{c_{surging}} P^{0.13} \xi_{s,-1}^{-P} \tan\alpha^{0.5} \frac{H_s}{\Delta D_{n50}} \left(\frac{H_{2\%}}{H_s} \right) \right)^5$$

with the non-dimensional damage level (Stability formulae rock - Van der Meer (1993)):

$$S = A_e / D_{n50}^2$$

in which A_e is the eroded area in a cross-section, and the relative buoyant density Δ (-) defined as ρ_a/ρ_w -1.

The transition from plunging to surging waves can be calculated using a critical value of ξ_c according to:

$$\xi_{mc} = \left[\frac{c_{plunging}}{c_{surging}} P^{0.31} \sqrt{\tan\alpha} \right]^{\frac{1}{P+0.5}}$$

The coefficients for plunging and surging waves are:

- $c_{plunging} = 8.4$
- $c_{surging} = 1.3$

The spectral surf-similarity parameter is defined as:

$$\xi_{s,-1} = \frac{\tan\alpha}{\sqrt{\frac{2\pi H_s}{g T_{m-1,0}^2}}}$$

The limits of S depend mainly on the slope angle of the structure. For a two diameter thick armour layer the values in Table 3a can be used. The initial damage of S = 2-3, according to the criterion of the Hudson formula gives 0-5% damage. Failure is defined as exposure of the filter layer. For values of S higher than 15-20 the deformation of the structure results in a S-shaped profile and should be called dynamically stable.

Slope	Initial damage	Intermediate damage	Failure
1:1.5	2	3 – 5	8
1:2	2	4 – 6	8
1:3	2	6 – 9	12
1:4 - 1:6	3	8 – 12	17

Table 3a Design values of S for a two diameter thick armour layer

Besides probabilistic approaches using the standard deviation, one may use a more simple approach, not based on formulae that describe the main trend through the data, but on more conservative formulae with a lower probability of exceeding the predicted damage. For instance formulae with a confidence level of 95% (*i.e.*, 5% of the data leads to a higher amount of damage and 95% of the data leads to a lower amount of damage) can be used, in combination with the following formula:

$$S_{5\%} = S_0 + S$$

(Depending on the selected output parameter H_s or M_{50} is calculated using $S_{5\%}$). For S_0 the value 2 is used. This damage level corresponds to start of damage and is considered acceptable for all types of rock slopes. S is calculated with the formula mentioned above, with the following coefficients for plunging and surging waves:

- $c_{plunging}$ = 7.25
- $c_{surging}$ = 1.05

$S_{5\%}$ is calculated when the ‘confidence bands’-check box is selected.

Input parameters

[H_s	Incident significant wave height	(m)]
[M_{50}	Armour unit mass	(kg)]
[S	Damage level	(-)]
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
$T_{m-1,0}$	Spectral wave period ($T_{m-1,0}=m_{-1}/m_0$ with $m_n={}_0\int^{\infty} f^n\, S(f)\, df$ with n = -1 or 0)	(s)
N	Number of incident waves at toe of structure	(-)
P	Notional permeability factor (Stability formulae rock - Van der Meer (1993))	(-)
$H_{2\%}/H_s$	Wave height ratio	(-)
cotα	Slope angle	(-)

Parameters between brackets [..] indicate which particular parameter may be selected as an input parameter or as an output parameter.

Limits

0.1	≤	P	≤	0.6
0.5	≤	H_s / ΔD_{n50}	≤	4.5
1.3	≤	$\xi_{s,-1}$	≤	15
0	≤	S	≤	30
2	≤	cotα	≤	4
0	≤	N	≤	3000
1.2	<	$H_{2\%}/H_s$	<	1.4

3.2.2.5 Stability formula rock –Van Gent et al (2003)

This option computes the stability of rock slopes under random wave attack for statically stable conditions. The main basic assumptions for the formulae are:

- a rubble mound structure with an armour layer consisting of rock;
- little or no overtopping, and
- the slope of the structure should be generally uniform.

Technical background

Van Gent *et al* (2003) proposed a stability formula, which is easier and/or simpler to use than the formulae by Van der Meer (1988a) (Stability formulae rock - Van der Meer (1993)), because of the following reasons:

- Although there is an influence of the wave period, this influence is considered small compared to the amount of spreading in the data due to other reasons. Therefore, the wave period is not used in this formula and there is no separation between "plunging waves" and "surging waves". That makes this formula useful when no information is available on the wave period.
- The influence of the ratio $H_{2\%} / H_s$ is present, but it is considered small. Therefore, this ratio has been omitted.
- The influence of the permeability of the structure is incorporated in a direct way by using a structure parameter, *i.e.* the diameter of the core material ($D_{n50\text{-}core}$).

The influence of the number of waves (N) is the same as found by Thompson and Shuttler (1975) and Van der Meer (1988a). The influence of the parameter $H_s / \Delta D_{n50}$ is the same as found by Van der Meer (1988a). The formula reads:

$$S = \sqrt{N}\left(0.57\frac{H_s}{\Delta D_{n50}}\tan\alpha^{0.5}\frac{1}{1+D_{n50core}/D_{n50}}\right)^5$$

with the non-dimensional damage level (Stability formulae rock - Van der Meer (1993)):

$$S = A_e / D_{n50}^2$$

in which A_e is the eroded area in a cross-section, and the relative buoyant density Δ (-) defined as ρ_a / ρ_w -1.

The influence of the permeability of the structure is incorporated by using the ratio $D_{n50\text{-}core}$ / D_{n50}. The influence of filters is not accounted for in this ratio, which means that no filter or a rather standard filter of 2-3 layers thick is assumed here. When the core consists of rock material with a very wide grading, it is recommended to use the $D_{n15core}$ (which corresponds in most cases reasonably well to the lower limit of the grading) instead of the $D_{n50core}$. This stability formula is valid for **shallow, as well as for deep water conditions**.

The limits of S depend mainly on the slope angle of the structure. For a two diameter thick armour layer the values in Table 3b can be used. The initial damage of S = 2-3, according to the criterion of the Hudson formula gives 0-5% damage. Failure is defined as exposure of the filter layer. For values of S higher than 15-20 the deformation of the structure results in a S-shaped profile and should be called dynamically stable.

Slope	Initial damage	Intermediate damage	Failure
1:1.5	2	3 – 5	8
1:2	2	4 – 6	8
1:3	2	6 – 9	12
1:4 - 1:6	3	8 – 12	17

Table 3b Design values of S for a two diameter thick armour layer

Besides probabilistic approaches using the standard deviation, one may use a more simple approach, not based on formulae that describe the main trend through the data, but on more conservative formulae with a lower probability of exceeding the predicted damage. For instance formulae with a confidence level of 95% (*i.e.*, 5% of the data leads to a higher amount of damage and 95% of the data leads to a lower amount of damage) can be used, in combination with the following formula:

$$S_{5\%} = S_0 + S$$

(Depending on the selected output parameter H_s or M_{50} is calculated using $S_{5\%}$). For S_0 the value 2 is used. This damage level corresponds to start of damage and is considered acceptable for all types of rock slopes. S is calculated with the formula mentioned above, with a factor 0.68 instead of 0.57. $S_{5\%}$ is calculated when the 'confidence bands'-check box is selected.

Input parameters

[H_s	Incident significant wave height	(m)]
[M_{50}	Armour unit mass	(kg)]
[S	Damage level	(-)]
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
$D_{n50\text{-}core}$ / D_{n50}	Core material	(-)
N	Number of incoming waves	(-)
$\cot\alpha$	Slope angle	(-)

Parameters between brackets [..] indicate which particular parameter may be selected as an input parameter or as an output parameter.

Limits

0.1	≤	$D_{n50\text{-}core}/D_{n50}$	≤	0.3
0.5	≤	$H_s / \Delta D_{n50}$	≤	4.5
0	≤	S	≤	30
2	≤	$\cot\alpha$	≤	4
0	≤	N	≤	3000

3.2.2.6 Cumulative damage after a sequence of storms

The formula 'Van der Meer, Cumulative Storms' calculates the total damage after various wave conditions for a given structure. This is a situation which actually occurs in nature, the damage after a certain storm condition is dependent on the damage after previous storms. Lower and upper limits of the damage level S for a two-diameter thick armour layer are:

Slope	Initial damage	Intermediate damage	Failure
1:1.5	2	3 - 5	8
1:2	2	4 - 6	8
1:3	2	6 - 9	12
1:4 - 1:6	3	8 - 12	17

Table 4 Description of damage levels

The 'start of damage' values for S are similar to the damage percentage obtained from the Hudson formula.

For the formula 'Van der Meer, Cumulative Storms' an iterative solution of the Van der Meer stability equations is needed. For this option only the total damage S or the required rock size M_{50} can be computed.

The same input parameters as for the stability calculations for a single wave condition (Rock armour layers) are required. Only with this option the wave conditions have to be given in a separate table called '(SEQ_STORMS) Cumulative damage input table'. Also the significant wave height (H_s) and mean wave period (T_m) , the number of waves (N) and the wave height distribution factor ($H_{2\%}/H_s$) of each storm are required.

The output consists of a table of values for the parameters

- S (damage level)
- ξ_m (breaker parameter)
- s_m (wave steepness)
- $H_s/\Delta D_{n50}$ (stability number).

For the damage level S, the value after each storm event is presented.

3.2.2.7 Probability of damage to a rock armour layer

The probabilistic design approach takes into account the fact that all parameters describing an event have a certain variability. Each parameter can be defined by a mean value with a standard deviation and a description of the type of probability distribution function that best describes its behaviour. The user must have this information for each input parameter before beginning with the calculations.

With that information Level II computations are made within which the reliability of the structure is determined for all possible combinations of parameters. The reliability (Z) is defined as the difference between resistance (R) and load (S). This is called the reliability function:

$$Z = R - S$$

The sum of the probabilities for all combinations of resistance and load parameters for which $Z < 0$ is computed, is defined as the probability of failure. Conditions for which $Z = 0$ are referred to as the 'Limit State' and conditions for which $Z > 0$ are considered survival conditions.

With the Van der Meer formulae for rock stability as an example, the probability that the defined damage level S is exceeded in one year (P_F) can be computed by evaluating

$$P_F = \underset{Z<0}{\iiint\iiint\iiint} f\left(S, D_{n50}, x_2, \Delta, \cot\alpha_s, P, H_s, F_{Hs}, s_{0m}\right)$$
$$dS\,dD_{n50}\,dx_2\,d\Delta\,d\cot\alpha_s\,dP\,dH_s\,dF_{Hs}\,ds_{0m}$$

in which $f(x_1,x_2, \ldots)$ is the joint probability density function of all resistance and load parameters. The method used here to solve this function is a First Order Second Moment analysis, in which the limit state surface (Z=0) is linearized by first order Taylor expansion. This linearized function is a Gaussian random variable characterised by its expected mean value μ_Z and its variance σ^2_Z. The probability that the linearized function Z_l is less than zero is approximated as

$$P\left(Z_l < 0\right) = \Phi\left\{\frac{-\mu_Z}{\sigma_Z}\right\} = \Phi\left\{-\beta\right\}$$

where Φ ( ) is the standard (cumulative) Gaussian probability function of a random variable, μ_Z is the expected mean value of Z_l , σ_Z is the variance of Z_l and β is called the reliability index.

The integral is solved using a Level II Approximate Full Distribution Approach (AFDA). With this method the distribution types of all parameters are transformed to an equivalent Gaussian probability function such that

$$u = \Phi^{-1}\left\{F_X\left(x\right)\right\}$$

in which u is the transformed (Gaussian) variable of non-Gaussian variable x and $F_X(x)$ is the cumulative probability function of non-Gaussian variable x

The relative contribution of each of the variables (x) to the total variance of the Z_l function is expressed as

$$\alpha_x^2 = \left(\frac{dZ}{dx}\right)^2 \frac{\sigma_x^2}{\sigma_Z^2}$$

The α^2 values reflect the relative importance of each variable in the reliability analysis.

For the rock slope stability formulae of Van der Meer (1993), the probability of exceedance of a specified damage level can be computed. This probability can be made for different time spans, or return periods following the procedure described in Van der Meer and Pilarczyk (1987).

The rock armour stability formulae can be rearranged into a reliability function as:

$$Z_p = S^{0.2} 8.68 \left(\frac{H_{2\%}}{H_s}\right)^{-1} P^{0.18} \left(\cot\alpha_s\right)^{0.5} \Delta D_{n50} - \left(H_s + F_{Hs}\right)\left(s_{0m}\right)^{-0.25} N^{0.1}$$

$$Z_s = S^{0.2} 1.4 \left(\frac{H_{2\%}}{H_s}\right)^{-1} P^{-0.13} \left(\cot\alpha_s\right)^{0.5-P} \Delta D_{n50} - \left(H_s + F_{Hs}\right)\left(s_{0m}\right)^{0.5P} N^{0.1}$$

For a given condition the value of the breaker parameter ξ_m determines which formula is relevant (type of breaking). For $\xi_m \leq \xi_{mc}$ Z_p is the relevant function and for $\xi_m > \xi_{mc}$ Z_s is the relevant function.

In order to compute the failure probability, however, for each given condition both Z functions have to be evaluated for their respective failure probabilities. In addition the probability that $\xi_m \leq \xi_{mc}$, defined as P(Z_ξ) is also evaluated. This is achieved by evaluating the relation:

$$Z_\xi = \xi_m - \xi_{mc}$$

The total "failure" probability P(Z) is then computed from the results of these three Z-functions.

This yields to the probability that the specified condition will occur in one year. The probability of exceedance of a given condition (damage level) for an X year period is evaluated by:

$$P\left[Z < 0; X\ years\right] = 1 - \left(1 - P\left[Z < 0; 1\ year\right]\right)^X$$

For the case of the stability of a rubble mound breakwater with rock armour using the formulae of Van der Meer, all parameters except the long-term variation in wave height can be described by a normal distribution. The long-term variation of H_s can be described by a Weibull distribution.

$$P(H_s) = P(H \le H_s) = 1 - \exp\left[-\left\{ \frac{H_s - C}{B} \right\}^k \right]$$

in which C is the lower boundary of H_s (for a return period of one year), B is the scale parameter and k is the shape parameter.

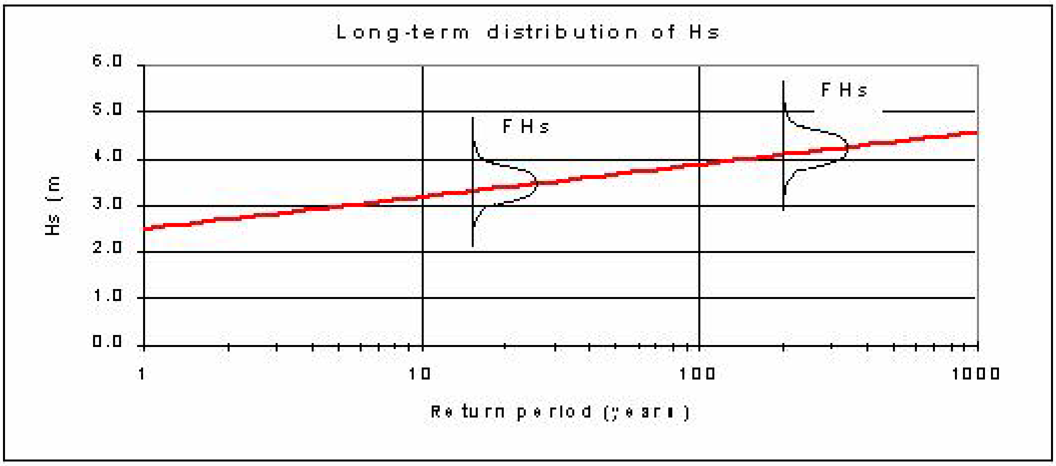


Figure 19 Example of long-term variation in H_s, with uncertainty parameter FH_s

The parameter FH_s in Figure 19 describes the uncertainty of the long term distribution function applied. It can also be described by a normal distribution with a mean value of 0 and a certain standard deviation.

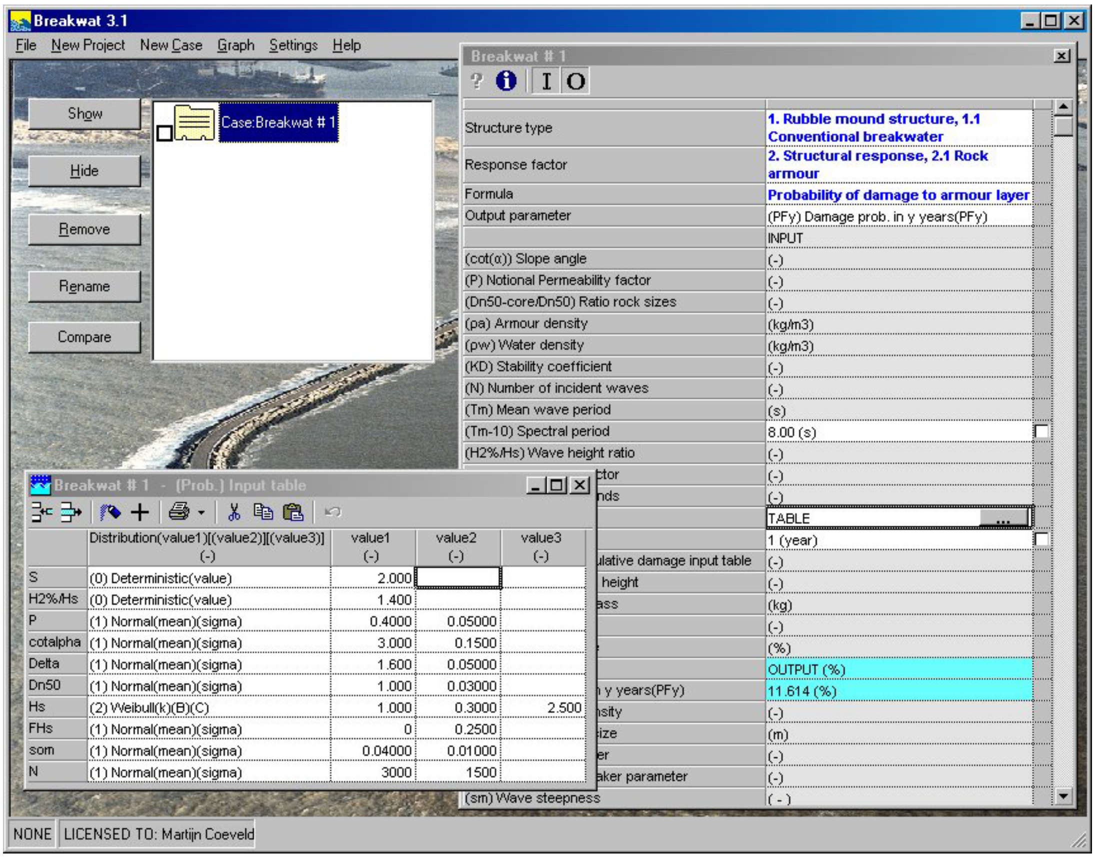


Figure 20 Input screen for probabilistic calculation

Input

The input screen for this option differs from the standard input screen. By clicking the grey table button behind '(Prob.) Input table', the table with the default input values is displayed, see Figure 20.

For each parameter in the input table at least 1 value must be defined. A total of 3 values is possible. Based on the number of input values entered for a given parameter the type of distribution is automatically determined, see Table 6.

Type of distribution	**Meaning of the input 'Value#'-parameters**		
	Value 1	Value 2	Value 3
Deterministic	number	-	-
Normal	mean (μ)	standard deviation (σ)	-
Weibull	shape parameter (k)	scale parameter (B)	lower bound (C)

Table 6 Definition of 'Value#'-parameters for probabilistic input table

After values have been entered for all parameters in the '(Prob.) Input table', the user can also enter the number of years to carry out the calculations for. This input parameter is located on the general input screen. The default value is 1 year. By checking the box on the right you can also define an array, so that the failure probability is evaluated for different return periods (eg. 1, 10, 50 and 100 years). Also to be computed are then the reliability index β, the design point in transformed variable space u and the relative parameter contribution to the total 'failure' probability $(-u/\beta)^2$.

Example

In an example given in Van der Meer and Pilarczyk (1987) with parameters in Table 7.

parameter	distribution	mean	standard deviation
S	deterministic	4, 6, 8, 10	-
D_{n50}	normal	1.0 m	0.03 m
Δ	normal	1.6	0.05
$\cot\alpha_s$	normal	3.0	0.15
P	normal	0.5	0.05
N	normal	3000	1500
H_s	Weibull	B=0.3	C=2.5; k=1.0
$H_{2\%}/H_s$	deterministic	1.4	-
F_{Hs}	normal	0	0.25
s_{om}	normal	0.04	0.01

Table 7 Input parameters example

The input screen for the first damage level value (S=4) is shown in Figure 21.

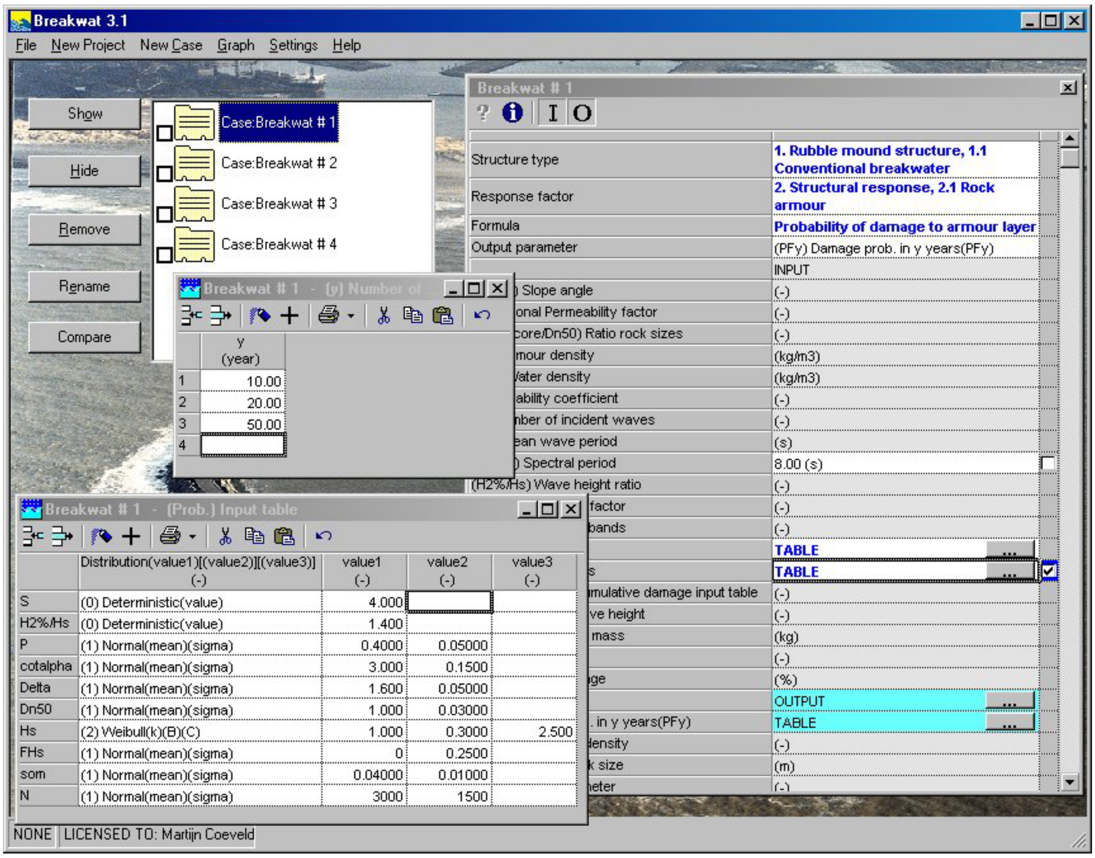


Figure 21 Input screen for damage level S=4.0

Figure 21 all the input values can be seen for both the '(Prob.) Input table'-field and the '(y) Number of years'-field. This is a separate 'Case', named '*S*=4.0'. Also three other 'Cases' are defined, named '*S*=6.0', '*S*=8.0' and '*S*=10.0'. This list of 'Cases' under 'Project:ProbOfDamage' can be seen in the contents window. The input tables for the other 'Cases' are similar to the one shown, the only difference being the value of *S*.

A graph of the results can be made by choosing Graph from the menu bar. The results will be plotted for the active 'Case'. The default settings in the graph definition screen are '(PFy) Damage probability in y years' on the y-axis and '(y) Number of years' on the x-axis. Results from the other cases can be added to the graph by dragging the output field button '(PFy) Damage prob. in y years (PFy)' to the graph. The new series will also be plotted. However, you will have to edit the series name on the graph to get an appropriate title for it. This is achieved by starting the graphical editor (double click anywhere on the graph) and selecting the desired series from the list.

Once these actions have been completed for each 'Case' a graph is created, like in Figure 22.

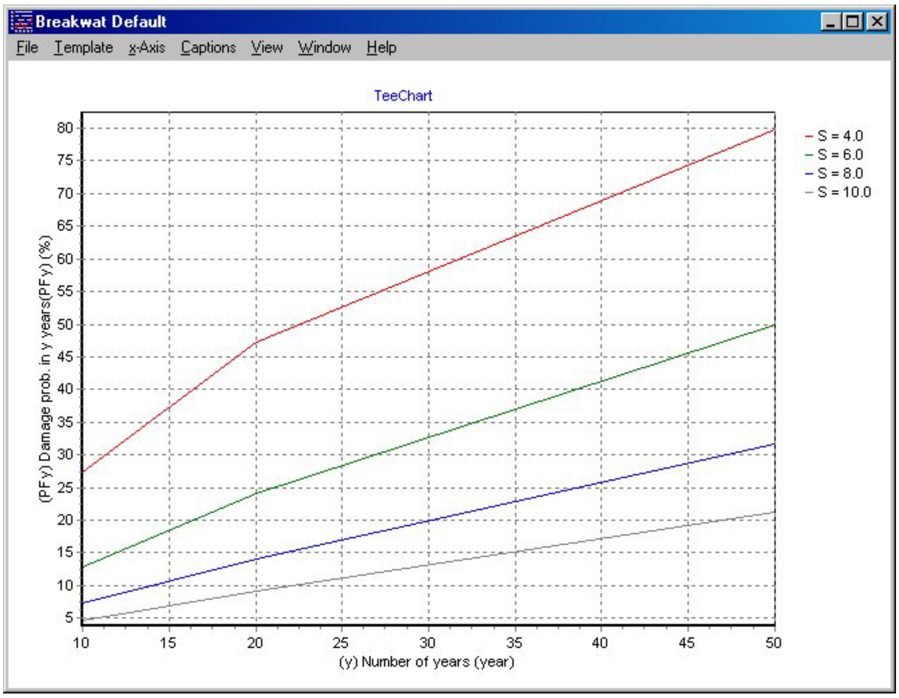


Figure 22 Output graph

3.2.2.8 Rear side stability

This option computes the stability of rock slopes on the rear side of rock armoured structures, taking into account several hydraulic and structural parameters, see Figure 22a. This design guideline is meant for structures of which the stability of the rear slope is not influenced by the stability of the front slope or the crest.

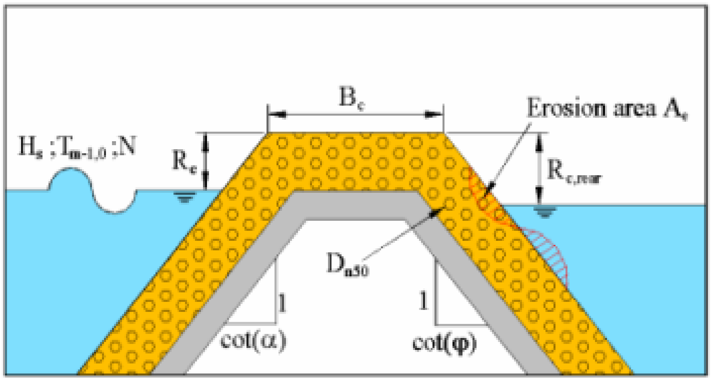


Figure 22a Definition sketch

Technical background

Van Gent and Pozueta (2004) proposed a formula to determine the stone diameter of the armour material at the rear side of a marine or coastal structure for a given damage level:

$$D_{n50} = 0.008\left(\frac{u_{1\%}T_{m-1,0}}{\Delta^{0.5}}\right)\left(\frac{S}{\sqrt{N}}\right)^{-1/6}\cot\varphi^{2.5/6}\left(1+10e^{\frac{-R_{c,rear}}{H_s}}\right)^{1/6}$$

The damage is represented with the non-dimensional damage level (Section Stability formulae rock – Van der Meer (1993)):

$$S = A_e / D_{n50}^2$$

in which A_e is the eroded area in a cross-section, and with a maximum velocity (depth-averaged) at the rear side of the crest during a wave overtopping event, exceeded by 1% of the incident waves. According to Van Gent (2002) this velocity can be calculated with:

$$u_{1\%} = 1.7\left(g\gamma_{f-C}\right)^{0.5}\frac{\left(\left(z_{1\%}-R_c\right)/\gamma_f\right)^{0.5}}{1+0.1B_c/H_s}$$

with a fictitious runup level $z_{1\%}$ (m), which is obtained using the following expression (Van Gent, 2002):

$$z_{1\%}/(\gamma\, H_s) = 1.45\,\xi_{s,-1} \qquad \text{for } \xi_{s,-1} \leq 1.76$$
$$z_{1\%}/(\gamma\, H_s) = 5.1 - 4.48/\xi_{s,-1} \qquad \text{for } \xi_{s,-1} \geq 1.76$$

in which:

- (-) is the reduction factor that takes the effects of angular wave attack (γ_β, which can be approximated by $\gamma_\beta=1-0.0022\beta$, where $\beta \leq 80°$) and roughness (γ_f) into account ($\gamma = \gamma_f\,\gamma_\beta$), and
- $\xi_{s,-1}$ (-) is the surf-similarity parameter defined as $\xi = \tan\alpha / (2\pi H_s / gT_{m-1,0}{}^2)^{0.5}$, with the front slope angle α.

Input parameters

[H_s	Significant wave height at toe of structure	(m)]
[D_{n50}	Nominal diameter of the material on the rear side slope	(m)]
[S	Damage level	(-)]
$T_{m-1,0}$	Spectral wave period ($T_{m-1,0}=m_{-1}/m_0$ with $m_n= {}_0\!\int^\infty f^n\, S(f)\, df$ with n = -1 or 0)	(s)
N	Number of incident waves at toe (dependent on storm duration)	(-)
$R_{c,rear}$	Crest freeboard relative to water level at rear side of crest	(m)
cotα	Front slope angle	(-)
cotφ	Rear side slope angle	(-)
γ_f	Roughness of seaward slope (γ_f = 0.47 for rough rock slopes, γ_f = 0.55 for rock slopes, and γ_f = 1 for impermeable slopes)	(-)
$\gamma_{f\text{-}C}$	Roughness of crest ($\gamma_{f\text{-}C}$ = 0.47 for rough rock slopes, $\gamma_{f\text{-}C}$ = 0.55 for rock crests, and $\gamma_{f\text{-}C}$ = 1 for smooth, impermeable crests)	(-)

R_c	Crest freeboard relative to still water at the seaward side of the crest	(m)
B_c	Crest width	(m)
β	Angle of incident wave ($\beta = 0$ corresponds with perpendicular wave attack)	(deg)

Parameters between brackets [..] indicate which particular parameter may be selected as an input parameter or as an output parameter.

Limits

0	$\leq$	S	$\leq$	30
0.3	$\leq$	R_c / H_s	$\leq$	2.3
0.3	$\leq$	$R_{c,rear} / H_s$	$\leq$	2.3
0	$\leq$	B_c / H_s	$\leq$	6
0	$\leq$	$(z_{1\%} - R_c) / (\gamma_f H_s)$	$\leq$	1.4
2	$\leq$	$\cot\varphi$	$\leq$	4
2	$\leq$	$\cot\alpha$	$\leq$	4
0.014	$\leq$	$s_{m-1,0}$	$\leq$	0.033
0	$\leq$	N	$\leq$	3000

3.2.2.9 Toe berm stability of rubble mound breakwater

In most cases the armour layer on the seaside near the bottom is protected by a supporting toe. If the rock in the toe berm has the same dimensions as the armour layer, it will be stable. However, it is generally desirable to apply a smaller gradation to the toe berm. A simple design graph has been given in the CIRIA/CUR Manual (1991), in which a simple relation was assumed between the stability number $H_s / \Delta D_{n50}$ and the relative depth of the toe h_t / h_s . However this relation, however, is only applicable for depth-limited conditions. Extended studies (Gerding, 1993) and (Van der Meer *et al.*, 1995) provided additional data, which has been analysed and described in Van der Meer (1998). With this new design formula the amount of damage can be quantified by means of the parameter N_{od}, which was described in the section on concrete armour units. For a toe berm the interpretation of the damage level is made as follows (Van der Meer, 1998):

$N_{od} = 0.5$: start of damage (should be used for conceptual design)
$N_{od} = 2.0$: some flattening out of the toe berm
$N_{od} = 4.0$: complete flattening out of the toe berm

These figures apply to a standard toe structure of about 3-5 stones wide and 2-3 stones high.

The Input/Output parameters and the formula are shown below.

Input

[H_s	Significant wave height	(m)]
[N_{od}	Number of units displaced in a width one D_n	(-)]
[D_{n50}	50% size of armour	(m)]
[h_t	Depth of toe wrt still water level	(m)]
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
h	Depth of water wrt MSL	(m)

Parameters in the square brackets [..] may be input or output parameters depending upon the user definition.

Stability factor

$$\frac{H_s}{\Delta D_{n50}} = \left[2 + 6.2\left(h_t / h\right)^{2.7}\right] N_{od}^{0.15}$$

Limits

0.01 m	<	H_s	<	20 m
0.4	<	h_t / h	<	0.9
3	<	h_t / D_{n50}	<	25
2	<	$N_{od}^{-0.15} H_s / (\Delta D_{n50})$		

3.2.2.10 Underlayers and filters

Rubble mound structures in coastal and shoreline protection are normally constructed with an armour layer and one or more underlayers. Sometimes an underlayer is called a filter. The dimensions of the first underlayer depend on the structure type.

Revetments often have a two diameter thick armour layer, a thin underlayer or filter and then an impermeable structure (clay or sand), with or without a geotextile. The underlayer in this case works as a filter. Smaller particles beneath the filter should not be washed through the layer and the filter stones should not be washed through the armour. In this case the geotechnical filter rules are strongly recommended. Roughly these rules give $D_{15\,(armour)}/D_{85(filter)} < 4$ to 5.

Structures such as breakwaters have one or two underlayers followed by a core of rather fine material (quarry-run). The spm (1984) a range of 1/10 to 1/15 of the armour mass for the stone size of the underlayer under the armour layer recommends. This criterion is more strict than the geotechnical filter rules and gives $D_{n50(armour)}/D_{n50(underlayer)} = 2.2 - 2.3$.

A relatively large underlayer has two advantages. First the surface of the underlayer is less smooth with bigger rocks and gives more interlocking with the armour. This is specially the case if the armour layer is constructed of concrete armour units. Second, a large underlayer results in a more permeable structure and therefore has a large influence on the stability (or required mass) of the armour layer.

Therefore, it is recommended to use a size of 1/10 to 1/15 M_{50} of the armour for the mass of the underlayer.

3.2.2.11 Breakwater head

Breakwater heads represent a special physical process. Jensen (1984) described it as follows:

"When a wave is forced to break over a roundhead it leads to large velocities and wave forces. For a specific wave direction only a limited area of the head is highly exposed. It is an area around the still-water level where the wave orthogonal is tangent to the surface and on the lee side of this point. It is therefore general procedure in design of heads to increase the weight of the armour to obtain the same stability as for the trunk section. Alternatively, the slope of the roundhead can be made less steep, or a combination of both".

An example of the stability of a breakwater head relative to that of the trunk section that shows the location of the damage as described in the previous paragraph is shown in Jensen (1984). It is demonstrated that the stability of the head is considerably less than the stability of the trunk at a location between 900 and 1500, relative to the wave direction. Damage is located at about 120°- 150° from the wave angle. This local damage is clearly found by research with long-crested waves. Possibly, the actual damage in prototype may be less concentrated as waves in nature are short-crested and multi-directional. Research in multi-directional wave basins should be undertaken to clarify this aspect.

No specific rules are available for the breakwater head. The required increase in weight can be a factor between 1 and 4, depending on the type of armour unit. The factor for rock is closer to 1.

Another aspect of breakwater heads was mentioned by Jensen (1984). The damage curve for a head is often steeper than for a trunk section. A breakwater head may show progressive damage. This means that if both head and trunk were designed on the same (low) damage level, an (unexpected) increase in wave height can cause failure of the head or a part of it, where the trunk still shows acceptable damage. This aspect is less pronounced for heads which are armoured by rock.

3.2.2.12 Concrete armour units

The Hudson formula was given in Section (Stability formula rock - Hudson (1975)) with KD values for rock. The SPM (1984) gives a table with values for a large number of concrete armour units. The most important ones are: K_D = 6.5 and 7.5 for cubes, K_D = 7.0 and 8.0 for Tetrapods and K_D = 15.8 and 31.8 for Dolosse (see summary in Table 8). For other units one is referred to SPM (1984).

parameter	**Tetrapod** (2 layers)	**Dolosse** (2 layers)	**Core-Loc™** (1 layer)	**Accropode** (1 layer)
K_D , trunk, nonbreaking	8.0	31.8[2)]		
K_D , trunk, breaking	7.0	15.8[2)]	16	12
K_D , head, nonbreaking	4 - 6[1)]	14-16[3)]		
K_D , head, breaking	3.5 - 5[1)]	7-8[3)]	13	
n_v , volumetric porosity	50%	56%	60% - 5m^3 56% 6.3 -12m^3 54% 14 - 22m^3	66% - 5m^3 62% 6.3 -12m^3 58% 14 - 22m^3
k_Δ , layer thickness coeff.	1.04	0.94	1.51	

1) lower value applies to slope 1:3, upper value applies to slope 1:1.5
2) refers to no damage criteria; if no rocking is desired, reduce K_D by 50%
3) lower value applies to slope 1:3, upper value applies to slope 1:2

Table 8 K_D, n_v and k_Δ values for Tetrapods, Dolosse, Core-Loc™ and Accropode units (SPM, 1984, and US Army, 1997)

The wave height factor α_{Hs} is used in the Hudson formula to compute the design wave height H from the significant wave height H_s. According to the Shore Protection Manual (1984) the design wave height of $H_{1/10} = 1.27*H_s$ may be applied for breakwaters subject to breaking waves.

Research by Van der Meer (1988c) and later by De Jong (1996) on breakwaters with concrete armour units was based on the governing variables found for rock stability. The test programme was limited to only one cross-section (i.e. one slope angle and permeability) for each armour unit. Therefore the slope angle ($\cot\alpha$) and consequently the surf similarity parameter, ξ_m, is not present in most of the stability formulae developed on the results of the research. The same holds for the notional permeability factor, P. This factor was $P = 0.4$.

Breakwaters with armour layers of interlocking units are generally built with steep slopes in the order of 1:1.5. Therefore this slope angle was chosen for tests on Cubes and Tetrapods. Accropode are generally built on a slope of 1:1.33, and this is the slope used for tests with these units. Cubes were chosen as these elements are bulky units which have good resistance against impact forces. Tetrapods are widely used all over the world and have a fair degree of interlocking. Accropodes were chosen as these units were regarded as the latest development at the time of the research programme, showing high interlocking, strong elements and a one layer system. A uniform 1:30 foreshore was applied for all tests. Only for the highest wave heights which were generated, some waves broke due to depth limited conditions.

Damage to concrete units can be described by the damage number N_{od}. N_{od} is the actual number of displaced units related to a width (along the longitudinal axis of the breakwater) of one nominal diameter, D_n.

Stability formulae for the following units are given:

- Stability formulae for Cubes
- Stability formulae for Tetrapods
- Stability formulae for Dolosses
- Stability formulae for Accropodes
- Stability formulae for Core-LocsTM

For some of these units, the stability can be computed with different formulae. For the Cubes and Accropodes, the formulae of Van der Meer (1993) are used. For Tetrapods the formulae by Van der Meer (1999) are used. For the Dolosse units the formula of Holthausen and Zwamborn (1992) is used. The stability of Tetrapods, Dolosse, Cubes and Core-Loc^{TM} units is also calculated with the Hudson formula. For this Hudson formula, the user must specify the K_D and $k_\Delta$values.

These formulae can be solved for one of three options: the damage level resulting from a given storm and armour size, the required armour size given the storm conditions and acceptable damage level or the maximum wave height given the armour size and allowable damage level. These choices are summarized in .

output	required input
N_{od}	H_s and M
M and D_n	H_s and N_{od}
H_s	M and N_{od}

Table 9 Several solving options for stability formulae

Input

Under the 'Formula'-option the type of unit is specified.

[H_s	Incident significant wave height	(m)]
[N_{od}	Number of units displaced in a width one D_n	(-)]
[M	Armour mass	(kg)]
N	Number of waves	(-)
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
s_{om}	Wave steepness with mean period	(-)
s_{op}	Wave steepness with peak period	(-)
cot(α_s)	Slope angle of the structure	(-)
K_D	Stability coefficient	(-)
α_{Hs}	Wave height factor	(-)
w_r	Dolosse waist ratio	(-)

n_t	Number of units in the 'thickness of the armour layer'	(-)
k_Δ	Armour layer coefficient	(-)
n_v	Volumetric porosity of armour layer	(%)

Parameters in the square brackets [...] may be input or output parameters depending upon the definition of the user.

The wave height factor α_{Hs} is used in the Hudson formula to compute the design wave height H from the significant wave height H_s. According to the Shore Protection Manual (1984) the design wave height of $H_{1/10}$ =1.27 H_s may be applied for breakwaters subject to breaking waves. The default value of α_{Hs} should be 1.0.

Output

The results from the Hudson formulae are displayed in the following way, for example for the damage level: 'Hudson:(%D_Hu) Percent damage'.

Stability formulae for Tetrapods

Technical background (Van der Meer)

Van der Meer's formulae for Tetrapods show a dependency on the wave steepness s_{om}. Different formulae apply for surging waves and plunging waves.

For surging waves $s_{om} \leq s_{omc}$:

$$\frac{H_s}{\Delta D_n} = \left[3.75\left(\frac{N_{od}^{0.5}}{N^{0.25}}\right) + 0.85\right] s_{om}^{-0.2}$$

For plunging waves $s_{om} > s_{omc}$:

$$\frac{H_s}{\Delta D_n} = \left[8.6\left(\frac{N_{od}^{0.5}}{N^{0.25}}\right) + 3.94\right] s_{om}^{0.2}$$

the transition from surging to plunging occurs at the critical wave steepness:

$$s_{omc} = \left[\frac{3.75\left(\frac{N_{od}^{0.5}}{N^{0.25}}\right) + 0.85}{8.6\left(\frac{N_{od}^{0.5}}{N^{0.25}}\right) + 3.94}\right]^{1/0.4}$$

Technical background (Hudson)

The technical backrground can be found in Concrete armour units.

Computation of percent damage D (Hudsons formula)

If both wave height and armour size are given as input parameters, the output is the amount of damage expressed as a percentage. The wave height for which no damage will occur ($H_{s,D=0}$) is calculated:

$$H_{s,D=0} = \left[\frac{M K_D \Delta^3 \cot \alpha_s}{\alpha_{H_s}^3 \rho_a} \right]^{1/3}$$

If the input wave height is higher than $H_{s,D=0}$ the amount of damage is calculated based on the coefficients in Table 10.

armour	**relative**	**damage D in percentage**[1]						
type	**wave height**	0 - 5% (5)	5-10% (10)	10-15% (15)	15-20% (20)	20-30% (30)	30-40% (40)	40-50% (50)
Quadripods and Tetrapods	$H_s/H_{s,D=0}$	1.00	1.09	1.17[3]	1.24[3]	1.32[3]	1.41[3]	1.50[3]
Tribar	$H_s/H_{s,D=0}$	1.00	1.11	1.25[3]	1.36[3]	1.50[3]	1.59[3]	<u>1.64</u>[3]
Dolosse	$H_s/H_{s,D=0}$	1.00	1.10	<u>1.14</u>[3]	<u>1.17</u>[3]	<u>1.20</u>[3]	<u>1.24</u>[3]	<u>1.27</u>[3]

[1] all values for breakwater trunk, n=2, randomly placed armour and non-breaking waves
[2] underlined values are interpolated or extrapolated
[3] effects of unit breakage <u>NOT</u> included - actual damage may be significantly higher

Table 10 Relative wave height for various damage levels for some concrete units (SPM, 1984).

Computation of packing density

$$\frac{N_r}{A} = n_t k_\Delta \left(1 - \frac{n_v}{100}\right) \left(\frac{\rho_a}{M}\right)^{2/3}$$

Computation of required armour mass

$$M = \frac{\rho_a \left(\alpha_{H_s} H_s\right)^3}{K_D \Delta^3 \cot \alpha_s}$$

Computation of stability number

$$N_s = \left(\frac{\alpha_{H_s} H_s}{\Delta D_n}\right) = \left(K_D \cot \alpha_s\right)^{1/3}$$

Limits (Van der Meer)

0.01 m	<	H_s	<	20 m
0.5 s	<	T_m	<	30 s
0.005	<	$2\pi H_s/gT^2_p$	<	0.07
0	<	N	<	7500
2000	<	ρ_a	<	3100 kg/m^3

Limits (Hudson)

0.01 m	<	H_s	<	20 m
1926	<	ρ_a	<	2884 kg/m^3
2	<	$\cot\alpha_s$	<	3

Stability formulae for Cubes

Technical background

$$\frac{H_s}{\Delta D_n} = \frac{1}{a}\left\{\left[6.7\frac{N_{od}^{0.4}}{N^3} + 1.0\right] s_{om}^{-0.1}\right\}$$

where a is a correction factor to be applied for the 90 % confidence bands. This formula only applies for slope angles of $\cot(\alpha_s) = 1.5$.

Limits

0.01 m	<	H_s	<	20 m
0.5 s	<	T_m	<	30 s
0.005	<	$2\pi H_s/gT^2_p$	<	0.07
0	<	N	<	7500
2000	<	ρ_a	<	3100 kg/m^3

Stability formulae for Dolosses

The stability of the Dolosse armour unit can be computed with the formula of Holthausen and Zwamborn (1992), also described in Van der Meer (1993).

Technical background

To calculate the damage level the formula by Holthausen and Zwamborn (1992) is used:

$$N_s = \frac{H_s}{\Delta D_n} = \frac{1}{\Delta^{0.26}}\left[\frac{N_{od} \pm 1.645\sigma(E)}{6250 s_{op}^3 w_r^{20 s_{op}^{0.45}}}\right]^{1/5.26}$$

where E is an error term. This term is normally distributed with a mean of 0 and a 90% confidence interval of:

$$1.645 \times \sigma(E) = 0.01936\left[\frac{H_s}{\Delta^{0.74} D_n}\right]^{3.32}$$

Technical background (Hudson)

The technical backrground can be found in Concrete armour units.

Computation of percent damage D (Hudsons formula)

If both wave height and armour size are given as input parameters, the output is the amount of damage expressed as a percentage. The wave height for which no damage will occur ($H_{s,D=0}$) is calculated:

$$H_{s,D=0} = \left[\frac{MK_D \Delta^3 \cot\alpha_s}{\alpha_{H_s}^3 \rho_a}\right]^{1/3}$$

If the input wave height is higher than $H_{s,D=0}$ the amount of damage is calculated based on the coefficients in Table 11.

armour	**relative**	**damage D in percentage[1)]**						
type	**wave height**	0 - 5% (5)	5-10% (10)	10-15% (15)	15-20% (20)	20-30% (30)	30-40% (40)	40-50% (50)
Quadripods and Tetrapods	$H_s/H_{s,D=0}$	1.00	1.09	1.17[3)]	1.24[3)]	1.32[3)]	1.41[3)]	1.50[3)]
Tribar	$H_s/H_{s,D=0}$	1.00	1.11	1.25[3)]	1.36[3)]	1.50[3)]	1.59[3)]	1.64[3)]
Dolosse	$H_s/H_{s,D=0}$	1.00	1.10	1.14[3)]	1.17[3)]	1.20[3)]	1.24[3)]	1.27[3)]

[1)] all values for breakwater trunk, n=2, randomly placed armour and non-breaking waves
[2)] underlined values are interpolated or extrapolated
[3)] effects of unit breakage NOT included - actual damage may be significantly higher

Table 11 Relative wave height for various damage levels for some concrete units (SPM, 1984).

Computation of packing density

$$\frac{N_r}{A} = n_t k_\Delta \left(1 - \frac{n_v}{100}\right)\left(\frac{\rho_a}{M}\right)^{2/3}$$

Computation of required armour mass

$$M = \frac{\rho_a \left(\alpha_{H_s} H_s\right)^3}{K_D \Delta^3 \cot \alpha_s}$$

Computation of stability number

$$N_s = \left(\frac{\alpha_{H_s} H_s}{\Delta D_n}\right) = \left(K_D \cot \alpha_s\right)^{1/3}$$

Limits

0.01 m	<	H_s	<	20 m
8.6 s	<	T_p	<	14 s
0.7	<	$N_s \Delta^{0.26}$	<	4.5
0.33	<	w_r	<	0.4
1800	<	ρ_a	<	3000 kg/m^3

Limits (Hudson)

0.01 m	<	H_s	<	20 m
1926	<	ρ_a	<	2884 kg/m^3
2	<	$\cot\alpha_s$	<	3

Stability formulae for Accropodes

Technical background

According to Van der Meer (1993), the storm duration and wave period have no influence on the stability of Accropodes and the "no damage" and "failure" criterions are very close. The stability can be described by two simple formulae. The "start of damage" criterion ($N_{od} = 0$) is:

$$\frac{H_s}{\Delta D_n} = 3.7$$

Failure occurs when $N_{od} > 0.5$ when:

$$\frac{H_s}{\Delta D_n} = 4.1$$

The above equations show that "start of damage" and "failure" are very close. That means that up to a relatively high wave height Accropodes are completely stable, but after initiation of damage at this wave height, the structure will fail progressively. Therefore a safety coefficient of 1.5 is applied for design purposes, leading to:

$$\frac{H_s}{\Delta D_n} = 2.5$$

This formula is only valid for slope angles of around 1:1.33 and Accropodes applied in 1 layer. If the value of this formula above is higher than 2.5, the output parameter N_{od} is set to an unrealistically high value (>>1000).

Limits

0.01 m	<	H_s	<	20 m
0.5 s	<	T_m	<	30 s
0.005	<	$2\pi H_s / gT^2_p$	<	0.07
0	<	N	<	7500
2000	<	ρ_a	<	3100 kg/m^3

Stability formulae for Core-Locs™

Technical background (Hudson)

The technical background can be found in Concrete armour units.

Input

Under the 'Formula'-option the type of unit is specified.

[H_s	Incident significant wave height	(m)]
[M	Armour mass	(kg)]
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
$\cot(\alpha_s)$	Slope angle of the structure	(-)
K_D	Stability coefficient	(-)
α_{Hs}	Wave height factor	(-)
n_t	Number of units in the 'thickness of the armour layer' (=1 for Core Locs™	(-)
n_v	Volumetric porosity of armour layer	(%)

Parameters in the square brackets [...] may be input or output parameters depending upon the definition of the user.

Computation of percent damage D (Hudsons formula)

If both wave height and armour size are given as input parameters, the output is the amount of damage expressed as a percentage. The wave height for which no damage will occur ($H_{s,D=0}$) is calculated:

$$H_{s,D=0} = \left[\frac{MK_D \Delta^3 \cot \alpha_s}{\alpha_{H_s}^3 \rho_a} \right]^{1/3}$$

If the input wave height is higher than $H_{s,D=0}$ the amount of damage is calculated based on the coefficients in Table 12.

armour	relative	damage D in percentage[1)]						
type	wave height	0 - 5% (5)	5-10% (10)	10-15% (15)	15-20% (20)	20-30% (30)	30-40% (40)	40-50% (50)
Quadripods and Tetrapods	$H_s/H_{s,D=0}$	1.00	1.09	1.17[3)]	1.24[3)]	1.32[3)]	1.41[3)]	1.50[3)]
Tribar	$H_s/H_{s,D=0}$	1.00	1.11	1.25[3)]	1.36[3)]	1.50[3)]	1.59[3)]	<u>1.64</u>[3)]
Dolosse	$H_s/H_{s,D=0}$	1.00	1.10	<u>1.14</u>[3)]	<u>1.17</u>[3)]	<u>1.20</u>[3)]	<u>1.24</u>[3)]	<u>1.27</u>[3)]

[1)] all values for breakwater trunk, n=2, randomly placed armour and non-breaking waves
[2)] underlined values are interpolated or extrapolated
[3)] effects of unit breakage <u>NOT</u> included - actual damage may be significantly higher

Table 12 Relative wave height for various damage levels for some concrete units (SPM, 1984).

Computation of packing density

$$\frac{N_r}{A} = n_t k_\Delta \left(1 - \frac{n_v}{100}\right)\left(\frac{\rho_a}{M}\right)^{2/3}$$

Computation of required armour mass

$$M = \frac{\rho_a \left(\alpha_{H_s} H_s\right)^3}{K_D \Delta^3 \cot\alpha_s}$$

Computation of stability number

$$N_s = \left(\frac{\alpha_{H_s} H_s}{\Delta D_n}\right) = \left(K_D \cot\alpha_s\right)^{1/3}$$

Limits (Hudson)

0.01 m	<	H_s	<	20 m
1926	<	ρ_a	<	2884 kg/m^3
1.33	<	$\cot\alpha_s$	<	2

3.3 Berm breakwaters

A berm breakwater is defined as a dynamically stable profile, which can be roughly classified by the stability parameter $H_s / \Delta D_{n50}$ in the range of 4 to 6 and has a relatively steep seaward slope. The profiles of dynamically stable structures such as gravel/shingle beaches, rock beaches or sand beaches change according to the wave climate. "Dynamically stable" means that the net cross-shore transport is zero and the profile has reached an equilibrium profile for a certain wave condition. It is possible that during each wave, the material is moving up and down the slope (shingle beach).

For dynamically stable structures with profile development a surf similarity parameter cannot be defined as the slope is not straight. Furthermore, dynamically stable structures are described by a large range of $H_s / \Delta D_{n50}$ values. In that case it is possible to relate also the wave period to the nominal diameter and to make a combined wave height - period parameter.

This parameter is defined by:

$$H_0 T_0 = \frac{H_s}{\Delta D_{n50}} T_m \sqrt{\frac{g}{D_{n50}}}$$

The relationship between $H_s / \Delta D_{n50}$ and $H_o T_o$ is listed below

structure	$H_s / \Delta D_{n50}$	$H_o T_o$
Statically stable breakwaters	1 - 4	< 100
Rock slopes and beaches	6 - 20	200 - 1500
Gravel beaches	15 - 500	1000 - 200,000
Sand beaches	> 500	> 200,000

Table 13 Relationship between $H_s / \Delta D_{n50}$ and $H_o T_o$

Another parameter which relates both wave height and period (or wave steepness) to the nominal diameter was introduced by Ahrens (1987). Ahrens included the local wave steepness in a modified stability number N_s^*, defined by:

$$N_s^* = \frac{H_s}{\Delta D_{n50}} s_p^{-1/3}$$

In this equation s_p, is the local wave steepness and not the deep water wave steepness. This modified stability number N_s^* has a close relationship with $H_o T_o$ defined earlier. An overview of possible wave height-period parameters is given below:

$$\frac{H_s}{\Delta D_{n50}} = N_s$$

$$\frac{H_s}{\Delta D_{n50}} s_p^{-1/3} = N_s^*$$

$$\frac{H_s}{\Delta D_{n50}} T_m \sqrt{\frac{g}{D_{n50}}} = H_0 T_0$$

$$\frac{H_s}{\Delta D_{n50}} s_{om}^{-0.5} \sqrt{\frac{2\pi H_s}{D_{n50}}} = H_0 T_0$$

$$\xi_m = \frac{\tan\alpha}{\sqrt{s_{om}}} = \frac{\tan\alpha}{\sqrt{\frac{2\pi H_s}{g T_m^2}}}$$

BREAKWAT computes the dynamically stable profile of berm breakwaters, rock slopes and gravel beaches. It is based on the relationships for a schematised profile, given by Van der Meer (1988) and Van der Meer (1992).

The main basic assumptions are:

- the structure should be "dynamically stable" ($H_s/\Delta D_{n50} > 3$),
- an arbitrary initial slope or profile can be used,
- the crest should be above the Still Water Level (SWL), and
- a sequence of storms can be simulated, including tides.

Besides the design of berm breakwaters, rock and gravel beaches, the program is able to predict the behaviour of core and filter layers under construction during yearly storm conditions.

After an initial profile has been defined, along with the required boundary conditions the profile development of a berm breakwater is computed, along with an estimate of the rate of longshore transport to be expected. The following hydraulic response factors can be calculated for berm breakwaters in BREAKWAT:

- Wave transmission

The following structural response factors can be calculated for berm breakwaters in BREAKWAT:

- Profile development
- Longshore transport
- Rear-side stability

3.3.1 Hydraulic response factors

3.3.1.1 Wave transmission over berm breakwaters

The transmission over a berm breakwater is calulated with the formula of De Jong and d'Angremond (1996), which is also used for conventional rubble mound breakwaters (Wave transmission over RMB). It is advised to calculate the minimal crest height first with Rear-side stability berm breakwater, and then calculate the wave transmission. Very often no transmission is allowed at all for berm breakwaters, because the stability of the rear-side is often rather small.

3.3.2 Structural response factors

3.3.2.1 Profile development

The initial profile(s) and the calculation scenario (wave, water level and structural parameters) must be first specified. A calculation scenario can contain one or more individual storm events, for each of which a deformed profile is computed. The deformed profile from one storm event can be used as the initial (input) profile for a subsequent storm. One or all the profiles can be plotted to visualise the results.

The shape of the dynamically stable profile is given by sets of equations which relate the profile parameters, shown in Figure 23, to the boundary conditions. A set of equations was developed in Van der Meer (1988) for relatively high values of $H_s/\Delta D_{n50} > 10\text{-}20$, and a set for lower values, which gives the transition from completely dynamically stable to almost statically stable structures. Both sets of equations are summarised in Van der Meer (1992). The schematised dynamically stable profile is defined by the 4 parameters listed below and shown in Figure 23:

- The runup length, l_r (m)
- The crest height h_c (m)
- The crest length l_c (m)
- The step height h_s (m).

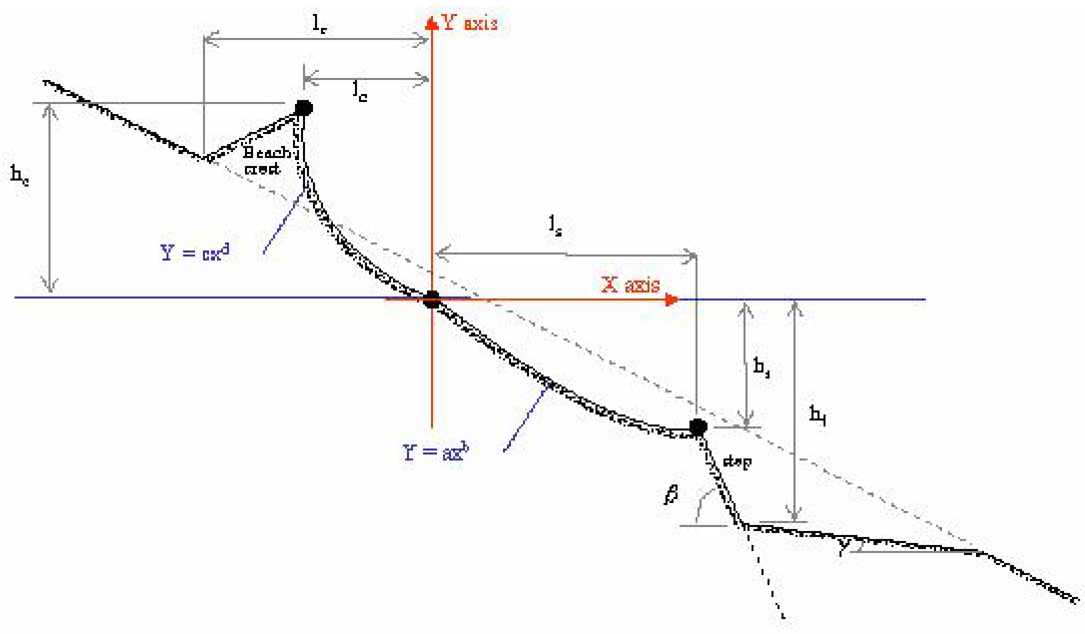


Figure 23 Schematic beach profile - definitions

Input

For a calculation scenario the initial profile(s) (x,y) coordinates for each profile (x ≥ 0; y ≥ 0) are needed. Next to those input parameters for a calculation scenario include the following:

H_s	Significant wave height	(m)
T_m	Mean wave period	(s)
M_{50}	Average mass of rock	(kg)
D_{85}/D_{15}	Grading parameter	(-)
N	Number of waves	(-)
h	Water depth at toe (w.r.t. y = 0)	(m)
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
β	Wave angle (0 = perpendicular)	(deg)
In	Number of the input profile	(-)
Out	Number of the output profile	(-)

The parameter "In" must be defined before a deformation computation can begin. If "Out" is already associated with an initial or computed profile, the user is asked to either define a new "Out" value or confirm that the existing "Out" profile be overwritten.

The input screen for this option differs from the standard input screens. Now only 2 input tables are shown,

- (Profln) Initial profile
- (ProfCalc) Calculation scenarios.

By clicking on the first input table button the initial profile(s) can be entered. The x and y coordinates are entered in columns. More than 1 initial profile can be defined. This way, the underlayers or the core can be visualised. The calculations do not take underlayers into account. This only gives a visual reference of how close the deformed profile will come to an underlayer. When entering the x and y values, the profile is automatically drawn in chart form for easy inspection.

Calculation scenarios are entered in the 2nd input table. Here all the environmental conditions are entered as well as which profile number has to be used as the initial profile and the number of the resulting profile. The resulting profile from one calculation can be used as the initial profile for a subsequent condition. An example is shown in the figure below.

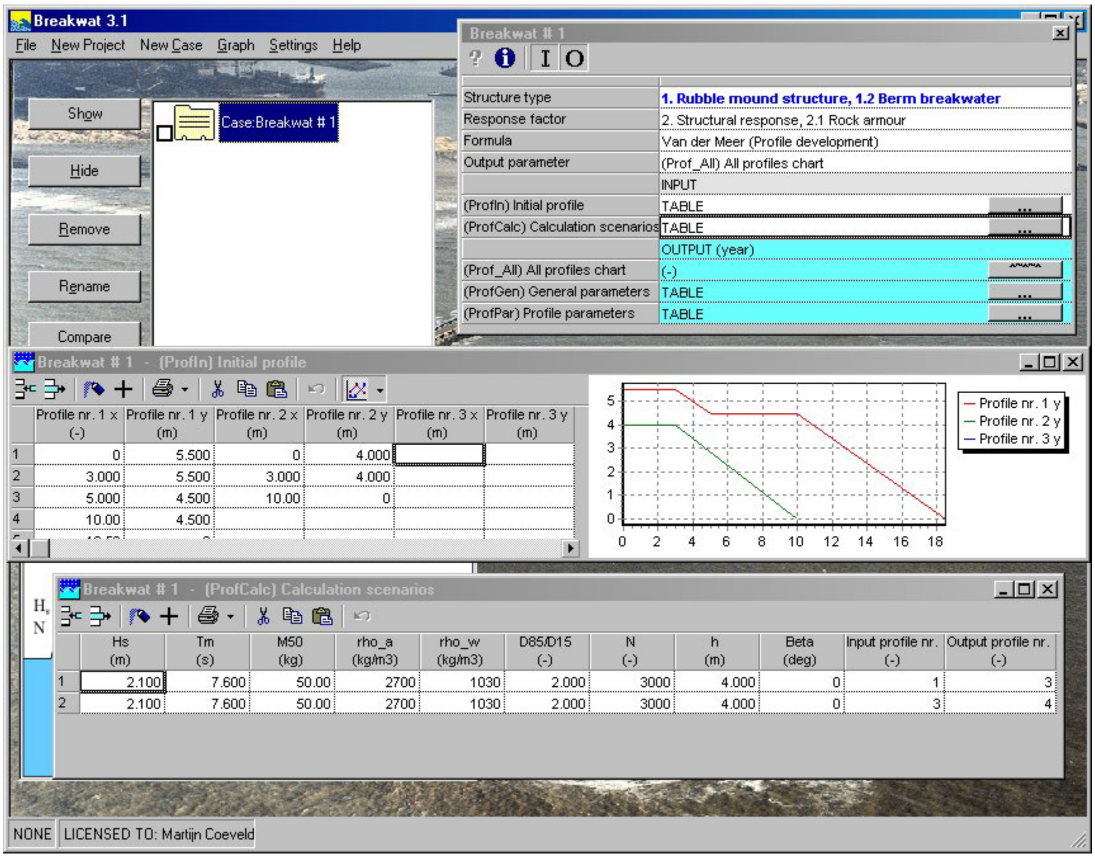


Figure 24 Example input for berm breakwater computations

In this example two initial profiles have been defined as well as two calculation scenario's. Profile number 1 is used as initial profile for the 1st calculation scenario and the resulting profile is Profile 3, which is in turn used as the initial profile for the 2nd calculation scenario, resulting in Profile 4. It is noted that warning messages appear in the window below the output fields (in red text). For the conditions entered, the profiles are close to static stability, making the results of the computations less reliable.

The output consists of 3 fields,

- (Prof_All) All profiles chart
- (ProfGen) General parameters
- (ProfPar) Profile parameters.

An example of these output tables and graphs is shown below.

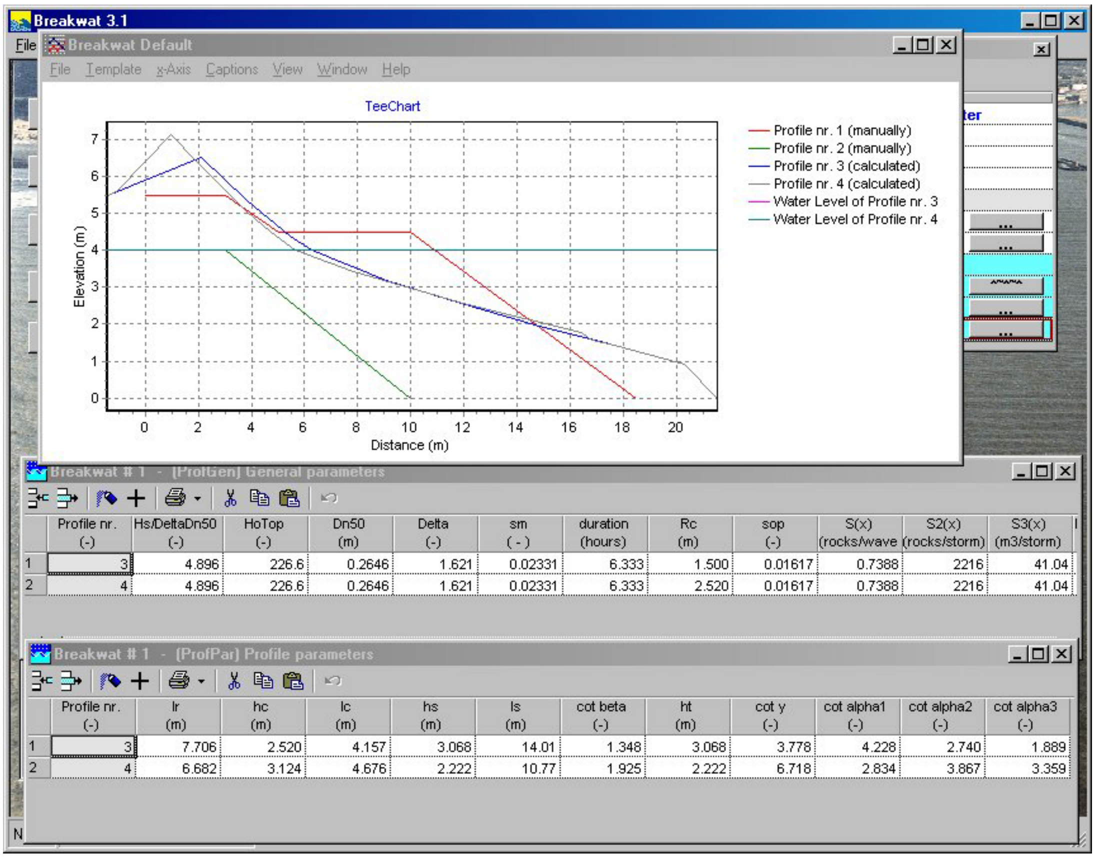


Figure 25 Example output for berm breakwater calculations

The chart shows all defined profiles, manually defined and calculated.

The general parameters include the various stability numbers ($H_s/\Delta D_{n50}$,$HoTo$) etc. as well as the longshore transport of material (see Longshore transport) (rocks/wave, rocks/storm and m3/storm) and the rear-side stability number (see Rear-side stability).

The profile parameters table includes the parameters as defined in Figure 25, which are the parameters which define the shape of the calculated profile.

3.3.2.2 Longshore transport

Statically stable structures such as revetments and breakwaters are only allowed to show damage under very severe wave conditions. Even then, the damage can be described by the displacement of only a number of rocks from the still-water level to (in most cases) a location downwards. Movement of rocks in the direction of the longitudinal axis is not relevant for these types of structures.

The profiles of dynamically stable structures as gravel/shingle beaches, rock beaches or sand beaches, change according to the wave climate. "Dynamically stable" means that the net cross- shore transport is zero and the profile has reached an equilibrium profile for a certain wave condition. It is possible that during each wave some material is moving up and down the slope (shingle beach).

Oblique wave attack gives wave forces parallel to the alignment of the structure. These forces may cause transport of material along the structure. This phenomenon is called longshore transport and is well known for sand beaches. Also shingle beaches change due to longshore transport, although the research on this aspect has been always limited.

Rock beaches and berm breakwaters are or can also be dynamically stable under severe wave action. This means that oblique wave attack may induce longshore transport, which can also cause problems for these types of structures. Longshore transport does not occur for statically stable structures, but it will start for conditions where the diameter is small enough in comparison with the wave height. Then the conditions for start of longshore transport are important.

The start of longshore transport is the most interesting consideration for the berm breakwater where profile development under severe wave attack is allowed. Nevertheless longshore transport should be avoided.

The rate of longshore transport is computed for a developed profile on a berm breakwater. The longshore transport rate S(x) is computed as the number of rocks displaced per incoming wave. This computation is made for each calculation scenario defined in the input.

The total number of displaced rocks in a storm is therefore:

$$S_2(x) = S(x)N$$

and the volume of displaced rocks is:

$$S_3(x) = S(x)ND_{n50}^3$$

For a berm breakwater the longshore transport should be zero.

Technical background

$$H_0T_0 = \frac{H_s}{\Delta D_{n50}} T_m \sqrt{\frac{g}{D_{n50}}}$$

For $H_0T_0 < 105$:

$$S(x) = 0$$

For $H_0T_0 \geq 105$:

$$S(x) = 5 \cdot 10^{-5} \left(H_0 T_0 - 105\right)^2$$

where

$$D_{n50} = \left(\frac{M_{50}}{\rho_a}\right)^{1/3}$$

$$T_p = 1.2 T_m$$

$$\Delta = \frac{\rho_a}{\rho_w} - 1$$

3.3.2.3 Rear side stability berm breakwater

Van der Meer and Veldman (1992) performed extensive test series on two different berm breakwater designs. A first design rule was assessed on the relationship between damage to the rear of a berm breakwater and the crest height, wave height, wave steepness and rock size.

The boundary condition is that that the rock at the crest and rear of the berm breakwater has the same dimensions as at the seaward profile. This means that $H_s/\Delta D_{n50}$ is in the order of 3.0 - 3.5. A further restriction is that the profile at the seaward side has been developed to an S-shape.

The parameter $R_c/H_s{*}s_{op}^{1/3}$ showed to be a good combination of relative crest height and wave steepness to describe the stability of the rear of a berm breakwater. The following values of $R_c/H_s{*}s_{op}^{1/3}$ can be given for various damage levels to the rear of a berm breakwater caused by overtopping waves and can be used for design purposes.

- $R_c/H_s{*}s_{op}^{1/3} = 0.25$:start of damage
- $R_c/H_s{*}s_{op}^{1/3} = 0.21$:intermediate damage
- $R_c/H_s{*}s_{op}^{1/3} = 0.17$:severe damage

In which R_c (m) is the initial crest height above SWL and s_{op} (-) is the wave steepness (= $2\pi H_s / (gT_p^2)$).

Limits on input parameters

0.01 m	<	H_s	<	20 m
0.5 s	<	T_m	<	30 s
0	<	M_{50}	<	40000 kg
1	<	D_{85} / D_{15}	<	2.5
0	<	N	<	9000
0	<	β	<	60 deg
999	<	ρ_w	<	1100 kg/m^3
1900	<	ρ_a	<	4000 kg/m^3
0	<	h		

Limits on derived parameters

0.005	<	s_{om}	<	0.07
2.59	<	$H_s / \Delta D_{n50}$	<	500

3.4 Reef-type structures

A reef breakwater is a low-crested homogeneous pile of rocks without a filter layer or core and is allowed to be reshaped by wave attack (see Figure 26). The initial crest height of a reef breakwater is around the water level. Under severe wave conditions the crest height reshapes to a certain equilibrium crest height. This equilibrium crest height and corresponding transmission are the main design parameters.

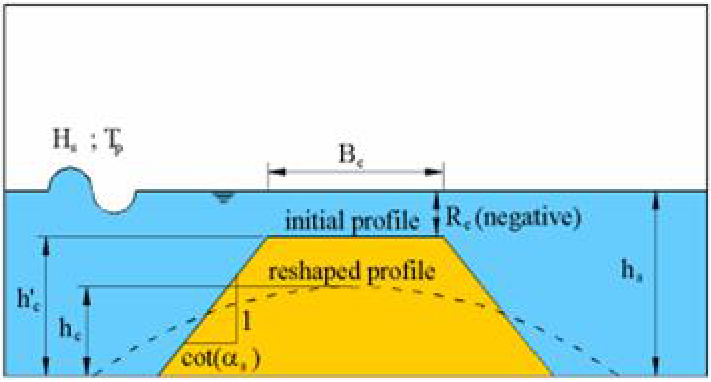


Figure 26 Definition sketch of reef breakwater

The following structural response factor can be calculated for berm breakwaters in BREAKWAT:

- Rock armour stability reef-type structure

3.4.1 Structural response factors

3.4.1.1 Rock armour stability reef-type structure

The stability analyses conducted by Ahrens (1987, 1989) and Van der Meer (1990a) were concentrated on the change in crest height due to wave attack. Ahrens defined a number of dimensionless parameters which described the behaviour of the structure. The main one is the relative crest height reduction factor h_c / h_c'. The crest height reduction factor h_c / h_c' is the ratio of the crest height at the completion of a test to the height at the beginning of the test. The natural limiting values of h_c / h_c' are 1.0 (no deformation) and 0.0 (structure not present anymore) respectively. For the reef breakwater, Ahrens found that a longer wave period caused more displacement of material than a shorter period. Therefore he introduced the spectral stability number, N_s^*, defined by the equations below.

However, it is not always true that a longer wave period should give more damage than a shorter period. Ahrens concluded that this was true for reef breakwaters where the crest height was lowered substantially during the test. However, it is not true for non or marginally overtopped breakwaters (Van der Meer, 1987 and 1988). The influence of the wave period in that case is much more complex than suggested in the equations below.

The (reduced) crest height, according to Van der Meer (1990a) or Van der Meer and Pilarczyk (1990) and taking in account all Ahrens' data, can be described by:

$$h_c = \sqrt{\frac{A_t}{\exp^{aN_s^*}}}$$

with:

$$a = -0.028 + 0.045c' + 0.034\frac{h_c'}{h} - 6\cdot 10^{-9}\left[\frac{A_t^2}{D_{n50}^4}\right]$$

$$N_s^* = \frac{H_s}{\Delta D_{n50}} s_p^{-1/3}$$

This formula requires the shallow water wave steepness, $s_p = H_s/L_p$. Here, L_p is the shallow water wavelength, computed from linear wave theory as:

$$L_p = \frac{gT_p^2}{2\pi}\tanh\left(\frac{2\pi h_s}{L_p}\right)$$

Initial profile

The initial cross section is assumed to have trapezoidal form defined by the crest width B_c, the height h'_c and the side slopes $\cot\alpha$. The cross-section area A_t is calculated by:

$$A_t = h_c'\left(B_c + h_c'\cot\alpha\right)$$

In which the initial height of the cross-section, above the sea bed, is calculated as

$$h_c' = h_s + R_c$$

Input parameters

[H_s	Significant wave height	(m)]
[h_c	Reduced crest height	(m)]
[D_{n50}	Armour size	(m)]
D_{85}/D_{15}	Grading parameter	(-)
T_p	Peak wave period	(s)
B_c	Crest width	(m)
R_c	Crest elevation from MSL	(m)
h_s	Water depth at toe	(m)
$\cot\alpha$	Front and rear structure slopes	(-)
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)

Parameters in square brackets [ ] may be input or output parameters depending upon user-definition

Limits

Concerning the validity of the formulae the following applies: $h_c = h_c'$ if h_c in above equations is larger than h_c'. Next to that:

0	<	h_c / h_c'	<	1
1.5	<	A_t / h'^2_c	<	3.5
200	<	A_t / D^2_{n50}	<	3500
1800	<	ρ_a	<	3000 kg/m^3

3.5 Near-bed structures

Near-bed structures are considered in this section. In particular submerged rubble mound structures with a relatively low crest such that wave breaking due to the structure hardly occurs are considered. In coastal engineering near-bed rubble mound structures are used for instance as pipeline covers or intake- and outfall structures for power-stations and desalination plants.

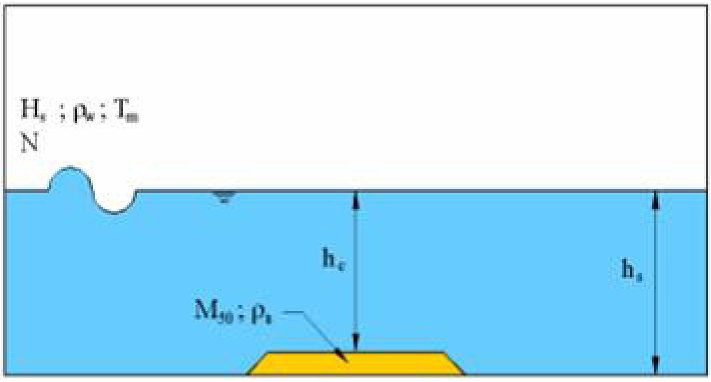


Figure 27 Near-bed structure

The following structural response factor can be calculated for berm breakwaters in BREAKWAT:

- Rock armour stability near-bed structure

3.5.1 Structural response factors

3.5.1.1 Rock armour stability near-bed structure

The stability of near-bed structures under wave loading, with or without a current has been investigated in physical model tests. Based on the analysis of the data several methods to predict the stability of near-bed structures have been analysed. The method found to be the most appropriate is described here. This method was calibrated to relate the erosion of near-bed structures to a mobility parameter. It was found that for low-to-moderate currents in combination with waves, the waves dominate the stability of the rock material; Therefore the stability of the near-bed structures could be predicted without taking the influence of the current. The obtained prediction method accounts for the effects of wave height, wave period, number of waves, stone diameter, rock density and crest elevation. More detailed information can be found in Van Gent and Wallast (2002).

For near-bed structures the amount of damage is best estimated with a method based on a mobility parameter. Based on the present data-set and an analysis of existing data (Lomónaco, 1994) the following formula was obtained:

$$S = 0.2\theta^3 N^{0.5}$$

with

$$\theta = \frac{\hat{u}_\delta^2}{g\Delta D_{n50}}$$

$$\hat{u}_\delta = \frac{\pi H_s}{T_m}\frac{1}{\sinh kh_c}$$

where S is a measure of the amount of damage, N is the number of waves, θ is the mobility parameter, k is the wave number ($2\pi/L$) and $\hat{u}_\delta$ a characteristic velocity.

Input

Input parameters for a calculation scenario include the following:

H_s	Significant wave height	(m)
T_m	Mean wave period	(s)
N	Number of waves	(-)
D_{n50}	Nominal rock size	(m)
h_c	Water depth above crest	(m)
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)

Limits

The conditions for the available data-set with a combination of currents and waves suggest a limited mean velocity of the currents [$u_c / \hat{u}_\delta < 2.2$ for $0.15 < \hat{u}_\delta^2 / (g\Delta D_{n50}) < 3.5$]. Although there are effects of the currents, these effects are small compared to the scatter in the data. However, it is likely that for larger velocities of currents these effects cannot be neglected. Therefore, neglecting the effects of currents for conditions outside the range of the present data-set cannot be justified based on the present analysis. The range of conditions for the present analysis is shown in Table 14 (armour density of 2650 kg/m^3 and water density of 1000 kg/m^3).

parameter (symbol)	range
Slope angle ($\tan\alpha$):	1:8 - 1:1
Crest height (h-h_c):	0.30 - 2.5 m
Crest width (B_c):	0.60 – 2.5 m
Stone diameter (D_{n50}):	0.031 - 0.083 m
Relative density (Δ):	1.45 - 1.7
Number of waves (N):	1000 – 3000
Wave height (H_s):	0.70 – 2.7 m
Wave steepness (s_m):	0.03 - 0.07
Water depth (undisturbed) (h):	3.7 – 9.0 m
Water depth above crest (h_c):	2.4 – 8.7 m
Mean velocity of current (u_c):	0 – 2.34 m/s
Non-dimensional velocity [$u_c^2/(g\ \Delta\ D_{n50})$]:	0 - 10.8
Ratio wave height-water depth (H_s/h):	0.15 - 0.51
Ratio wave height - depth at crest (H_s/h_c):	0.20 - 0.88

Table 14 Parameter ranges for the prediction formula

3.6 Vertical (caisson) breakwaters

For a vertical (caisson) type breakwater many advances have been made in recent years in order to compute the wave overtopping and wave forces on the structure. However, earlier works in Japan are often used as a basis, for comparison with newer studies. Most notable work is that one described in Goda (1985).

For the calculation of pressures and forces on the structure this method assumes that no impulsive wave breaking occurs on the structure. This method cannot be used in such situations and if impulsive breaking may occur a warning message is issued to the user.

For wave overtopping the graphs provided in Goda (1985) are based on the deep water wave height. However, nowadays it is common for the designer to know the wave height at the location of the structure. Van der Meer (2000) has made a review of many research studies on wave overtopping of vertical structures and has suggested a formula which reasonably describes the data sets analysed. This formula has therefore been incorporated.

The following hydraulic response factors for vertical breakwaters can be computed with BREAKWAT:

- Wave overtopping of vertical (caisson) breakwaters
- Wave transmission over vertical (caisson) breakwaters

The following structural response factors for vertical (caisson) breakwaters which can be computed with BREAKWAT are:

- Pressures, forces and safety factors
- Safety factors
- Bearing pressures on foundation
- Toe berm stability of vertical (caisson) breakwater

3.6.1 Hydraulic response factors

3.6.1.1 Wave overtopping of vertical (caisson) breakwaters

In the design of many sea walls and breakwaters, the controlling hydraulic response is often the wave overtopping discharge. Under random waves this varies greatly from one wave to another. For many cases it is sufficient to use the mean discharge, q, usually expressed as a discharge per metre run ($m^3/s/m$). However, safety for pedestrians may be better assessed based on more extreme overtopping events and for this reason also an estimate of the maximum volume of water in a single overtopping wave can be computed.

For many years the work of Goda reported in (Goda, 1985) on wave overtopping for vertical walls was the standard reference for this topic. In that work design graphs are given for the overtopping amount for various configurations of structure type, foreshore slope and wave steepness. Those graphs are based on the deep water wave height and therefore also include the effects of wave transformations over the foreshore. In current design formulae, however, it is assumed that the designer already has information on the wave height in front of the structure, which is commonly the required input. For this reason the design graphs from Goda cannot be applied directly in the current design formula.

The calculation of the overtopping discharge for a particular structure geometry, water level, and wave condition is based on empirical equations fitted to hydraulic model test results. In recent years many research programmes on this topic have been conducted. A brief summary and assessment of the more notable work is given in Van der Meer (2000). The formula applied here is that recommended in the latest version of the TAW guidelines for the design of vertical sea defences (TAW, 2002b).

To derive a design formula the mean discharge rate is generally related to the relative freeboard, R_c/H_s. The general form of the relationship for the overtopping of vertical structures is

$$\frac{q}{\sqrt{gH_s^2}} = A\exp\left[-B\frac{1}{\gamma_\beta\gamma_V}\frac{R_c}{H_s}\right]$$

in which

q	Mean overtopping rate	($m^3/s/m$)
g	Gravitational acceleration	(m/s^2)
H_s	Significant wave height in front of structure	(m)
R_c	Crest freeboard w.r.t. the still water level	(m)
γ_β	Reduction factor for oblique wave incidence	
γ_V	Reduction factor for structure geometry	
A, B	Empirical coefficients	(-)

Analysis of data sets from various researchers by Van der Meer (2000) led to the conclusion that for values of A and B such that

- A = 0.08
- B = 3.0

The formula gives a reasonable representation of the mean trend of the available data sets, especially for conditions with relatively deep water at the structure. A data set from Allsop *et al.* (1995) with shallow water conditions at the structure seems to give much higher overtopping amounts (factor 100 or more!) than the other data sets, so for such conditions the results from this formula should be treated with caution. For design purposes, a safety margin has been included in the formula and so the following values are applied (TAW, 2002b):

- A = 0.13
- B = 3.0

The reduction factors are defined as:

$$\gamma_\beta = 1 \qquad \text{for } \beta < 20^o$$
$$\gamma_\beta = \cos\left(\beta - 20^o\right) \qquad \text{for } 20^o < \beta \leq 90^o$$

$$\gamma_V = 0.7 \text{ to } 1$$

(Depending on the structure form and crest level.)

This formula has been incorporated in BREAKWAT.

Input

Input parameters for a calculation scenario include the following:

H_{si}	Significant wave height at the toe of the structure	(m)
T_m	Mean wave period	(s)
g	Gravitational acceleration	(m/s^2)
R_c	Crest height above MSL	(m)
β	Angle of wave approach	(deg)
ρ_a	Armour density	(kg/m^3)
ρ_w	Water density	(kg/m^3)
γ_{vw}	Freeboard reductio factor	(-)
	= 1 for vertical (caisson) and composite breakwaters without 'nose'	
	= 0.7 for vertical (caisson) breakwaters with 'nose'	

Preliminary indication of probability of overtopping

The following formulae are used to estimate the percentage of overtopping waves and the maximum volume of water in a single overtopping wave.

The percentage of overtopping waves can be computed as

$$p_{ov} = \frac{N_{ov}}{N} \times 100 = \exp\left[-\left(\frac{1}{k}\frac{R_c}{H_{si}}\right)^2\right] \times 100$$

in which

p_{ov}	Percentage of overtopping waves	(%)
N_{ov}	Number of overtopping waves	(-)
N	Number of incident waves	(-)
k	= 0.91 for vertical (caisson) breakwaters	(-)

Maximum volume in a wave

The volume of water contained in individual waves can be considerably higher than the mean overtopping amount. For conditions with relatively deep water at the structure the distribution of volumes in individual waves has been shown to follow a Weibull distribution with shape factor ¾ and scale factor *a*. The scale factor is a function of the mean overtopping rate and the chance of overtopping. **In the same way as in the formula for the mean overtopping, use of these factors for conditions with shallow water at the structure should only be made with discretion, as these formulae have not yet been verified for such conditions. Preliminary results (Allsop 1995) indicate that the amounts for shallow water conditions may be higher than when computed with the formulae here.**

The maximum volume in a single wave, for a given percentage of overtopping and mean overtopping rate is given by

$$V_{\max} = \left(\frac{0.84 T_m \left[q / 1000 \right]}{p_{ov} / 100} \right) \left[\ln \left(N p_{ov} / 100 \right) \right]^{4/3}$$

in which

V_{max}	Maximum volume in overtopping wave per m length	(m^3/m)
T_m	Mean wave period	(s)
q	Mean overtopping discharge rate	(l/s/m)

Limits

0	<	R_c		
0.01 m	<	H_s	<	20 m
0	<	R_c / H_{si}	<	3
0.7	<	γ_β	<	1

3.6.1.2 Wave transmission over vertical (caisson) breakwaters

The transmission of wave energy over a vertical structure can be described by the method of Goda et al. (1967). This formula included two coefficients, whose values dependend on the type of structure. This formula is also used in the wave model SWAN and is described by:

$$C_t = 1 \qquad \text{for } \frac{R_c}{H_{si}} \leq -\alpha - \beta$$

$$C_t = \frac{1}{2}\left(1 - \sin\left(\frac{\pi}{2}\frac{\frac{R_c}{H_{si}} + \beta}{\alpha}\right)\right) \qquad \text{for } -\alpha - \beta \leq \frac{R_c}{H_{si}} \leq \alpha - \beta$$

$$C_t = 0.03 \qquad \text{for } \frac{R_c}{H_{si}} > \alpha - \beta$$

The transmitted wave height is defined as:

$$H_{st} = C_t H_{si}$$

in which

α	Coefficient depending on structure type	(-)
β	Coefficient depending on structure type	(-)
H_{st}	Transmitted significant wave height	(m)
H_{si}	Incident significant wave height	(m)
R_c	Crest freeboard, vertical distance from MSL	(m)

The values of and must given as input. Typical values are defined as:

- vertical (caisson) breakwater: α = 2.2; β = 0.40
- vertical wall (no crest width): α = 1.8; β = 0.10

3.6.2 Structural response factors

3.6.2.1 Design wave height

The design wave height (H_D) is the wave height to be used in the computations and it is dependent on whether the structure is located

- seaward of the surf-zone, or
- within the surf-zone.

Definition of surf-zone:

Wave breaking generally begins in water depths of 2.5 times the deep water wave height. Therefore the beginning of the surf-zone has been defined as the depth at which

$$h_s \leq 2.5 H_{s0}$$

Seaward of the surf-zone: $h_s > 2.5H_{s0}$

$$H_D = 1.8H_{s0}$$

Within the surf-zone:

$H_D = 1.8H_s$ at a distance $5H_{si}$ seawards of the breakwater.

The water depth at a distance $5H_{si}$ seawards of the breakwater is h_{5Hs}:

$$h_{5H_s} = h_s + \left(5H_{si} / \cot\alpha_v\right)$$

The wave height at that depth is estimated from results of a 1-D energy decay computation over a uniform bottom. Results of the incident wave height at the structure (H_{si}) and the design wave height (H_D) are calculated and presented in the list of output parameters.

This method has been included only as a design aid and it is stressed that the results must be viewed as an approximation only. The user is strongly advised to perform a detailed wave climate study to determine the actual wave height in front of the structure and therefor the design wave height. If the results of such a study are available the user can adjust the offshore wave height, H_{s0}, such that the correct incident wave height, H_{si}, is achieved so that the calculations for pressures, forces and safety factors are performed correctly.

3.6.2.2 Pressures, forces and safety factors

Pressures Forces and Safety Factors against sliding and overturning of a vertical caisson are computed following the method described by Goda (1985). It is assumed that the shape of the caisson, with or without a vertical parapet wall on the front side. The purpose of a vertical parapet is to reduce the amount of wave overtopping. When a vertical parapet wall is present, the horizontal force on the caisson will increase but the amount of overtopping water will decrease. To maintain adequate safety factors the width or weight of the caisson will have to be increased, compared to the situation without a wall.

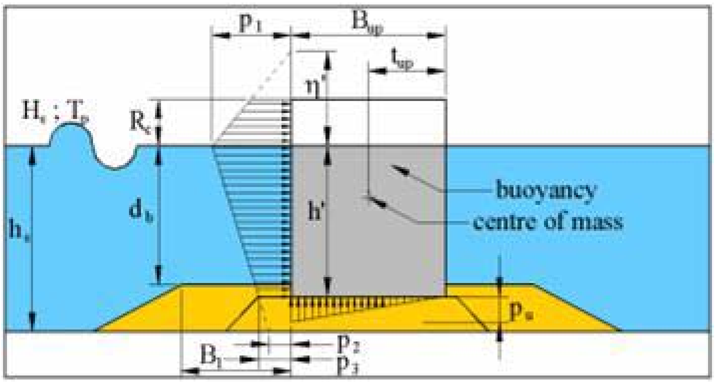


Figure 28 Definition sketch for vertical (caisson) breakwater

The calculation procedure assumes that the wave height offshore of the structure (H_{s0}) is known and a simple procedure is applied to estimate the wave height close to the structure, and therefore defining the design wave height (H_D). If the incident wave height (H_{si}) at the structure location is already known, then the user can adjust the offshore wave height such that the correct H_{si} is obtained. This is described in more detail in Design wave height. For more background on the calculation of horizontal and vertical pressures, see Horizontal and vertical wave pressures and Bearing pressures on foundation. For more background on the calculation of safety factors, see Safety factors.

Input parameters

The following input parameters are required:

H_{s0}	Deep water wave height	(m)
T_{max}	Wave period ($T_{\max} \approx T_{\mathrm{p}} \approx T_{\mathrm{m}}$)	(s)
h_s	Depth of water at the toe of the structure	(m)
B_{up}	Width of upright section	(m)
B_1	Width of toe berm in front of breakwater	(m)
db	Depth of toe berm below SWL	(m)
h'	Distance from SWL to base of caisson	(m)
R_c	Crest freeboard	(m)
t_{up} / B_{up}	Relative horizontal distance between centre of gravity and heel of upright section	(-)
$\cot(\alpha_v)$	Cotangent of the average foreshore slope near the structure	(-)
β	Angle of wave attack with respect to structure	(deg)
μ	Coefficient of friction between upright sectio ans the ground	(-)
ρ_{fill}	Mass density of fill material (sand)	(kg/m^3)
ρ_c	Mass density of concrete cap	(kg/m^3)
ρ_w	Mass density of water	(kg/m^3)

3.6.2.3 Horizontal and vertical wave pressures

In this section the computation of the pressure distribution on the front face and underside of the caisson is described. The method of Goda (1985) is applied. However this method is however, not suitable for situations where impulsive breaking onto the caisson can occur - in those situations the peak wave pressures can be much higher than those computed with the Goda approach. Situations with impulsive breaking may occur for a certain combination of factors. In such cases a warning message is given, stating that impulsive breaking may occur and the results should be treated with caution. In such situations the pressures and forces should be evaluated by other methods.

Technical background

The elevation to which the horizontal wave pressure is exerted (measured from SWL) is

$$\eta^* = 0.75\left(+\cos\beta\right)H_D$$

Wave pressures on the front of vertical wall:

$$p_1 = \frac{1}{2}(1+\cos\beta)\left(\alpha_1 + \alpha_2\cos^2\beta\right)\rho_w H_D g/100$$

$$p_2 = \frac{p_1}{\cosh\left(2\pi h_s / L\right)}$$

$$p_3 = \alpha_3 p_1$$

The α-values in the above equations are:

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi h_s / L}{\sinh\left(4\pi h_s / L\right)}\right]^2$$

$$\alpha_2 = \min\left[\frac{h_{sH_s} - d}{3h_{sH_s}}\left(H_D / d_b\right)^2, \frac{2d_b}{H_D}\right]$$

$$\alpha_3 = 1 - \frac{h^{'}}{h_s}\left[1 - \frac{1}{\cosh\left(2\pi h_s / L\right)}\right]$$

$$L = \left(gT_{1/3}^2 / 2\pi\right)\tanh\left(2\pi h_s / L\right)$$

Uplift pressure:

$$p_u = \frac{1}{2}(1+\cos\beta)\alpha_1\alpha_3\rho_w H_D g/100$$

The total horizontal wave force, F_h, and it's moment, M_h, about the heel of the upright section:

$$F_h = \frac{1}{2}\left(p_1 + p_3\right)h^{'} + \frac{1}{2}\left(p_1 + p_4\right)h_c^*$$

$$M_h = \frac{1}{6}\left(2p_1 + p_3\right)h^{'2} + \frac{1}{2}\left(p_1 + p_4\right)h^{'}h_c^{'} + \frac{1}{6}\left(p_1 + 2p_4\right)h_c^{*2}$$

in which:

$$p_4 = p_1\left(1 - R_c / \eta^*\right) \text{ if } \eta^* > R_c$$
$$p_4 = 0 \qquad \text{if } \eta^* \le R_c$$

$$h_c^* = \min\left[\eta^*, R_c\right]$$

The total uplift force, F_u, and its moment, M_u, about the heel of the upright section:

$$F_u = \frac{1}{2} p_u B_{up}$$

$$M_u = \frac{2}{3} F_u B_{up}$$

The weight of upright section per unit length of breakwater is W_{up}:

$$W_{up} = \left[h'\left(\rho_{fill} - \rho_w\right) + R_c\rho_c\right] B_{up} g / 100$$

in which:

ρ_{fill}	Density of fill material (sand)	(kg/m^3)
ρ_c	Density of concrete cap	(kg/m^3)
ρ_w	Density of water	(kg/m^3)

3.6.2.4 Safety factors

Technical background

Factor of safety against sliding:

$$FS_s = \mu \frac{\left(W_{up} - F_u\right)}{F_h}$$

where the coefficient of friction, μ, commonly has a value of about 0.6.

Factor of safety against overturning:

$$FS_o = \frac{W_{up} t_{up} - M_u}{M_h}$$

in which t_{up} is the horizontal distance between centre of gravity and the heel of the caisson.

3.6.2.5 Bearing pressures on foundation

Technical background

The largest bearing pressure p_e on the foundation is located at the heel of the caisson. Recommended values of this quantity should be below $40 \cdot 10^3$ to $60 \cdot 10^3$ kg/m2 (Goda, 1985).

$$t_e = \frac{M_e}{W_e}$$

$$M_e = W_{up} t_{up} - M_u - M_h$$

$$W_e = W_{up} - F_u$$

$$p_e = \frac{2W_e}{3t_e} \qquad \text{if } t_e \le \frac{1}{3} B_{up}$$

$$p_e = \frac{2W_e}{3t_e}\left(2 - 3\frac{t_e}{B_{up}}\right) \qquad \text{if } t_e > \frac{1}{3} B_{up}$$

Impulsive breaking on vertical (caisson) breakwater

The formulae applied here can be used when no impulsive breaking occurs on the structure. In the following situations impulsive breaking may occur and steps should be taken to avoid such situations. If this is not possible then the wave pressures and forces have to be evaluated by other means. Two conditions are checked for impulsive breaking:

IF	β	<	20 deg
AND	B_1 / L	<	0.02
AND	$\cot a_v$	<	50
AND	H_{s0} / L_{0p}	<	0.03
AND	R_c / H_{si}	>	0.3

IF	β	<	20 deg
AND	$0.02 < B_1 / L$	<	0.3
AND	d_b / h_s	<	0.6
AND	R_c / H_{si}	>	0.3

If one of both conditions or both conditions apply, a warning message is shown.

Limits

0	<	β	<	90 deg
10	<	$\cot\alpha_v$	<	100
0 s		T_{max}	<	30 s
0		t_{up} / B_{up}	<	1
0	<	h_s		
0	<	B_1		
0	<	B_{up}		

3.6.2.6 Toe berm stability of vertical (caisson) breakwater

The toe berm of a vertical breakwater has the function of transmitting the loading to the foundation and to protect the seabed in front of the caisson from scouring. The berm on the rear side can also function providing a buffer area in case the caisson slide backwards. The design formula used in breakwat is based on a formulation by Tanimoto *et al*. (1982), as described in Goda (1985). This formula is based on the assumption that the toe protection consists of rectangular concrete blocks on the horizontal berm in front and at the rear of the upright section, and that the remainder of the berm and slope consists of rock armour. With the formulation by Tanimoto *et al*. (1982) it is possible to calculate the minimum mass of the rock armour. The formulation also assumes perpendicular wave attack.

Input parameters

The following input parameters are required:

[H_{si}	Significant wave height	(m)]
[D_{n50}	Nominal rock size	(m)]
h_t	Depth from SWL to armour (= h')	(m)
h'	Depth from SWL to bottom of upright section	(m)
T_s	Significant wave period	(s)
ρ_a	Mass density of armour	(kg/m^3)
ρ_w	Mass density of water	(kg/m^3)
B_1	Width of toe berm in front of breakwater	(m)
β	Angle of wave attack with respect to structure	(deg)

Parameters in the square brackets [ ] may be input or output parameters.

Technical background

$$M_{50} = \frac{\rho_a}{N_s^3 \Delta^3} H_s^3$$

with:

$$\Delta = \frac{\rho_a}{\rho_w} - 1$$

$$N_s = \max\left[1.8, \left(1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h^{'}}{H_s} + 1.8\exp\left\{-1.5\frac{(1-\kappa)2}{\kappa^{1/3}}\frac{h^{'}}{H_s}\right\}\right)\right]$$

$$\kappa = \frac{4\pi h^{'}/L}{\sinh\left(4\pi h^{'}/L\right)}\sin^2\left(2\pi B_M/L\right)$$

$$L = \frac{gT_p^2}{2\pi}\tanh\left(\frac{2\pi h^{'}}{L}\right)$$

Limits

0	<	N_s
0	<	B_1

3.7 List of symbols

a	coefficient	-
b	coefficient	-
B	crest width	m
B_b	crest width	m
B_M		m
B_{up}	width of upright section of caisson	m
B_1	width of berm in front of caisson	m
c	coefficient	-
c_a	coefficient	-
c_b	coefficient	-
c_0	coefficient	-
c_1	coefficient	-
c_2	coefficient	-
C_r	reflection coefficient	-
C_t	transmission coefficient	-
d	coefficient	-
db	depth of berm for caisson breakwater under SWL	m
d_h	depth of berm of rubble mound breakwater or dike under SWL	m
D_{n15}	diameter $D_{n15} = (M_{15} / \rho_s)^{1/3}$	m
D_{n50}	diameter $D_{n50} = (M_{15} / \rho_s)^{1/3}$	m
D_{n85}	diameter $D_{n85} = (M_{15} / \rho_s)^{1/3}$	m
D_n	nominal diameter	m
Fr	number of Froude, defined as $Fr = u^2 / gL$	-
g	gravitational acceleration = 9.81	m/s^2
h	water depth at the toe of the structure	m
h'	water depth from still water level to elevation where armour is placed	m
h_c	height of crest above still water level	m
h_d	water depth above armour layer of rubble mound foundation	m
h_{min}	minimum height of rubble mound foundation	m
h_s	water depth at toe of structure	m
h_{5Hs}	water depth at a distance 5 H_s from toe of structure seaward	m
H_{m0}	measured total wave height based on the energy desity spectrum as, defined as $H_{m0} = 4\sqrt{m_0}$	m
H_s	significant wave height	m
$H_{s,t}$	transmitted wave height	m
$H_{10\%}$	mean wave height of highest 1/10 fraction of waves	m
$H_{2\%}$	mean wave height of highest 1/50 fraction of waves	m

$H_{1\%}$	mean wave height of highest 1/100 fraction of waves	m
$H_{0.1\%}$	mean wave height of highest 1/1000 fraction of waves	m
k_t	layer thickness coefficient	-
K_d	stability coefficient in Hudson formula	-
K_Δ	armour layer coefficient in Hudson formula	-
L	wave length	m
m_{0i}	area under the incident energy density spectrum	m^2
M_{15}	mass of stones given by 15% on weight exceedance curve	kg
M_{50}	mass of stones given by 50% on weight exceedance curve	kg
M_{85}	mass of stones given by 85% on weight exceedance curve	kg
M_f	mass of foot protection blocks (vertical (caission) breakwater)	kg
n_r	porosity of armour layer	-
n_t	number of armour layers	-
n_v	volumetric porosity	-
N	number of waves	-
N_r	number of armour unit elements	-
N_{of}	number of displaced units in a width of 1 D_{n50}	-
N_s	stability number	-
N_s^*	spectral stability number	-
P	notional permeability in stability formulae	-
q	mean overtopping discharge, per m running length	m^3/s/m
Q_n	dimensionless overtopping rate for non-breaking waves	-
Q_b	dimensionless overtopping rate for breaking waves	-
R_n	dimensionless crest freeboard for non-breaking waves	-
R_b	dimensionless crest freeboard for breaking waves	-
R_c	crest freeboard, wrt. still water level	-
R_t	return period	year
R_u	runup level, wrt. still water level	m
R_{ux}	runup level exceeded by x% of the waves, wrt. still water level	m
s_{om}	wave steepness based on mean wave period	-
s_{op}	wave steepness based on peak wave period	-
S	damage number in stability formulae	-
t_{up}	distance from caisson heel to centre of gravity of caisson	m
T_m	mean wave period	s
$T_{m-1,0}$	spectral wave period based on zeroth en negative first moment	s
T_p	peak wave period	s
u	velocity	m/s
W_{min}	minimum width of foundation	m
α_l	leeward slope angle	deg

α_s	structure sea side slope angle	deg
α_v	foreshore slope angle	deg
β	angle of wave attack, wrt. breakwater orthogonal	deg
γ	total influence factor = γ_β* γ_f* γ_v	-
γ_b	influence factor for a berm in the slope	-
γ_β	influence factor for oblique wave attack	-
γ_f	influence factor for roughness	-
γ_v	influence factor for a vertical wall on top of rubble mound	-
Δ	relative buoyant density, defined as $\Delta = (\rho_s-\rho_w) / \rho_w$	-
ρ_s	density of armour material	kg/m^3
ρ_w	density of water	kg/m^3
ξ_{om}	breaker parameter, based on mean wave period	-
ξ_{op}	breaker parameter, based on peak wave period	-
ξ_{mc}	critical breaker parameter (transition from plunging to surging waves)	-

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