Pocket perched beaches
Computational modelling and calibration in Delft3D

REPORT

Status: FINAL

MSc Thesis
F.J.H. Olijslagers
September 2003
Pocket perched beaches
Computational modelling and calibration in Delft3D

Status: FINAL

MSc Thesis
F.J.H. Olijslagers
September 2003

Thesis committee:
Prof. dr. ir. M.J.F. Stive, Delft University of Technology
Dr. ir. J. van de Graaff, Delft University of Technology
Ir. J.H. de Vroeg, WL | Delft Hydraulics
Ir. A. Mol, Lieverse Consulting Engineers, Breda
Drs. P.J.T. Dankers, Delft University of Technology
Preface

This thesis is the finishing part of the MSc program on the faculty of Civil Engineering and Geosciences at Delft University of Technology. The study took place under the supervision of Prof. Stive of the section of hydraulic engineering. The work is carried out at Lievense Consulting Engineers in Breda and at the department of Marine and Coastal Infrastructure (MCI) of WL | Delft Hydraulics. This report is aimed to scientists, engineers and other people that are interested in this matter.

Special thanks go out to all the thesis committee members in particular, all the employees of Lievense Consulting Engineers for their help, facilities and information on the Third Harbour project, WL | Delft Hydraulics for the facilities, all the people of MCI and MCM of WL | Delft Hydraulics and Dano Roelvink in particular for his support on Delft3D, all the graduating students at WL | Delft Hydraulics for their help and the stimulating environment.

Paul Olijslagers
Delft, September 1, 2003
Summary

This study was initiated by Lievense Consulting Engineers, who were involved in the Third Harbour project in IJmuiden, the Netherlands. A part of the project was the design of a perched beach. Soon, little information on perched beaches seemed to be available and a need for general design rules arose. In literature, most perched beaches behaved stable in 2DV situations, but after construction, often alongshore effects resulted in unexpected erosion. In IJmuiden, secondary effects caused by non-uniformity were expected to take place.

First, an extensive literature study was done. The theory as used is explained, followed by information on pocket beaches and perched beaches. For pocket beaches, different definitions for the bay shape are explained, either between headland breakwaters or between groins. For perched beaches two different options are discerned; the breakwater option and the sill option. Subsequently, different formulations are discussed concerning the location, submergence and width of the submerged structure and the perched beach profile. The advantages go hand in hand with disadvantages of which uncertainty is the main. An extensive inventory of perched beaches listed, most of them with an extensive description of the profile development, either from the mathematical or physical simulation or from the actual prototype beach performance.

For sake of better understanding, a mathematical model is used to simulate the perched beach scheme. The model as used is called Delft3D. Computations are done three-dimensionally, with the use of the FLOW, MOR, and WAVE module, the latter based on SWAN. Especially the simulation of the submerged breakwater asked for a special technique. One of them was the wave transmission over the breakwater and non-erodability of the breakwater during the bottom updating.

After the model set-up, this model is calibrated with the use of a data set from measurements during physical scale model tests for a perched beach in Gibraltar. For these tests, three different wave conditions and water levels are used in different sequences and for different duration. Several hydraulic aspects are calibrated successfully assuming a fixed bottom in the computational model.

Concerning the wave height decay over the breakwater, a different, horizontal bathymetry is used, only for the WAVE computation. First, one imposed transmission coefficient is used to reduce the wave height, but this did not simulate the effect of the crest width. Therefore, nine imposed transmission coefficients are used, i.e. for every grid cell on the breakwater. Though having a good wave height reduction, proper simulation of a transition zone required movement of the imposed transmission coefficient onshore, to delay the initiation of the influence of energy dissipation on the generation of currents. This movement approximated the measured velocities behind the breakwater within the range of acceptation.

The wave height reduction offshore of the breakwater and between the breakwater and the shore is controlled by adaptation of the breaker parameter and the wave friction parameter.

The water level variation offshore over the breakwater, wave set-up, is controlled by a slight adaptation of the imposed transmission coefficients.

After the successful hydraulic calibration, the bottom updating is activated. The first computation approached the measured end bathymetry in broad outlines, but at second sight, there were, sometimes unacceptable, differences. Firstly, although the correlation between wave height and transport rate did agree, the calculated transport was directed onshore where as the measured transport was directed offshore. Adjustment of the horizontal eddy viscosity and diffusivity terms eventually resulted in an offshore directed transport. Secondly, the erosion around the waterline, and especially the erosion of dry cells had to be fine-tuned by a parameter called THETSD. Thirdly, the erosion between the breakwater and the shore was determined by spurious circulation cells, forced by radiation stress gradients. Future versions of Delft3D will handle the forcing of currents by waves in a different way, but for this moment, this problem seemed inevitable.

Fourthly, during a validation test, the mass balance seemed to be inconsistent. During the simulation, an artificial production of mass, linear in time and relative to the wave height, took place. The production rate was as large as the transport rate over the breakwater and therefore not negligible. Several parameters and tunings are investigated on the influence of sand production, but none with satisfying result.
Although the model could not be calibrated on direct measurements of the sediment transport, a sensitivity analysis is done on numerical and empirical parameters. Except for calibration parameters, the influence of variations of numerical parameters was little.

Initially, the goal was to investigate the influence of non-uniform boundary conditions and non-uniform geometry on pocket perched beaches with the use of the calibrated model. However, since considerably more effort and time was required to calibrate the model in a cross-shore direction than anticipated, it was decided to stop after this calibration phase.

The conclusions as drawn concerning literature is that most of the perched beaches appeared to be unstable due to alongshore losses, in contrast to the expectations based on models. Creation of a pocket beach lends itself perfect to stabilise the beach in alongshore direction. Concerning the calibration process it can be said that the wave height reduction, wave set-up and velocities over the submerged breakwater can be simulated well using Delft3D. Concerning the sediment transports, one faces some problems that have to be untangled. The problems to untangle are summarised in the recommendations. The forcing of wave-driven currents in Delft3D has to be related to a certain wave height to depth ratio, as will be adapted in the future. The cause of the production of sand has to be revealed and if possible adjusted in the source code. Concerning the perched beach concept, further investigation is desirable. The perched beach in IJmuiden lends itself perfectly for continuation of investigation.
# Table of contents

**PREFACE** .......................................................................................................................... I  
**SUMMARY** .......................................................................................................................... II  
**TABLE OF CONTENTS** .......................................................................................................... IV  
**LIST OF FIGURES** ................................................................................................................ VII  
**LIST OF TABLES** .................................................................................................................. XI  
**NOTATION** .......................................................................................................................... XII  

## 1 INTRODUCTION  .................................................................................................................. 1

1.1 **GENERAL INTRODUCTION** ...................................................................................... 1  
1.2 **PROBLEM ANALYSIS** ............................................................................................... 1  
  1.2.1 **Problem description** ........................................................................................... 1  
  1.2.2 **Problem definition** ............................................................................................. 3  
  1.2.3 **Aim of the study** ................................................................................................ 3  
  1.2.4 **Problem approach** ............................................................................................. 3  
1.3 **LAY-OUT OF THE REPORT** ........................................................................................ 3  

## 2 LITERATURE STUDY  ......................................................................................................... 4

2.1 **INTRODUCTION** ......................................................................................................... 4  
2.2 **WAVES AND BEACHES** ............................................................................................. 4  
  2.2.1 **Wave transmission** ............................................................................................ 4  
  2.2.2 **Transition zone** .................................................................................................. 6  
  2.2.3 **Closure depth** ..................................................................................................... 8  
  2.2.4 **Wave set-up** ....................................................................................................... 8  
  2.2.5 **Beach profiles** .................................................................................................... 9  
2.3 **POCKET BEACHES** ..................................................................................................... 13  
  2.3.1 **Introduction** ......................................................................................................... 13  
  2.3.2 **Parabolic bay shape between headland breakwaters** ........................................ 14  
  2.3.3 **Equilibrium beach shape between groins** ....................................................... 17  
2.4 **PERCHED BEACHES** .................................................................................................. 18  
  2.4.1 **Perched beach concept** ...................................................................................... 18  
  2.4.2 **Theory involving perched beaches** .................................................................... 18  
  2.4.3 **Advantages and disadvantages of the perched beach concept** ...................... 23  
  2.4.4 **Inventory of perched beaches** ........................................................................... 24  

## 3 DESCRIPTION OF THE MODELLING SYSTEM ................................................................ 28

3.1 **INTRODUCTION** ......................................................................................................... 28  
3.2 **DELFT3D-MOR** ......................................................................................................... 29  
3.3 **DELFT3D-WAVE** ...................................................................................................... 30  
3.4 **DELFT3D-FLOW** ....................................................................................................... 30  
3.5 **SEDIMENT ONLINE** .................................................................................................. 31  
3.6 **MODELLING OF A SUBMERGED BREAKWATER IN DELFT3D** ............................... 32  
  3.6.1 **Introduction** ......................................................................................................... 32  
  3.6.2 **SWAN** ................................................................................................................ 32  
  3.6.3 **FLOW and Sediment Online** ............................................................................. 36
# Table of contents

4 MODELL CALIBRATION ............................................................................................................. 37

4.1 INTRODUCTION .................................................................................................................. 37
4.2 RESULTS OF THE PHYSICAL TESTS .................................................................................. 37
4.3 MATHEMATICAL MODEL SET-UP ...................................................................................... 41
    4.3.1 Delft3D set-up ............................................................................................................. 41
    4.3.2 Overall set-up .............................................................................................................. 43
4.4 MODELL CALIBRATION ....................................................................................................... 44
    4.4.1 Wave transmission ....................................................................................................... 45
    4.4.2 Wave set-up ................................................................................................................ 50
    4.4.3 Remaining wave propagation ..................................................................................... 51
    4.4.4 Transition zone and velocities .................................................................................... 52
    4.4.5 Sediment transport ..................................................................................................... 56
4.5 MODELL VALIDITY CHECKS .............................................................................................. 63
    4.5.1 Check on negative mass values .................................................................................. 63
    4.5.2 Check on the conservation of mass .......................................................................... 63
    4.5.3 Check on the exhaustion of available sediment ....................................................... 63
    4.5.4 Check on uniformity in alongshore direction ........................................................... 63
    4.5.5 Deviation between depths and wave heights ............................................................ 66
4.6 DIFFERENCES BETWEEN THE MATHEMATICAL AND PHYSICAL MODEL ................. 68

5 SENSITIVITY ANALYSIS ....................................................................................................... 69

5.1 INTRODUCTION .................................................................................................................. 69
5.2 NUMERICAL ASPECTS ....................................................................................................... 69
    5.2.1 Time step .................................................................................................................... 69
    5.2.2 Grid size ..................................................................................................................... 70
    5.2.3 Number of iterations ................................................................................................. 70
    5.2.4 Influence of the morphological time factor ............................................................... 71
    5.2.5 Transition between two wave conditions .................................................................. 71
5.3 EMPIRICAL PARAMETERS ................................................................................................. 76
    5.3.1 Viscosity and diffusivity parameters ........................................................................ 76
    5.3.2 Influence of bottom roughness on sediment transport ............................................ 81
    5.3.3 Sediment transport parameters .............................................................................. 81
5.4 HYDRAULIC PARAMETERS .............................................................................................. 82
    5.4.1 Spectral resolution .................................................................................................... 82
    5.4.2 Directional spreading ............................................................................................... 82

6 CONCLUSIONS AND RECOMMENDATIONS ........................................................................ 84

6.1 CONCLUSIONS ................................................................................................................... 84
    6.1.1 Conclusions on the literature study ......................................................................... 84
    6.1.2 Conclusions on the calibration process and Delft3D ................................................ 84
    6.1.3 Conclusions on the investigation of influences of non-uniformities ....................... 85
6.2 RECOMMENDATIONS ......................................................................................................... 86

REFERENCES ................................................................................................................................ 87

APPENDIX A. LOCATION OF THE PROJECT AREA

APPENDIX B. NON UNIFORMITY IN IJMUIDEN

APPENDIX C. BEACH SHAPE AROUND GROINS

APPENDIX D. PHYSICAL STUDIES INVOLVING PERCHED BEACHES

APPENDIX E. MATHEMATICAL STUDIES INVOLVING PERCHED BEACHES
APPENDIX F. PERFORMANCE OF PROTOTYPE PERCHED BEACHES
APPENDIX G. DELFT3D-WAVE
APPENDIX H. DELFT3D-FLOW
APPENDIX I. SEDIMENT ONLINE TRANSPORT FORMULAS
APPENDIX J. MEASURED WAVE HEIGHTS
APPENDIX K. COMPARISON OF TRANSMISSION MODELS
APPENDIX L. MEASURED WATER LEVELS
APPENDIX M. MEASURED VELOCITIES
APPENDIX N. BATHYMETRY
APPENDIX O. INITIAL FLOW INPUT PARAMETERS
APPENDIX P. MORPHOLOGY INPUT FILE PARAMETERS
APPENDIX Q. SEDIMENT INPUT FILE PARAMETERS
APPENDIX R. INITIAL WAVE INPUT PARAMETERS
APPENDIX S. INPUT SCRIPTS
APPENDIX T. DETERMINATION OF THE TRANSMISSION COEFFICIENTS (1)
APPENDIX U. DETERMINATION OF THE TRANSMISSION COEFFICIENTS (2)
APPENDIX V. DETERMINATION OF THE TRANSMISSION COEFFICIENTS (3)
APPENDIX W. DETERMINATION OF THE TRANSMISSION COEFFICIENTS (4)
APPENDIX X. DETERMINATION OF THE TRANSMISSION COEFFICIENTS (5)
APPENDIX Y. DETERMINATION OF THE TRANSMISSION COEFFICIENTS (6)
APPENDIX Z. CALCULATION OF TRANSPORT RATES
APPENDIX AA. CHECK ON CONSERVATION OF SAND MASS
APPENDIX BB. MASS BALANCE FOR DIFFERENT CALCULATIONS
APPENDIX CC. INFLUENCE OF PARAMETERS ON MASS PRODUCTION
APPENDIX DD. EROSION AND TRANSPORTS
APPENDIX EE. INFLUENCE OF BOTTOM ROUGHNESS
APPENDIX FF. ENCOUNTERED PROBLEMS
## List of figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>Perched beach principle, cross-section (Sorensen, 1987)</td>
</tr>
<tr>
<td>1-2</td>
<td>Former recreational beach versus new Third Harbour and perched beach</td>
</tr>
<tr>
<td>1-3</td>
<td>Perched beach layout</td>
</tr>
<tr>
<td>1-4</td>
<td>Cross-shore scheme IJmuiden perched beach</td>
</tr>
<tr>
<td>2-1</td>
<td>Definition sketch of the transmission coefficient $K_t$</td>
</tr>
<tr>
<td>2-2</td>
<td>Surf zone division (Svendsen, 1984)</td>
</tr>
<tr>
<td>2-3</td>
<td>Predicted and measured wave height decay, set-up and transition zone (Nairn et al., 1992)</td>
</tr>
<tr>
<td>2-4</td>
<td>Cross-shore definition sketch closure depth</td>
</tr>
<tr>
<td>2-5</td>
<td>Example of wave set-up, wave height decay and the balance of momentum at the breakwater</td>
</tr>
<tr>
<td>2-6</td>
<td>A-values versus $H_b/W_sT$ (Dean, 1986)</td>
</tr>
<tr>
<td>2-7</td>
<td>Natural pocket beach between headlands, Long Beach, Australia (source: internet)</td>
</tr>
<tr>
<td>2-8</td>
<td>Artificial pocket perched beach between groins, Kemigawa beach (Van Rijn, 1993)</td>
</tr>
<tr>
<td>2-9</td>
<td>Typical beach shape of pocket beaches between headlands at Isla Vieques (source: internet)</td>
</tr>
<tr>
<td>2-10</td>
<td>Definition sketch pocket beach (Berenguer and Enriquez, 1988)</td>
</tr>
<tr>
<td>2-11</td>
<td>Definition sketch of and coefficient used in the parabolic equation (Hsu and Evans, 1989)</td>
</tr>
<tr>
<td>2-12</td>
<td>Definition sketch of and coefficient used in the parabolic equation</td>
</tr>
<tr>
<td>2-13</td>
<td>Different sill options</td>
</tr>
<tr>
<td>2-14</td>
<td>Definition sketch for rock sill and associated sand beach of mild slope (Dean, 1988)</td>
</tr>
<tr>
<td>2-15</td>
<td>Definition sketch of three regions according Gonzalez et al. (1999)</td>
</tr>
<tr>
<td>2-16</td>
<td>Definition sketch of protected and non-protected profiles (Muñoz-Pérez et al., 1999)</td>
</tr>
<tr>
<td>2-17</td>
<td>Comparison between measurement and expression for protected and non-protected profiles (Muñoz-Pérez et al., 1999)</td>
</tr>
<tr>
<td>2-18</td>
<td>Comparison for best fitted values of $A_p$ and predicted values of $A_p$ using Equation (2.59) (Muñoz-Pérez et al., 1999)</td>
</tr>
<tr>
<td>2-19</td>
<td>Nourishment areas for beach elevation over the entire scheme and with a perched beach</td>
</tr>
<tr>
<td>2-20</td>
<td>Perched beach on a Caribbean island (source: internet)</td>
</tr>
<tr>
<td>3-1</td>
<td>Concept of morphological process based models</td>
</tr>
<tr>
<td>3-2</td>
<td>Delft3D system architecture</td>
</tr>
<tr>
<td>3-3</td>
<td>Overview of the used Delft3D components</td>
</tr>
<tr>
<td>3-4</td>
<td>The Delft3D staggered grid</td>
</tr>
<tr>
<td>3-5</td>
<td>Horizontal and sloping SWAN bathymetries</td>
</tr>
<tr>
<td>3-6</td>
<td>Breakwater cross-section with FLOW and SWAN bathymetry</td>
</tr>
<tr>
<td>3-7</td>
<td>FLOW and SWAN computational and bottom grids in top view perspective</td>
</tr>
<tr>
<td>3-8</td>
<td>Wave transmission with one sheet</td>
</tr>
<tr>
<td>3-9</td>
<td>Wave transmission with nine sheets</td>
</tr>
<tr>
<td>3-10</td>
<td>Breakwater cross-section depth (m) and available sediment (kg/m²)</td>
</tr>
<tr>
<td>4-1</td>
<td>Location of the gauges</td>
</tr>
<tr>
<td>4-2</td>
<td>Adjustment of the water level boundary of wave condition 3</td>
</tr>
<tr>
<td>4-3</td>
<td>Location of the velocity gauges</td>
</tr>
<tr>
<td>4-4</td>
<td>Initial and end bathymetry of the physical tests</td>
</tr>
<tr>
<td>4-5</td>
<td>FLOW grid and cross-shore bathymetry</td>
</tr>
<tr>
<td>4-6</td>
<td>SWAN and FLOW bottom- and computational grids</td>
</tr>
<tr>
<td>4-7</td>
<td>Two computational cycles</td>
</tr>
<tr>
<td>4-8</td>
<td>Directory structure</td>
</tr>
<tr>
<td>4-9</td>
<td>Wave height and water level in meters per successive WAVE and FLOW (WF) computation at location 2, 3 and 4</td>
</tr>
<tr>
<td>4-10</td>
<td>Wave transmission for all conditions, with equal bathymetry for FLOW and SWAN</td>
</tr>
<tr>
<td>4-11</td>
<td>Wave transmission for all conditions, using one sheet</td>
</tr>
<tr>
<td>4-12</td>
<td>Wave transmission for all conditions, using nine sheets, values according Goda</td>
</tr>
<tr>
<td>4-13</td>
<td>Wave transmission for all conditions, using nine adapted sheet values</td>
</tr>
<tr>
<td>4-14</td>
<td>Transmission coefficients with wave transmission and right water levels</td>
</tr>
<tr>
<td>4-15</td>
<td>Location of the gauges, initial and end bathymetry</td>
</tr>
<tr>
<td>4-16</td>
<td>Wave height decay with and without onshore movement of sheets, two grid cells onshore</td>
</tr>
<tr>
<td>4-17</td>
<td>Location of the velocity gauges</td>
</tr>
<tr>
<td>4-18</td>
<td>Cross-sectional velocity field</td>
</tr>
<tr>
<td>4-19</td>
<td>Accordance of signs of wave forcing term and velocity in the top layer</td>
</tr>
<tr>
<td>Figure A 1</td>
<td>Location of the Third Harbour project area</td>
</tr>
<tr>
<td>Figure A 2</td>
<td>Ijmuiden Third Harbour design layout</td>
</tr>
<tr>
<td>Figure A 3</td>
<td>Ijmuiden Third Harbour and pocket perched beach during construction, March 2003</td>
</tr>
<tr>
<td>Figure A 4</td>
<td>Ijmuiden perched beach, detailed layout</td>
</tr>
<tr>
<td>Figure A 5</td>
<td>Ijmuiden perched beach cross-section D-D</td>
</tr>
<tr>
<td>Figure A 6</td>
<td>Ijmuiden pocket perched beach, beach profile, March 2003</td>
</tr>
<tr>
<td>Figure A 7</td>
<td>Ijmuiden perched beach with recreational houses, May 2003</td>
</tr>
<tr>
<td>Figure A 8</td>
<td>Wave approach, difference in wave height and direction</td>
</tr>
<tr>
<td>Figure A 9</td>
<td>Alongshore cross-section, difference in set-up and driving forces</td>
</tr>
<tr>
<td>Figure A 10</td>
<td>Alongshore velocity components</td>
</tr>
<tr>
<td>Figure A 11</td>
<td>Coastline development around a groin</td>
</tr>
<tr>
<td>Figure A 12</td>
<td>Lido di Dante nourishment profile</td>
</tr>
<tr>
<td>Figure A 13</td>
<td>Typical sand bags breakwater (1m³)</td>
</tr>
<tr>
<td>Figure A 14</td>
<td>Profile development physical tests</td>
</tr>
</tbody>
</table>
List of figures

Figure A 15 Profile development
Figure A 16 Lido di Ostia physical test
Figure A 17 Lido di Ostia Crostran results
Figure A 18 Lido di Ostia design scheme
Figure A 19 Recommended scheme for Riccione
Figure A 20 Recommended scheme for Cesenatico
Figure A 21 Wave flume initial layout
Figure A 22 Profile development for the physical tests with the submerged breakwater
Figure A 23 Comparison between initial profile and developed profile with and without the breakwater
Figure A 24 Flemish North Sea coast near Knokke (source: internet)
Figure A 25 Investigated beach suppletion profiles (source: internet)
Figure A 26 Resulting volume balance development (source: internet)
Figure A 27 Cumulative sand balances (source: internet)
Figure A 28 Breakwater cross-section to be evaluated
Figure A 29 Temporary best beach profile, Eastern beach
Figure A 30 Final recommended beach profile, Eastern beach (mirror image)
Figure A 31 Final recommended cross-section of the Eastern beach breakwater
Figure A 32 Breakwater cross-section to be evaluated for the Rubble tip area
Figure A 33 Recommended beach profile, Rubble tip area
Figure A 34 Final recommended beach profile, Rubble tip area (mirror image)
Figure A 35 Final recommended cross-section of the Rubble tip area breakwater
Figure A 36 Preferred perched beach scheme
Figure A 37 Profile development of the recommended nourishment scheme
Figure A 38 Layout and cross-section of Marina East perched pocket beach
Figure A 39 Cross-section view of Marina East perched pocket beach
Figure A 40 Equilibrium position of the perched beach with the use of 220 μm sand
Figure A 41 Cap Cod design scheme
Figure A 42 Profile development Cape Cod
Figure A 43 Typical Emilio Romagna coast cross-section
Figure A 44 Profile development Emilio Romagna
Figure A 45 Typical beach protection system
Figure A 46 Pellestrina beach protection works
Figure A 47 Beach profile of Keino-Matsubara-Beach, just after construction of the submerged breakwater
Figure A 48 Initial profile Lido di Ostia perched beach
Figure A 49 Lido di Ostia profile development after three years (Ferrante et al., 1992)
Figure A 50 Lido di Ostia profile development, 1989-1991-1995 (Tomasicchio, 1996)
Figure A 51 Staggered grid
Figure A 52 Vertical distribution of turbulent kinetic energy production (WL Delft Hydraulics, 2001-a)
Figure A 53 Selection of kmx layer; where ‘a’ is Van Rijn’s reference height (WL Delft Hydraulics, 2001-a)
Figure A 54 Sediment balance in a computational cell
Figure A 55 Location of the water level gauges
Figure A 56 Schematised breakwater profile, sloping seaward side without a berm.
Figure A 57 Transmission versus crest with, measured versus approaching models (H_s=1.5m)
Figure A 58 Transmission versus crest with, measured versus approaching models (H_s=2.5m)
Figure A 59 Transmission versus crest with, measured versus approaching models (H_s=3.5m)
Figure A 60 Location of the velocity gauge
Figure A 61 Grid and bathymetry
Figure A 62 Initial bathymetry around the breakwater for both the physical and the mathematical model
Figure A 63 Bathymetry and Manning coefficient in cross-shore direction
Figure A 64 Wave condition 1: Hsig = 1.5 m
Figure A 65 Wave condition 2: Hsig = 2.5 m
Figure A 66 Wave condition 3: Hsig = 3.5 m
Figure A 67 Wave condition 1: Hsig = 1.5 m
Figure A 68 Wave condition 2: Hsig = 2.5 m
Figure A 69 Wave condition 3: Hsig = 3.5 m
Figure A 70 Wave condition 1: Hsig = 1.5 m
Figure A 71 Wave condition 2: $H_{sig} = 2.5$ m
Figure A 72 Wave condition 3: $H_{sig} = 3.5$ m
Figure A 73 Wave condition 1: $H_{sig} = 1.5$ m
Figure A 74 Wave condition 2: $H_{sig} = 2.5$ m
Figure A 75 Wave condition 3: $H_{sig} = 3.5$ m
Figure A 76 Wave condition 1: $H_{sig} = 1.5$ m
Figure A 77 Wave condition 2: $H_{sig} = 2.5$ m
Figure A 78 Wave condition 3: $H_{sig} = 3.5$ m
Figure A 79 Wave condition 1: $H_{sig} = 1.5$ m
Figure A 80 Suspended and bed-load transport in u-direction above the breakwater
Figure A 81 Available sediment in cross-shore direction at the beginning and end of a simulation
Figure A 82 Initial and end bathymetry
Figure A 83 Depth layers, velocity, concentration and advection transport above the breakwater
Figure A 84 Location of the monitoring point on the grid and the breakwater cross-section
Figure A 85 Location of the cross-section on the grid and breakwater cross-section
Figure A 86 Transport paradox: offshore transport and bed level rise
Figure A 87 Calculation of cross-shore area for mass conservation check
Figure A 88 Available sediment at $t=0$ and $t_{end}$
Figure A 89 Increase of the mass balance, $dt=6$ seconds
Figure A 90 Increase of the mass balance, $dt=3$ seconds
Figure A 91 Increase of the mass balance, $dt=1.5$ seconds
Figure A 92 Increase of the mass balance, $dt=3$ seconds, morfac=1
Figure A 93 THETSD = 0.0
Figure A 94 THETSD = 0.4
Figure A 95 THETSD = 0.8
Figure A 96 THETSD=1.0
Figure A 97 Reference run
Figure A 98 BED=0.0
Figure A 99 SUS=0.0
Figure A 100 BEDW=0.0
Figure A 101 SUSW=1.0
Figure A 102 FWFAC=0.0
Figure A 103 FWFAC=0.5
Figure A 104 Simulation with excluded fixed layer or erodible breakwater
Figure A 105 Run001, $dt=6$ sec, morfac=6, THETSD=0.8
Figure A 106 Run0014, $dt=6$ sec, morfac=6, THETSD=0.0
Figure A 107 Run0015, $dt=6$ sec, morfac=6, THETSD=0.4
Figure A 108 Run0018, THETSD=0.4 $dt=1.5$-3.0-6.0 sec, hed=50
Figure A 109 Run0018, THETSD=1.0 $dt=1.5$-3.0-6.0 sec, hed=30
Figure A 110 Run0018. THETSD=0.8
Figure A 111 Run0018, THETSD=1.0, hed=60 instead of 10
Figure A 112 Run002, calculated and measured offshore losses and end bathymetries versus the initial bathymetry, $d=3$ sec., THETSD=0.8
Figure A 113 Run0021, calculated and measured offshore losses and end bathymetries versus the initial bathymetry, $d=3$ sec, sheets moved 5 meter onshore, THETSD=1.0
Figure A 114 Run0025, morfac=1 instead of 6,
Figure A 115 Run0026, morfac=1 instead of 6,
Figure A 116 Run003, $dt=1.5$ sec, morfac=6
Figure A 117 Run half001, THETSD=0.8, $dt=3$ sec.
Figure A 118 Run half002, THETSD=0.8, $dt=1.5$ sec
Figure A 119 Reference height versus Manning coefficient
Figure A 120 Relative layer thickness, velocity, concentration and advection transport
Figure A 121 Influence of bottom roughness on transport rates above the breakwater crest
Figure A 122 Depth points for succeeding time steps after interpolation
Figure A 123 Depth and available sediment of two succeeding tests, illustrating example
Figure A 124 Breakwater initial depth and available sediment at the beginning of the first and last run
List of tables

Table 2-1  Studies involving perched beaches......................................................................................... 25
Table 2-2  Inventory of perched beaches................................................................................................ 26
Table 4-1  Test sequence and wave condition and duration per test........................................................ 37
Table 4-2  Measured wave heights and periods....................................................................................... 38
Table 4-3  Water levels measured in the wave flume ......................................................................... 38
Table 4-4  Adjusted water level for in the mathematical study .............................................................. 39
Table 4-5  Measured velocities............................................................................................................. 40
Table 4-6  Mean offshore losses per wave condition ......................................................................... 40
Table 4-7  Bottom friction factors according Jonswap and breaker parameters............................... 51
Table 4-8  Measured and calculated landward velocities for sheets moved respectively 0,1,2 and 3 grid cells onshore (n.a. = not available due to numerical instability) ............................................. 53
Table 4-9  Calculated and measured offshore transport (positive) versus production and calculated transport and production ....................................................................................................... 57
Table 4-10 Influence of reducing the computational time step, grid size and morphological time step on the mass balance............................................................................................................... 58
Table 5-1  Default viscosity and diffusivity values ................................................................................. 76
Table A 1  Lido di Ostia test results
Table A 2  Measured wave height per test and averaged for mean wave condition at four locations
Table A 3  Measured water levels and adjusted water levels for the mathematical model
Table A 4  Measured velocities, averaged of highest 1/3
Table A 5  Velocity measurements per individual test, mean velocity gauge signal
Table A 6  Layer thickness
Table A 7  Manning coefficients
Table A 8  Initial wave height and wave period conditions at the boundary
Table A 9  Different ways to obtain the cross-shore transport rate
Table A 10 Offshore transport rates per wave condition (m$^3$/m/hr)
Table A 11 Production and offshore transports for various time steps and morphological time factors
Table A 12 Area increase for different THETSD values
Table A 13 Influence of transport parameters, reduction relative to reference run
Table A 14 Reference height as a function of the Manning coefficient
Table A 15 Relative, absolute and cumulative layer thickness
<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>beach shape parameter</td>
<td>[-]</td>
</tr>
<tr>
<td>A_0</td>
<td>pocket beach width</td>
<td>[m]</td>
</tr>
<tr>
<td>A_1</td>
<td>pocket beach indentation</td>
<td>[m]</td>
</tr>
<tr>
<td>A_(m,n)</td>
<td>surface area of cell m,n</td>
<td>[m^2]</td>
</tr>
<tr>
<td>ALFABS</td>
<td>longitudinal bed gradient factor for bed load transport</td>
<td>[-]</td>
</tr>
<tr>
<td>ALFABN</td>
<td>transverse bed gradient factor for bed load transport</td>
<td>[-]</td>
</tr>
<tr>
<td>AKSFAC</td>
<td>factor that influences Van Rijn’s reference height</td>
<td>[m]</td>
</tr>
<tr>
<td>a</td>
<td>Van Rijn’s reference height</td>
<td>[m]</td>
</tr>
<tr>
<td>B</td>
<td>crest width</td>
<td>[m]</td>
</tr>
<tr>
<td>B_0</td>
<td>pocket beach headlands width</td>
<td>[m]</td>
</tr>
<tr>
<td>BED</td>
<td>multiplication factor for the bed load transport</td>
<td>[-]</td>
</tr>
<tr>
<td>BEDW</td>
<td>wave related multiplication factor for the bed load transport</td>
<td>[-]</td>
</tr>
<tr>
<td>bodsed</td>
<td>available sediment at the bottom</td>
<td>[kg/m^2]</td>
</tr>
<tr>
<td>C</td>
<td>Chézy coefficient</td>
<td>[m^1/2/s]</td>
</tr>
<tr>
<td>C2D</td>
<td>2D-Chézy coefficient</td>
<td>[m^1/2/s]</td>
</tr>
<tr>
<td>C3D</td>
<td>3D-Chézy coefficient</td>
<td>[m^1/2/s]</td>
</tr>
<tr>
<td>C_0</td>
<td>empirical coefficients for parabolic bay equation</td>
<td>[-]</td>
</tr>
<tr>
<td>C_1</td>
<td>empirical coefficients for parabolic bay equation</td>
<td>[-]</td>
</tr>
<tr>
<td>C_2</td>
<td>empirical coefficients for parabolic bay equation</td>
<td>[-]</td>
</tr>
<tr>
<td>Cbottom</td>
<td>bottom friction coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>CDRYB</td>
<td>dry bed density</td>
<td>[kg/m^3]</td>
</tr>
<tr>
<td>CSOIL</td>
<td>reference density for hindered settling calculations</td>
<td>[kg/m^3]</td>
</tr>
<tr>
<td>c</td>
<td>mass concentration</td>
<td>[kg/m^3]</td>
</tr>
<tr>
<td>c</td>
<td>wave velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>c_a</td>
<td>reference concentration</td>
<td>[kg/m^3]</td>
</tr>
<tr>
<td>c_km (c_km)</td>
<td>average concentration of kmx layer</td>
<td>[kg/m^3]</td>
</tr>
<tr>
<td>c_g</td>
<td>wave group velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>D</td>
<td>eddy diffusivity coefficient</td>
<td>[m^2/s]</td>
</tr>
<tr>
<td>D</td>
<td>sediment deposition rate</td>
<td>[kg/m^2/s]</td>
</tr>
<tr>
<td>D</td>
<td>grain size</td>
<td>[m]</td>
</tr>
<tr>
<td>D_50</td>
<td>nominal grain size</td>
<td>[m]</td>
</tr>
<tr>
<td>D_e</td>
<td>equilibrium energy dissipation per unit volume</td>
<td>[J/m^3]</td>
</tr>
<tr>
<td>D</td>
<td>uniform wave energy dissipation per unit volume</td>
<td>[J/m^3]</td>
</tr>
<tr>
<td>D*</td>
<td>non-dimensional particle diameter</td>
<td>[-]</td>
</tr>
<tr>
<td>D_tot</td>
<td>rate of dissipation of the total energy due to wave breaking</td>
<td>[-]</td>
</tr>
<tr>
<td>DPSED</td>
<td>depth of sediment available at the bed</td>
<td>[m]</td>
</tr>
<tr>
<td>DENSIN</td>
<td>parameter to include the effect of the sediment on density gradients</td>
<td>[-]</td>
</tr>
<tr>
<td>d</td>
<td>thickness of the active layer over which changes take place</td>
<td>[m]</td>
</tr>
<tr>
<td>d</td>
<td>water depth below reference plane in FLOW</td>
<td>[m]</td>
</tr>
<tr>
<td>d_b</td>
<td>depth at breaker point</td>
<td>[m]</td>
</tr>
<tr>
<td>d_g</td>
<td>mean depth at the gap of a pocket beach</td>
<td>[m]</td>
</tr>
<tr>
<td>d_c</td>
<td>closure depth</td>
<td>[m]</td>
</tr>
<tr>
<td>d_f</td>
<td>freeboard</td>
<td>[m]</td>
</tr>
<tr>
<td>d_s</td>
<td>representative diameter of the sediment</td>
<td>[m]</td>
</tr>
<tr>
<td>d_i</td>
<td>depth at inner limit of the transition zone</td>
<td>[m]</td>
</tr>
<tr>
<td>d_prototype</td>
<td>depth in prototype scale</td>
<td>[m]</td>
</tr>
<tr>
<td>d_scale</td>
<td>depth in physical model scale</td>
<td>[m]</td>
</tr>
<tr>
<td>E</td>
<td>sediment erosion rate</td>
<td>[kg/m^2/s]</td>
</tr>
<tr>
<td>E</td>
<td>wave energy density</td>
<td>[J/m^3]</td>
</tr>
<tr>
<td>E_tot</td>
<td>total wave energy density</td>
<td>[J/m^3]</td>
</tr>
<tr>
<td>EROPAR</td>
<td>erosion parameter</td>
<td>[-]</td>
</tr>
<tr>
<td>EQMBC</td>
<td>parameter to use equilibrium concentration at the boundary</td>
<td>[-]</td>
</tr>
<tr>
<td>F</td>
<td>wave energy flux</td>
<td>[W/m]</td>
</tr>
<tr>
<td>Notation</td>
<td>Description</td>
<td>Unit</td>
</tr>
<tr>
<td>----------</td>
<td>-------------</td>
<td>------</td>
</tr>
<tr>
<td>$F_e$</td>
<td>amount of wave energy flux per unit width reaching the structure</td>
<td>[W/m]</td>
</tr>
<tr>
<td>$F_{er}$</td>
<td>wave energy flux per unit width reflected on the structure</td>
<td>[W/m]</td>
</tr>
<tr>
<td>$F_i$</td>
<td>wave energy flux per unit width that reaches the perched beach profile</td>
<td>[W/m]</td>
</tr>
<tr>
<td>$F_r$</td>
<td>stable wave energy flux per unit width</td>
<td>[W/m]</td>
</tr>
<tr>
<td>$F_s$</td>
<td>wave induced forcing term in x-direction</td>
<td>[N/m²]</td>
</tr>
<tr>
<td>$F_t$</td>
<td>wave induced forcing term in y-direction</td>
<td>[N/m²]</td>
</tr>
<tr>
<td>FWFAC</td>
<td>morphology input parameter</td>
<td>[-]</td>
</tr>
<tr>
<td>$f$</td>
<td>Coriolis parameter</td>
<td>[s⁻¹]</td>
</tr>
<tr>
<td>$f_{FIXFAX}$</td>
<td>fixed layer proximity factor</td>
<td>[-]</td>
</tr>
<tr>
<td>$f_{MORFAC}$</td>
<td>morphological time factor</td>
<td>[-]</td>
</tr>
<tr>
<td>$g$</td>
<td>gravity acceleration</td>
<td>[m/s²]</td>
</tr>
<tr>
<td>H</td>
<td>wave height in general</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_b$</td>
<td>water depth in Chézy formulation with the use of Manning</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_d$</td>
<td>deep water wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_b$</td>
<td>wave height at breaking</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_e$</td>
<td>near shore storm wave height that is exceeded only 12 hr/yr</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_e$</td>
<td>incoming wave height in the formulation of Gonzáles et al.</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_i$</td>
<td>incoming wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_r$</td>
<td>reflected wave height in the formulation of Gonzáles et al.</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_s$</td>
<td>significant wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_{sig}$</td>
<td>significant wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_{si}$</td>
<td>incoming significant wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>$H_t$</td>
<td>transmitted wave height</td>
<td>[m]</td>
</tr>
<tr>
<td>HMAXTH</td>
<td>maximum depth for variable THETSD</td>
<td>[m]</td>
</tr>
<tr>
<td>h</td>
<td>water depth</td>
<td>[m]</td>
</tr>
<tr>
<td>$h_b$</td>
<td>water depth at breaker point</td>
<td>[m]</td>
</tr>
<tr>
<td>$h_e$</td>
<td>water depth at the seaward side of the structure</td>
<td>[m]</td>
</tr>
<tr>
<td>$h_i$</td>
<td>water depth at the shoreward side of the structure</td>
<td>[m]</td>
</tr>
<tr>
<td>$h_T$</td>
<td>depth at which the linear beach slope is tangent to the $Ax^{2/3}$ profile</td>
<td>[-]</td>
</tr>
<tr>
<td>i</td>
<td>grid counter</td>
<td>[-]</td>
</tr>
<tr>
<td>K</td>
<td>length scale over which the wave energy flux is reduced from its value during breaking to a stable value</td>
<td>[m]</td>
</tr>
<tr>
<td>$K_s$</td>
<td>transmission coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>k</td>
<td>wave number</td>
<td>[m⁻¹]</td>
</tr>
<tr>
<td>$k_s$</td>
<td>Nikuradse roughness length</td>
<td>[m]</td>
</tr>
<tr>
<td>L</td>
<td>wavelength</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_b$</td>
<td>wavelength at breaking</td>
<td>[m]</td>
</tr>
<tr>
<td>$L_d$</td>
<td>deep water wavelength</td>
<td>[m]</td>
</tr>
<tr>
<td>LSED</td>
<td>number of sediment fractions</td>
<td>[-]</td>
</tr>
<tr>
<td>$M^s$</td>
<td>wave induced mass fluxes</td>
<td>[m³/sm]</td>
</tr>
<tr>
<td>m</td>
<td>beach slope</td>
<td>[-]</td>
</tr>
<tr>
<td>$m_b$</td>
<td>bottom slope in the vicinity of the breakpoint</td>
<td>[-]</td>
</tr>
<tr>
<td>MORFAC</td>
<td>morphological scale factor</td>
<td>[-]</td>
</tr>
<tr>
<td>MORSTT</td>
<td>minutes of delay before bottom updating</td>
<td>[min]</td>
</tr>
<tr>
<td>MORUP</td>
<td>parameter to activate or deactivate bottom updating during FLOW run</td>
<td>[-]</td>
</tr>
<tr>
<td>N</td>
<td>action density</td>
<td>[N/m/s]</td>
</tr>
<tr>
<td>n</td>
<td>ratio of wave group velocity to wave celerity</td>
<td>[-]</td>
</tr>
<tr>
<td>$n_s$</td>
<td>Manning coefficient</td>
<td>[m¹⁶]</td>
</tr>
<tr>
<td>$Q_b$</td>
<td>fraction of breaking waves</td>
<td>[-]</td>
</tr>
<tr>
<td>p</td>
<td>proportionality factor</td>
<td>[-]</td>
</tr>
<tr>
<td>R</td>
<td>reflection coefficient</td>
<td>[-]</td>
</tr>
<tr>
<td>R</td>
<td>pocket beach bay parameter, location of the shoreline at angles $\theta$</td>
<td>[-]</td>
</tr>
<tr>
<td>$R_0$</td>
<td>pocket beach bay parameter, control line between the ends of the two headlands</td>
<td>[-]</td>
</tr>
<tr>
<td>$R_c$</td>
<td>freeboard</td>
<td>[m]</td>
</tr>
<tr>
<td>RHOSOL</td>
<td>sediment density</td>
<td>[kg/m³]</td>
</tr>
<tr>
<td>ROUSE</td>
<td>factor to set the equilibrium sediment concentration values to standard Rouse profile</td>
<td>[-]</td>
</tr>
<tr>
<td>Notation</td>
<td>Description</td>
<td>Unit</td>
</tr>
<tr>
<td>-----------------</td>
<td>------------------------------------------------------------------------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>RWAVE</td>
<td>Wave roughness adjustment factor</td>
<td>[-]</td>
</tr>
<tr>
<td>S</td>
<td>Pocket beach width between headlands</td>
<td>[m]</td>
</tr>
<tr>
<td>S</td>
<td>Maximum stable surface area of the pocket beach</td>
<td>[m]</td>
</tr>
<tr>
<td>S_b`</td>
<td>Magnitude of the bed-load vector</td>
<td>[m^2/m/s]</td>
</tr>
<tr>
<td>S_e`</td>
<td>Equilibrium beach face slope</td>
<td>[-]</td>
</tr>
<tr>
<td>S</td>
<td>Source term in wave energy balance</td>
<td>[J/m^2]</td>
</tr>
<tr>
<td>S</td>
<td>Radiation stress</td>
<td>[N/m]</td>
</tr>
<tr>
<td>S_{xx}</td>
<td>Radiation stress acting on a plane parallel to the coast</td>
<td>[N/m]</td>
</tr>
<tr>
<td>S_{yx}</td>
<td>Radiation stress acting on a plane perpendicular to the coast</td>
<td>[N/m]</td>
</tr>
<tr>
<td>S_{yy}</td>
<td>Radiation stress acting on a plane perpendicular to the coast</td>
<td>[N/m]</td>
</tr>
<tr>
<td>S_b</td>
<td>Magnitude of the bed-load vector</td>
<td>[m^3/m/s]</td>
</tr>
<tr>
<td>S_{eq}</td>
<td>Equilibrium beach face slope</td>
<td>[-]</td>
</tr>
<tr>
<td>S_s</td>
<td>Constant rate of change of sediment transport along the x-axis</td>
<td>[m^3/yr]</td>
</tr>
<tr>
<td>s_s</td>
<td>Multiplication factor for suspended sediment transport</td>
<td>[m^3/m/s]</td>
</tr>
<tr>
<td>s_suu</td>
<td>Suspended sediment transport in u-direction</td>
<td>[m^3/m/s]</td>
</tr>
<tr>
<td>s_btr</td>
<td>Bed-load transport in u-direction through cross-section</td>
<td>[m^3/m/s]</td>
</tr>
<tr>
<td>s_buu</td>
<td>Bed-load transport in u-direction</td>
<td>[m^3/m/s]</td>
</tr>
<tr>
<td>SALMAX</td>
<td>Salinity for saline settling velocity</td>
<td>[-]</td>
</tr>
<tr>
<td>SEDDIA</td>
<td>Sediment diameter</td>
<td>[m]</td>
</tr>
<tr>
<td>SUS</td>
<td>Multiplication factor for suspended sediment transport</td>
<td>[-]</td>
</tr>
<tr>
<td>SUSW</td>
<td>Wave related multiplication factor for suspended sediment transport</td>
<td>[-]</td>
</tr>
<tr>
<td>T</td>
<td>Non-dimensional bed shear stress</td>
<td>[-]</td>
</tr>
<tr>
<td>TCRERO</td>
<td>Critical stress for erosion</td>
<td>[N/m^2]</td>
</tr>
<tr>
<td>TCRSED</td>
<td>Critical stress for sedimentation</td>
<td>[N/m^2]</td>
</tr>
<tr>
<td>THRESH</td>
<td>User-defined erosion threshold</td>
<td>[m]</td>
</tr>
<tr>
<td>THETSD</td>
<td>Fraction of erosion that is assigned to the adjacent dry cell</td>
<td>[-]</td>
</tr>
<tr>
<td>Û_o</td>
<td>Orbital motion amplitude near the bottom</td>
<td>[m/s]</td>
</tr>
<tr>
<td>u</td>
<td>Depth averaged velocity in x-direction</td>
<td>[m/s]</td>
</tr>
<tr>
<td>|u_b</td>
<td></td>
<td>Magnitude of the horizontal velocity in the first layer just above the bed</td>
</tr>
<tr>
<td>\u</td>
<td>Wave-mean velocity component</td>
<td>[m/s]</td>
</tr>
<tr>
<td>\u^{GLM}</td>
<td>Generalised Langrangian Mean-velocity vector</td>
<td>[m/s]</td>
</tr>
<tr>
<td>\u^{E}</td>
<td>Eulerian-velocity vector</td>
<td>[m/s]</td>
</tr>
<tr>
<td>\u^{S}</td>
<td>Stokes drift velocity-vector</td>
<td>[m/s]</td>
</tr>
<tr>
<td>u_b</td>
<td>Bed shear stress velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>u_{eff}</td>
<td>Effective bed shear stress velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>V</td>
<td>Alongshore current velocity, averaged over the depth</td>
<td>[m/s]</td>
</tr>
<tr>
<td>v</td>
<td>Depth averaged velocity in y-direction</td>
<td>[m/s]</td>
</tr>
<tr>
<td>W</td>
<td>Total length of the reef protected beach profile</td>
<td>[m]</td>
</tr>
<tr>
<td>WSO</td>
<td>Settling velocity for fresh water</td>
<td>[m/s]</td>
</tr>
<tr>
<td>WSM</td>
<td>Settling velocity for saline water</td>
<td>[m/s]</td>
</tr>
<tr>
<td>w</td>
<td>Sediment fall velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>w</td>
<td>Flow velocity component in z-direction</td>
<td>[m/s]</td>
</tr>
<tr>
<td>X</td>
<td>Horizontal distance from original shoreline to pocket beach headland breakwater</td>
<td>[m]</td>
</tr>
<tr>
<td>X_s</td>
<td>Horizontal distance of the transition zone</td>
<td>[m]</td>
</tr>
<tr>
<td>x</td>
<td>Offshore distance</td>
<td>[m]</td>
</tr>
<tr>
<td>Y</td>
<td>Accretion length</td>
<td>[m]</td>
</tr>
<tr>
<td>y</td>
<td>Alongshore distance</td>
<td>[m]</td>
</tr>
<tr>
<td>z_0</td>
<td>Bed roughness height</td>
<td>[m]</td>
</tr>
<tr>
<td>z_{buu}</td>
<td>Bed load transport in u-direction from observations points</td>
<td>[m^3/m/s]</td>
</tr>
<tr>
<td>z_{suu}</td>
<td>Suspended transport in u-direction from observations points</td>
<td>[m^3/m/s]</td>
</tr>
</tbody>
</table>
Greek symbols
\(\alpha\) = empirical parameter depending on the shape of the dam [-]
\(\alpha\) = empirical parameter in hyperbolic tangent shape equation [m]
\(\alpha_{BJ}\) = Battjes and Janssen dissipation coefficient due to wave breaking [-]
\(\beta\) = empirical parameter depending on the shape of the dam [-]
\(\beta\) = angle between the control line \(R_0\) and the predominant wave direction [degr]
\(\beta\) = slope at breaker point [-]
\(\beta\) = unknown concentration factor [-]
\(\beta\) = Van Rijn’s ‘Beta’ factor [-]
\(\beta_{eff}\) = effective Van Rijn’s ‘Beta’ factor [-]
\(\gamma\) = peak enhancement factor [-]
\(\gamma\) = wave height to depth ratio [-]
\(\Gamma\) = wave height to depth ratio for reef protected beaches [-]
\(\Delta z\) = difference in elevation between the centre of the \(k_{nx}\) cell and Van Rijn’s ref. height [m]
\(\eta\) = water level variation above the reference plane [m]
\(\eta\) = relative availability of the sediment fraction in the mixing layer [m]
\(\eta\) = coefficient to determine the degree to which the scheme in spectral space is upwind or central [-]
\(\varepsilon_s\) = eddy diffusity [m²/s]
\(\varepsilon_{fs}\) = vertical fluid mixing coefficient [m²/s]
\(\mu_c\) = current efficiency factor [-]
\(\nu\) = coefficient to determine the degree to which the scheme in spectral space is upwind or central [-]
\(\nu\) = diffusion coefficient (eddy viscosity) [m²/s]
\(\phi\) = latitude [degr]
\(\phi\) = angle of wave approach at depth \(d\) [degr]
\(\phi_0\) = angle of wave approach in deep water [degr]
\(\pi\) = constant (=3.14159265) [-]
\(\rho_s\) = sediment density [kg/m³]
\(\rho_w\) = water density [kg/m³]
\(\sigma\) = relative frequency [s⁻¹]
\(\sigma_c\) = Prandtl-Schmidt number [-]
\(\tau_v\) = bed shear stress due to currents [N/m²]
\(\tau_{bx,by}\) = \(x\)- and \(y\)- component of the bed load shear stress [N/m²]
\(\tau_{cwx}\) = current generated friction force [N/m²]
\(\tau_w\) = bed shear stress due to waves [N/m²]
\(\tau_{ij}\) = components of the wave-averaged normal stress tensor [N/m²]
\(\nu\) = kinematic viscosity coefficient of water [m²/s]
\(\theta\) = angle of the predominant wave crest [degr]
\(\phi\) = angle between the currents and the waves [degr]
\(\zeta\) = wave-mean free surface elevation [m]
\(\zeta\) = Iribarren parameter [-]
\(\xi\) = coefficient [-]
\(\omega\) = wave frequency [rad/s]
\(\Omega\) = angular rotation of the earth [rad/s]

Generally used subscripts and superscripts
\((..)_0\) = to indicate value in deep water
\((..)_{rp}\) = to indicate the reef protected beach
\((..)_x\) = component in \(x\)-direction
\((..)_y\) = component in \(y\)-direction
\((..)_0\) = component in \(\theta\)-direction
\((..)_\sigma\) = component in \(\sigma\)-direction
\((..)^{m,n}\) = indicators for cell \(m,n\)
1 Introduction

1.1 General introduction

Our beaches are always subject to erosion and accretion. Economic and safety reasons sometimes require appropriate measures. One can distinguish soft, hard and intermediate measures. A perched beach is an intermediate measure. The nourished beach profile is intersected by a submerged toe structure that retains the beach in a perched position, see Figure 1-1.

Another application area of the perched beach concept is the creation of recreational beaches in deep water or for improvement of swimming conditions. Economic and aesthetic considerations can plead for the use of perched beaches.

The advantages go hand in hand with the uncertainties it brings about. An example of a perched beach in Lido di Ostia, Italy, illustrates unpredicted and unwanted erosion, see Appendix F.

![Perched beach principle, cross-section (Sorensen, 1987)](image.png)

Figure 1-1 Perched beach principle, cross-section (Sorensen, 1987)

1.2 Problem analysis

1.2.1 Problem description

Framework

Due to lack of capacity, it was decided to construct the Third Harbour in the port of IJmuiden, see Figure 1-2. Lievense Consulting Engineers in Breda is involved with this project. A part of this project is the relocation of the former recreational beach. This beach has vanished by the construction of the Third Harbour. Local politics decided to reconstruct this beach and place it westerly of the new Third Harbour, see Appendix A. Since there is little physical space, it was decided to construct a perched beach to prevent the gully to the marina from silting up and to be able to design a less steep beach profile.
CHAPTER 1. Introduction

**Figure 1-2** Former recreational beach versus new Third Harbour and perched beach

**Design**
In IJmuiden, the perched beach at the westerly side is intersected with the deeper entrance to the marina (NAP –4m to NAP –9m) by sheet piles, see Figure 1-3 and Figure 1-4. These sheet piles form a sill from NAP –2.0 m and deeper. From this sill, the designed perched beach with 300 micron sand has a slope of 1:30 up to the revetment like construction at NAP +1.5m, see Figure 1-4. At the higher terrace (NAP +5.0m), recreational houses are placed during the summer season, see Figure A 7. The 250-meter wide beach is bounded between a new built dam at the north and at the existing breakwater at the south. Therefore this beach satisfies the pocket beach definition, see §2.3.1. A more extensive description of the layout, cross-section and photo’s can be found in Appendix A.

**Figure 1-3** Perched beach layout

**Problems**
Under influence of wave action sand will be transported. Losses over the toe construction during storm surges are permanent. In calm periods sand can not be brought back by natural processes and the beach has to be replenished by artificial nourishment to prevent the hard transitional construction that retains the elevation of the recreational houses from undermining by scour. Beside that, scour holes could endanger swimmers and large sand losses could silt up the marina entrance.
For the design of a new perched beach, no general design rules are available. For different perched beaches, the profile has been designed with the use of several specific mathematical and physical model tests. The physical models are often 2DV and the mathematical models are often used without much calibration. After construction, these designs often appeared to be unstable, e.g. Lido di Ostia, see Appendix F.

Also for the pocket perched beach at IJmuiden, one expects that secondary currents, due to non-uniformity of the hydraulic conditions, i.e. different wave heights and directions alongshore, play a significant role in beach development, see Appendix B. The beach in IJmuiden as well as future perched beaches induce need for investigation of these effects.

1.2.2 Problem definition
There is a need for general insight into pocket perched beaches and the influence of non-uniformity on pocket perched beaches. Current mathematical models are often not yet calibrated for perched beach conditions.

1.2.3 Aim of the study
First, a general insight of pocket perched beaches is wanted. The second and main goal is the calibration of the mathematical model by means of physical measurements. Then, the used mathematical model has to be understood for the use of perched beach conditions. Several hydraulic measurements have to be calibrated, such as water level including wave set-up, wave transformation over a submerged breakwater, velocities patterns and sediment transport. The model has to satisfy several conservation laws.

1.2.4 Problem approach
To satisfy the need for general insight of pocket perched beaches, first theory, studies and performances of pocket perched beaches will be investigated by means of a literature study. The pocket perched beach concept is investigated with a mathematical model, called Delft3D. First, the model is calibrated with the use of a dataset from physical tests for a perched beach in Gibraltar, see Appendix D. The calculations are done three-dimensionally, with ten depth layers. Initially, the main goal was to investigate the influence of non-uniformity in geometry or hydraulic conditions, but restrictions of the model during the calibration procedure prevented further investigation.

1.3 Lay-out of the report
Chapter 2 explains the literature study on pocket perched beaches and some underlying theory about waves and beaches that is used later in the study. Chapter 3 describes the mathematical model that is used and some specific features concerning perched beach modelling with the mathematical model. Chapter 4 explains the calibration procedure. The used physical tests and test results are discussed first. After a model set-up description, the actual calibration is described and the validity is checked. In Chapter 5, an analysis is made of the sensitivity of the calibrated model. This concerns sensitivity to numerical, empirical and hydraulic parameters. This report is finished with conclusions and recommendations in Chapter 6. Generally, it is attempted to report concisely with a more profound elaboration in the appendices. The appendices are added unattached.
2 Literature study

2.1 Introduction
In this chapter, first in §2.2 some underlying theory about waves and beaches is put forward which is used and referred to later on in this study. In §2.3 the pocket beaches and their shapes are explained. In §2.4, the perched beach concept is explained, followed by an inventory of studies and prototype perched beaches. Together, this information should satisfy the need for information about pocket perched beaches.

2.2 Waves and beaches
In this paragraph, theory is put forward that is used later during the research. It is assumed that the reader is familiar with hydrodynamics.

2.2.1 Wave transmission
The transmission coefficient is defined as the quotient of outgoing and incoming wave height over a submerged structure, see definition sketch in Figure 2-1. These relations are used and compared later on in this study.

\[ K_t = \frac{H_f}{H_i} \]

Several equations describe the relation between these wave heights. Here a distinction is made between relations that do and do not consider the crest width.

Goda et al. (1967) proposed a transmission coefficient relation as in Equation (2.1).

\[ K_t = 0.5 \left[ 1 - \sin \left( \frac{\pi}{2\alpha} \left( \frac{R_c}{H_i} + \beta \right) \right) \right] \]

with boundary condition:

\[ \beta - \alpha \leq \frac{R_c}{H_i} \leq \alpha - \beta \]

where

- \( \alpha \) = empirical coefficient, depending on the shape of the dam
- \( \beta \) = empirical coefficient, depending on the shape of the dam
- \( R_c \) = freeboard
- \( H_i \) = incoming wave height

Van der Meer (1991) defined a relation through a scatter of measured values, depending on the freeboard and wave height.

\[ K_t = 0.8 \]

for \(-2.0 < R_c/H_i < -1.13\)

\[ K_t = 0.46 - 0.3 \frac{R_c}{H_i} \]
for \(-1.13 < \frac{R_c}{H_i} < 1.2\) 

\[ K_t = 0.1 \] \hspace{1cm} (2.5)

for \(1.2 < \frac{R_c}{H_i} < 2.0\)

D’Angremond et al. (1996) recognise a dependence of \(K_t\) on permeability and crest width:

\[ K_t = -0.4 \frac{R_c}{H_{st}} + \left( \frac{B}{H_{st}} \right)^{-0.31} \cdot \left( 1 - e^{-0.5\xi} \right) \cdot 0.64 \] \hspace{1cm} (2.6)

for permeable breakwaters

\[ K_t = -0.4 \frac{R_c}{H_{st}} + \left( \frac{B}{H_{st}} \right)^{-0.31} \cdot \left( 1 - e^{-0.5\xi} \right) \cdot 0.80 \] \hspace{1cm} (2.7)

for impermeable breakwaters

both with limits for the value of \(K_t\): 0.075 < \(K_t\) < 0.80

where

\(R_c = \text{freeboard} \quad [\text{m}]\)

\(B = \text{crest width} \quad [\text{m}]\)

\(H_{st} = \text{incoming significant wave height} \quad [\text{m}]\)

\(\xi = \text{Iribarren parameter, } \xi = \tan \alpha / (H/L)^{0.5} \quad [-]\)

Seabrook and Hall (1998) also consider the crest width, see Equation (2.8).

\[ K_t = 1 - \left( e^{-0.65 \left( \frac{d_s}{H_i} \right) - 1.09 \left( \frac{d_s}{B} \right)} + 0.047 \left( \frac{B}{L} \frac{d_s}{D_{50a}} \right) - 0.067 \left( \frac{d_s}{B} \frac{H_i}{D_{50a}} \right) \right) \] \hspace{1cm} (2.8)

where

\(d_s = \text{freeboard} \quad [\text{m}]\)

\(H_i = \text{incoming wave height} \quad [\text{m}]\)

\(B = \text{crest width} \quad [\text{m}]\)

\(D_{50a} = \text{nominal armour material size} \quad [\text{m}]\)

\(L = \text{wavelength} \quad [\text{m}]\)

For other relationships, the reader is referred to the list of references. Seelig (1979) underscores the importance of the relative crest height, i.e. \(R_c/H_i\). Powell and Allsop (1985) take the influence of the wave period into account. Deamrich et al. (1985) studied the influence of the wave period, crest width and roughness. Ahrens (1987) investigated reef breakwaters and underscored the correlation between cross-sectional area and the decrease of the transmission. Hearn (1987) describes an exponential relation between transmission and relative crest height for submerged breakwaters.
CHAPTER 2. Literature study

2.2.2 Transition zone

Svendsen (1984) divides the surf zone into an outer zone, inner zone and run-up region or swash zone, see Figure 2-2. “The transition zone is the region just shoreward of the point of breaking and is characterised by rapid decay and also by a constant wave set-down, and thus constant radiation stress”, Nairn et al. (1992). During the calibration in §4.4 the transition zone is simulated according to formulations in this paragraph.

Figure 2-2 Surf zone division (Svendsen, 1984)

Svendsen (1984) proposed a definition of the transition zone as the region of nearly horizontal or very weakly sloping water level just landward of the breaking point, before the beginning of a steep gradient of the water level due to wave set-up. Svendsen suggested that potential energy is converted to forward momentum. Basco and Yamashita (1986) viewed the process in the transition zone as the transformation from oscillatory wave motion to highly turbulent motion.

The nearly constant water level implies that the radiation stress is also constant, considering the momentum balance equation. Consequently, the wave-induced cross-shore currents due to reduction of radiation stress will not commence until the inner limit of the transition zone is reached. Visser (1984) found that it was critically important to delay the initiation of the influence of energy dissipation on the generation of currents until the plunge point. As noted by Basco and Yamashita (1986) the distance from the break point to the plunge point is a significant fraction of the transition zone.

Three interpretations of this width will be explained:
- Basco (1985) proposed values of the transition zone width of 10% to 30% of the wavelength at breaking.
- Nairn et al. (1992) propose a distance, dependent on the surf similarity parameter ($\xi$), the depth at breaker point ($d_b$) and the depth at the inner limit of the transition zone ($d_t$). A dataset was gathered to determine a width, defined as the distance from the breakpoint to the abrupt change in slope of the mean water level, see Figure 2-3. The following relations are derived:

$$\frac{d_t}{d_b} = 0.47 \xi_{bb}^{-0.275}$$

, for $\xi \geq 0.05$ (2.9)

$$\frac{d_t}{d_b} \approx 1$$

, for $\xi_{bb} < 0.05$ (2.10)
where

\[ \begin{align*}
\delta_t & = \text{depth at the inner limit of the transition zone} \quad [\text{m}] \\
\delta_b & = \text{depth at breaking} \quad [\text{m}] \\
\zeta_{bb} & = \text{surf similarity parameter;} \quad [-] \\
\end{align*} \]

\[ \zeta_{bb} = \frac{m_b}{\left( \frac{H_b}{L_b} \right)^{0.5}} \quad (2.11) \]

\[ \begin{align*}
\zeta_{bb} & = \text{surf similarity parameter;} \\
m_b & = \text{bottom slope in the vicinity of the breakpoint} \quad [\text{m}] \\
H_b & = \text{wave height at breaking} \quad [\text{m}] \\
L_b & = \text{wavelength at breaking} \quad [\text{m}] \\
\end{align*} \]

In this relation (Equation (2.9)), \( \delta_t/\delta_b \) decreases if the surf similarity parameter increases. This means that the transition zone becomes less for smaller wave steepness and an equal slope.

\[ \zeta_{bb} = \frac{m_b}{\left( \frac{H_b}{L_b} \right)^{0.5}} \quad (2.11) \]

\[ \begin{align*}
X_t & = 2 \left( 4 - 9.52 \tan \beta \right) H_b \\
X_t & = 8 H_b \\
\end{align*} \quad (2.12) \quad (2.13) \]

where

\[ \begin{align*}
X_t & = \text{horizontal distance of transition zone} \quad [\text{m}] \\
H_b & = \text{wave height at breaking} \quad [\text{m}] \\
\beta & = \text{slope at breaker point} \quad [-] \\
\end{align*} \]

- Sawaragi’s (1995) definition only relates the distance to the slope (\( \beta \)) and the breaking wave height:

\[ X_t = 2 \left( 4 - 9.52 \tan \beta \right) H_b \quad (2.12) \]

\[ X_t = 8 H_b \quad , \text{for } \tan \beta = 0 \quad (2.13) \]

\[ \begin{align*}
X_t & = \text{horizontal distance of transition zone} \quad [\text{m}] \\
H_b & = \text{wave height at breaking} \quad [\text{m}] \\
\beta & = \text{slope at breaker point} \quad [-] \\
\end{align*} \]
2.2.3 Closure depth

Birkemeier (1985) defines the closure depth as the depth at which no measurable change in bottom elevation occurs after a storm, see Figure 2-4. The best fit to measurement data eventually led to Equation (2.14).

\[ d_l = 1.57H_e \]  

where

- \( d_l \) = closure depth, relative to mean low water [m]
- \( H_e \) = near-shore storm wave height that is exceeded only 12 hr/yr [m]

![Cross-shore definition sketch closure depth]

2.2.4 Wave set-up

With perpendicular incoming waves, (long crested, no directional spreading) as with the physical scale model, the water level gradient within the breaker zone is determined by a gradient in radiation stress. Conservation of momentum requires equilibrium of forces, see Figure 2-5 for an example. At every place, there are hydraulic forces and forces by radiation stress.

The hydraulic pressure force equals:

\[ P = \frac{1}{2} \rho gh^2 \]  

where

- \( \rho \) = water density [kg/m³]
- \( g \) = gravity acceleration [m/s²]
- \( h \) = water depth [m]

The radiation stress component perpendicular to the coast exerts a force as described in Equation (2.16).

\[ S_{xx} = \left( 2n - \frac{1}{2} \right) E \]  

where

- \( E \) = wave energy density [J/m²]

\[ E = \frac{1}{8} \rho g H^2 \]  

\[ n = \frac{c_g}{c} = \frac{kh}{\sinh 2kh} + \frac{1}{2} \]
where
\[
\begin{align*}
    c_g &= \text{wave group celerity} \\
    c &= \text{wave celerity} \\
    k &= \text{wave number, } k = 2\pi/\lambda.
\end{align*}
\]

for shallow water: \(n=1\), for deep water \(n=0.5\).

---

**Figure 2-5**  
Example of wave set-up, wave height decay and the balance of momentum at the breakwater

For equilibrium of forces, the sum of the hydraulic force and the radiation stress must remain constant in the breaker zone. A reduction of the wave height by breaking results in a smaller radiation stress and therefore the hydraulic force must increase to retain the equilibrium.

The set-up can be reduced by gaps in the submerged structure or at the end of the barrier, but this induces strong currents through the gaps, where the bed must be protected adequately.

### 2.2.5 Beach profiles

This paragraph expands the development of several beach profile equations, either for sake of completion or to be used later in this report. The most widely used formulation of the equilibrium beach profile was proposed by Bruun (1954) and Dean (1977). Bruun assumed a beach profile shape given by Equation (2.19).

\[
h = Ax^p
\]  \quad (2.19)

where
\[
\begin{align*}
    h &= \text{depth relative to m.w.l.} \\
    x &= \text{offshore distance, } x=0 \text{ at m.w.l.} \\
    A &= \text{shape parameter} \\
    p &= \text{power}
\end{align*}
\]

Dean (1977) showed that the assumption of constant wave energy dissipation per unit volume is consistent with this Equation (2.19), see Equation (2.20) to (2.22).
\[ \frac{1}{h} \frac{dF}{dx} = D_{eq} \]  

(2.20)

where

- \( h \) = water depth [m]
- \( x \) = distance from the shoreline [m]
- \( F \) = wave energy flux in shallow water \( \{1/8 \rho h^2 (g d)^{1/2}\} \) [W/m]
- \( D_{eq} \) = equilibrium energy dissipation per unit volume [J/m³]

Integrating with Equation (2.21) gives Equation (2.22)

\[ H_b = \gamma h_b \]  

(2.21)

where

- \( H_b \) = breaking wave height [m]
- \( h_b \) = depth at breaker point [m]
- \( \gamma \) = wave height to depth ratio [-]

\[ x = \left( \frac{h}{A} \right)^{3/2} \]  

(2.22)

Equation (2.22) can be written to Equation (2.23)

\[ h = Ax^{2/3} \]  

(2.23)

where

- \( A \) = shape parameter [m¹/³]

\[ A = \left( \frac{24D_{eq}}{5 \rho g^{7/3} \gamma^2} \right)^{2/3} \]  

(2.24)

Several empirical relations are proposed on the general power profile of Bruun (1954).

Wiegel (1964) derived a graphical empirical relation between sediment size and beach foreshore slope for protected and unprotected beaches, obtained from Bascom (1959), where the foreshore is determined as the area between MLW and summer berm.


\[ y = 0.10x^{2/3} \]  

(2.25)

Moore (1982), related the shape parameter \( A \) graphically to the median grain size, \( d_{50} \).

Vellinga (1984) proposed the relation in Equation (2.26).

\[ y = 0.70 \left( \frac{H_0}{L_0} \right)^{0.17} w^{0.44} \gamma^{0.78} \]  

(2.26)

which is valid for

- depth: 0.7 up to 1.0 times the significant wave height
- \( H_0/L_0 \): 0.025 up to 0.04
- \( H_{33} \): 3.0 m up to 8.0 m
- \( D_{50} \): 150 µm up to 400 µm

where

- \( H_0 \) = deep water wave height [m]
- \( L_0 \) = deep water wavelength [m]
- \( w \) = sediment fall velocity [m]

Dean (1986) depicted the value ‘A’ in Figure 2-6 as a function of the breaking wave height, period and sediment fall velocity.
Figure 2-6  \( A \)-values versus \( \frac{H_b}{W_sT} \) (Dean, 1986)

Larson and Kraus (1989) defined a general equilibrium profile with sloping beach-face, using the dissipation model of Dally et al. (1985), see Equation (2.27).

\[
D_{eq} = \frac{K}{h^3} \left( F - F_s \right)
\]

(2.27)

where

- \( D_{eq} \) = equilibrium energy dissipation per unit volume \([\text{J/m}^3]\)
- \( K \) = parameter related to the length scale over which the wave energy flux is reduced from its value during breaking to a stable value \([\text{m}]\)
- \( F \) = energy flux during breaking, see Equation (2.28) \([\text{W/m}]\)
- \( F_s \) = stable energy flux, see Equation (2.29) \([\text{W/m}]\)
- \( \Gamma \) = stable wave height-to-depth ratio \([-]\)

For a beach in equilibrium, Equation (2.27) is solved to obtain the breaker wave height at any depth, as described by Equation (2.30).

\[
H = \left( \Gamma^2 h^2 + \frac{8D_e}{\rho g^{3/2} K} h^{3/2} \right)^{1/2}
\]

(2.30)

Using this wave height description and integration of Equation (2.20) resulted in an equilibrium beach profile as described in Equation (2.31).

\[
x = \frac{2}{K} h + \frac{\rho g^{3/2} \Gamma^2}{24D_e} h^{3/2}
\]

(2.31)

Kriebel et al. (1991) have suggested that a fraction \( \epsilon \) of the wave energy dissipation per unit volume due to wave breaking must equal the energy dissipation associated with suspended sand grains falling under their own weight.

\[
\epsilon D_e = \beta (s - 1) \rho gw
\]

(2.32)
where
\[ \beta = \text{unknown concentration factor} \quad [-] \]
\[ s = \text{specific gravity of the sediment} \quad [-] \]
\[ w = \text{sediment fall velocity} \quad [\text{m/s}] \]

Substituting this expression in Equation (2.24) yields:
\[
A = \left( \frac{24\beta(s-1)}{5\epsilon k^2} \right)^{2/3} \left( \frac{w^2}{g} \right)^{1/3} \approx 2.25 \left( \frac{w^2}{g} \right)^{1/3} \tag{2.33}
\]

This correlation between \( A \) and sediment fall velocity \( w \) is valid for water temperature 20°C and sediment fall velocities 1-10 cm/s.

Kriebel et al. (1991) modified Dean’s Equilibrium beach profile. Dean’s profile form is adopted in the surf zone and replaced by a linear beach-slope near and above the still water level
\[
x = \frac{h}{m}, \quad \text{if } h < h_T \tag{2.34}
\]
\[
x = x_0 + \left( \frac{h}{A} \right)^{3/2}, \quad \text{if } h > h_T \tag{2.35}
\]
where
\[ x = \text{distance from the still water shoreline to the virtual origin of the concave equilibrium form} \quad [\text{m}] \]
\[ x_0 = h_T - \left( \frac{h_T}{A} \right)^{3/2} \tag{2.36} \]

where
\[ h_T = \text{depth at which the linear beach slope is tangent to the } Ax^{2/3}\text{ profile} \quad [\text{m}] \]
\[ h_T = \frac{4}{9} \frac{A^3}{m^2} \tag{2.37} \]

where
\[ m = \text{beach slope, by Sunamura (Eq. (2.38)) or Kriebel et al. (Eq. (2.39))} \quad [-] \]
\[ m = 0.12 \left( \frac{g^{1/2} d_{50}^{1/2} T}{H} \right)^{1/2}, \quad \text{(Sunamura, 1984)} \tag{2.38} \]
\[ m = 0.15 \left( \frac{wT}{H} \right)^{1/2}, \quad \text{(Kriebel et al. 1991)} \tag{2.39} \]

Walton (1998) summarised the recommended ‘A’ values in a table, depending on the grain size
\[
\begin{array}{cc}
D=0.22 & \Lambda=0.106 \\
D=0.30 & \Lambda=0.125 \\
D=0.50 & \Lambda=0.161 \\
\end{array}
\]

Van Rijn (1998) proposed a relationship between equilibrium beach-face slope, \( S_{eq} \) (upper beach slope), grain size and wave properties \( H_{br} \) and \( T \), see Equation (2.40).
\[
S_{eq} = 0.12 \left( \frac{H_{br}^2}{gd_{50}^2 T^2} \right) \tag{2.40}
\]

Chia (2001) divided the beach profile in three parts; the submerged beach, the intertidal beach and the upper beach, the latter with a maximum beach gradient of 1:8.
CHAPTER 2. Literature study

Effect of the available sediment
When the beach itself has enough sand deposit, a cliff can be formed.

Effects of sediment size
Increasing the sediment size will result in a steeper upper beach slope, because of the following points:
- Higher natural angle of repose.
- Asymmetry of the swash motion of the waves. The backwash motion tends to be weaker than the up-rush motion because of the water percolating into the beach face. Since the rate of percolation increases with increasing grain size, beaches with coarser materials would have a steeper slope.
- The grading of the bed material. The degree of sorting of the bed material affects the rate of percolation. Beaches with non-uniform, well-sorted grains have a steeper slope.

Effect of the tidal range
- During the flood tide, the sea level raises faster than the ground water level. This increases the percolation and results in a steeper slope around the high waterline.
- During ebb tide, the sea level falls faster than the ground water level. Instead of percolation, the water comes out of the beach. This additional ground water increases the backwash, resulting in a flatter beach slope around the low water line.

Effect of water level change
Relative sea level rise will result in the same beach profile as before the rise, but either as higher as the sea level rise or moved onshore, Bruun (1954).

2.3 Pocket beaches

2.3.1 Introduction
Bruun has stated that “Nature has not only demonstrated how to erode but also how to protect”. The concept of using headland breakwaters to protect beach erosion is credited to engineers’ willingness to copy nature’s work. Pocket beaches are defined as beaches between barriers, where no sediment comes in or goes out in alongshore directions. These headlands can either be natural as at Long Beach, Australia, see Figure 2-7, or artificial, as the pocket perched beach at Kemigawa, Japan, see Figure 2-8.

Figure 2-7 Natural pocket beach between headlands, Long Beach, Australia (source: internet)
Since no sediment is transported in or out of the pocket beach, pocket beaches may lose beach width, but may, by definition, not lose their total sand volume. This is, of course when the beach is enclosed deeper than the closure depth, see §2.2.3. The next two paragraphs discuss the beach shape for a pocket beach between headlands and between groins.

### 2.3.2 Parabolic bay shape between headland breakwaters

The shape of pocket beaches between headlands is influenced by the location of the headlands. Where the headlands are closely spaced and a limited volume of sediment is present, small pocket beaches are formed. Where the headlands are far apart and an adequate sediment supply exists, long and wide beaches are formed. Generally, between these two extremes, most beaches between natural headlands take a shape that is related to the predominant wave approach; on the down-coast sector is a long and straight beach, while on the up-coast end the beach is curved, see Figure 2-9.
Berenguer and Enriquez (1988) published a useful set of design tools for pocket beaches where the tidal range is less than 1 meter, based on an analysis of 24 Spanish artificial pocket beaches. With reference to the definition sketch, see Figure 2-10, the following relations were derived from the analysis of geometric parameters:

\[ A_0 = 2A_i \]  
\[ A_i = 25 + 0.85S \]  
\[ XB_0 = 2.5A_i^2 \]  
\[ S_p = XB_0 - \frac{1}{2}\pi A_i^2 = 0.37XB_0 \]

where 
\[ S_p = \text{maximum stable surface area of the beach} \quad [\text{m}^2] \]

In addition, a graphical relationship was developed between the cross-sectional surface of the gap (\(S_{dg}\)) and the dimensions of the resulting beach (\(A_i^{-1/2}D_m\)), see Figure 2-10, where \(d_g\) is the mean depth in the gap and \(D_m\) is the sediment grain size (in mm).

Hsu and Evans (1989) defined a parabolic beach profile by:

\[ \frac{R}{R_0} = C_0 + C_1\left(\frac{\beta}{\theta}\right) + C_2\left(\frac{\beta}{\theta}\right)^2 \]  

(2.45)
Where the parameters $R$, $R_o$, $\beta$ and $\theta$ are shown in the left graph of Figure 2-11 and the coefficients $C_o$, $C_1$ and $C_2$ are empirically derived constants, see the right graph of Figure 2-11. $R_o$ represents a control line between the ends of the two headlands. The angle $\beta$ represents the angle between the control line $R_o$ and the predominant wave direction. The distance $R$ defines the location of the shoreline at angles $\theta$ measured from the predominant wave crest.

Another study by Hsu et al. (1989) gives a relationship for $R/R_o$ in terms of $\beta$ and $\theta$ only:

$$\frac{R}{R_o} = 0.81 \frac{\beta^{0.83}}{\theta^{0.77}}$$  \hspace{1cm} (2.46)

Although simpler to use, this formula does not provide a good fit for large values of $\beta$ and $\theta$. Hence, this relationship is not considered universal.

Tan and Chiew (1998) provided another version of the parabolic relationship for $R/R_o$:

$$\frac{R}{R_o} = (1 - \beta \cot \beta + \alpha) + (\beta \cot \beta + \alpha) \left( \frac{\beta}{\theta} \right) + \alpha \left( \frac{\beta}{\theta} \right)^2$$  \hspace{1cm} (2.47)

The constant $\alpha$ can be derived from Equation (2.48):

$$\log^{10} \left( 0.277 - \alpha \right) = \left( \frac{\beta \pi}{180} \right) - 1.105$$  \hspace{1cm} (2.48)

This equation is similar to Hsu and Evans’ equation, except that the coefficients $C_o$, $C_1$ and $C_2$ are now given in terms of $\alpha$ and $\beta$.

The latest development in static equilibrium shape of headland-bay beaches is the hyperbolic tangent shape proposed by Moreno and Kraus (1999). The hyperbolic tangent shape is defined, in Cartesian coordinates, by:

$$y = (\pm) a \tanh^m (bx)$$  \hspace{1cm} (2.49)

where

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$y$</td>
<td>distance across shore</td>
<td>[m]</td>
</tr>
<tr>
<td>$x$</td>
<td>distance alongshore</td>
<td>[m]</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>empirical parameter</td>
<td>[m]</td>
</tr>
<tr>
<td>$b$</td>
<td>empirical parameter</td>
<td>[m]</td>
</tr>
<tr>
<td>$m$</td>
<td>empirical parameter</td>
<td>[-]</td>
</tr>
</tbody>
</table>
2.3.3 Equilibrium beach shape between groins

Chia (2001) describes a graphical method to estimate the equilibrium beach shape by iterative steps. Figure 2-12 shows a definition sketch of the equilibrium beach shape between groins. In Appendix C an extensive derivation is given of the groin accretion length formula.

The steps to be taken are:

1. A line making an angle $\phi'$ with the coastline was drawn through point O, located somewhere near the centre of the two groins, in the direction of the initial sediment transport, i.e. to the right in Figure 2-12.
2. The angle $\phi'$ is defined as the wave approach angle between the wave crest and the coastline at the active depth $d$. The accretion length $Y$ is demarcated by point $Y$.
3. From the tip of the up-wave groin, point A, the H=100% line is drawn to intersect the line drawn in step 1 at point E. H=100% is where the diffracted wave height equals the incident wave height and is extracted from Wiegel's wave diffraction diagram (Shore Protection Manual, 1984).
4. The curve from point E to point B was drawn using Hsu and Evans (1989) Equation:

$$ \frac{R}{R_0} = C_0 + C_1 \left( \frac{\beta'}{\theta} \right) + C_2 \left( \frac{\beta'}{\theta} \right)^2 $$

(eq. (2.45))  \hspace{1cm} (2.50)

Steps 1 to 4 are iterated until the eroded area is equal to the accreted area. The accretion length at the groin is defined by $Y$, which initially can be approached by Equation (A.18) in Appendix C. The maximum indentation is defined by the distance between the original coastline and point C.
2.4 Perched beaches

In this paragraph first the principle of the perched beach concept is explained in §2.4.1. In §2.4.2 theory involving perched beaches is expanded to serve as an aid when designing a perched beach. In §2.4.3, the advantages and disadvantages of the perched beach concept are enumerated, followed by an inventory of perched beaches in §2.4.4.

2.4.1 Perched beach concept

Beach erosion can be stopped or slowed down in three different ways. By applying hard structures, soft solutions or a combination of the latter two.

One of the latter concepts of controlling beach erosion is the perched beach concept. “The basic concept of the perched beach is to reproduce the former beach profile to some convenient seaward point and then intersect this profile with a submerged toe structure to retain the beach in a perched position.” (Chatham 1972). Figure 2-13 shows different options for the submerged toe structure.

Within the perched beach concept, there are generally two variants.

1. The breakwater variant. Here, the structure has a relatively small freeboard and is intended to act as both a wave dissipation- and a beach retaining structure. The advantages of a minor nourishment and a small occupation of physical space is counteracted by larger construction costs and a great sensitivity to water level fluctuations since the wave height reduction is dependent on the free board of the crest.

2. The sill variant. Here the structure has a relatively large free board; it only supports the beach and has no longer a function to reduce the incident wave energy. Compared to the breakwater option it is less sensitive for the actual water level and the potential of beach profile restoration during low waves is higher.

In this report, it is tried to discern these different variants in the text by speaking of the submerged structure as submerged breakwater or as submerged sill.

2.4.2 Theory involving perched beaches

Concerning the location of the submerged structure:

The convenient seaward point is usually where the natural protective offshore bar is formed during storm condition. The submerged toe structure is usually constructed parallel to the existing shoreline and sometimes closed off by groins. Gunyakti (1987) advises to locate the breakwater offshore off the natural breaking zone. The Canadian handbook for coastline property owners’ (Pilarczyk and Zeldler, 1996) rule of thumb is to locate the sill thirty times the difference between the design high water level (DHWL) and the mean summer water level (MSWL) plus 10 meters lakeward of the proposed shoreline. Lamberti and Mancinelli (1996) propose a sufficient distance from the shoreline, so turbulence induced by breakers decays before reaching the inner beach. It might be superfluous to state that for equal crest heights a breakwater becomes cheaper placing it more onshore in shallower waters.
CHAPTER 2. Literature study

Concerning the crest height:
Arai and Tamura (1987) state that the crest height depends on the breaking point of most incident waves, stability reasons and on safety reasons for swimmers. The Canadian handbook’s rule of thumb is to design the top elevation of the sill approximately 0.3 meters lower than MSWL. The reduction of the wave height on the barrier should be strong enough to ensure the formation inshore of the barrier itself of a beach profile down to its closure depth, Lamberti and Mancinelli (1996). Lamberti and Mancinelli propose a submergence less than half the local depth, or less than 1/3 for storm conditions. The WL | Delft Hydraulics report (1991-b) proposes a crest height above MLW for passableness. In addition, the relation between crest height and offshore losses is emphasised.

Concerning the crest width:
The breakwater crest width should be at least wide enough so that breaking wave collapse on the breakwater instead of on the beach. Instead of a wide breakwater, one can use an apron.

Concerning the beach profile:
Dean (1988) emphasises that the perched beach profile is equal to a beach profile without a sill, viz. \( h = A x^{2/3} \), see Figure 2-14.

\[ D^* = \frac{1}{h} \frac{\partial}{\partial x} \left( E c_g \right) \] (2.51)

where

- \( E \) = wave energy density, see Equation (2.17) \([J/m^2]\)
- \( c_g \) = group velocity \([m/s]\)

Assuming that this hypothesis is right, the equilibrium profile for a perched beach depends on the amount of energy flux that is transmitted over the toe structure. The model assumes that wave reflection at the seaward and leeward side is the most important factor that modifies Dean’s equilibrium profile for non-perched beaches. Besides friction damping, wave breaking over the structure is neglected since the breakwater crest width is usually much smaller than the wavelength and breaking occurs on the perched beach.
Using Dean’s equilibrium beach profile, \( h = A x^{2/3} \), the perched beach profile is defined if \( h_i \) is given, which, due to the constant breaker-to-depth ratio, can be determined if \( F_i \) is known, see Figure 2-15. \( F_i \) can be determined by solving the energy balance in region 2:

\[
F_i = F_e - F_{er}
\]  
(2.52)

where

\[
F_i = \text{wave energy flux per unit width that reaches the perched profile} \quad [\text{Nm/m/s}]
\]
\[
F_e = \text{amount of wave energy flux per unit width reaching the structure} \quad [\text{Nm/m/s}]
\]
\[
F_{er} = \text{wave energy flux per unit width reflected on the structure} \quad [\text{Nm/m/s}]
\]

Assuming linear shallow water wave theory, constant breaker to depth ratio and that only the oscillatory part of the wave contributes to the reflected flux of energy, this can be rewritten as:

\[
h_i = h_e \left(1 - R^2 \right)^{2/5}
\]
(2.53)

where

\[
h_i = \text{depth at the shoreward side} \quad [\text{m}]
\]
\[
h_e = \text{depth at the seaward side} \quad [\text{m}]
\]
\[
R = \text{reflection coefficient} \quad [-]
\]

\[
R = \frac{H_r}{H_e}
\]
(2.54)

The reflection coefficient \( R \) is given as a function of the dimensionless breakwater crest width, \( B/L \) for different values of the dimensionless water depth, \( d/h_e \). Since \( R \) depends on \( h_i \) an iterative process must be carried out to solve \( R \) and \( h_i \). Using this procedure, a relation is plotted between the water depth ratio \( h_i/h_e \) and the dimensionless water depth \( d/h_e \) from which it can be concluded that for dimensionless water depths \( d/h_e \) greater than 0.5 minor benefits are achieved. Only for \( d/h_e \) less than 0.1 considerable reduction in \( h_i/h_e \) is obtained.
Laboratory data from Chatham (1972) and Sorensen and Beil (1988) were used to conclude that this equation gives an adequate estimation of the equilibrium water depth for a perched beach.

For waves breaking on a reef, shelf or wide breakwater, the spilling wave breaking assumption with a constant wave height to depth ratio $\gamma$ is not valid. Muñóz-Pérez et al. (1999) derived a simple relationship between the shape parameter for reef-protected beaches $A_{rp}$ and for non-protected beaches $A_r$, as defined by Dean, see Equation (2.19). Figure 2-16 shows a definition sketch of protected and non-protected profiles.

\[ \left( Ec_g \right)_{hr} = \int Dhdx \]  \hspace{1cm} (2.55)

Assuming linear wave theory and the fact that Equation (2.55) is valid along the entire profile yields:

\[ \left( \frac{H_{rp}}{H} \right)^2 = \left( \frac{W_{rp}}{W} \right) \]  \hspace{1cm} (2.56)

where

- $H$ = wave height [m]
- $W$ = total length of the profile [m]
- $(\ )_{rp}$ = subscript to indicate the reef-protected beach [-]

Since $H_{rp}$ is less than $H$ at the same depth, the total length of the profile for the reef-protected beach will also be less and consequently, the beach will be steeper. With the breaker to depth ratio $\gamma = H/h$, Equation (2.56) can be rewritten to Equation (2.57).

\[ W_{rp} = W \left( \frac{\Gamma}{\gamma} \right)^2 \]  \hspace{1cm} (2.57)

where

- $\Gamma$ = breaker to depth ratio for reef-protected beaches, e.g. according to Fredsøe and Deigaard (1992), see Equation (2.58). [-]

\[ \Gamma = \frac{H}{h_r} = 0.5 + 0.3e^{0.11 \frac{h_r}{H_{rp}}}, \text{ varying between } 0.55 \text{ to } 0.35 \text{ (Nelson, 1994)} \]  \hspace{1cm} (2.58)

$\gamma$ = breaker to depth ratio for unprotected beaches [-]
Introducing Equation (2.58) in Equation (2.57) leads to a relationship between the non-protected and reef-protected shape parameters.

\[
\frac{A}{A_{rp}} = \left( \frac{\gamma}{\Gamma} \right)^{4/3}
\]  \hspace{1cm} (2.59)

where

- \( A \) = shape parameter for non-reef-protected beaches
- \( A_{rp} \) = reef-protected beach shape parameter

The proposed model is verified by 50 reef profiles. For every measured profile, the best-fitted \( A_{rp} \) value is determined, as in Figure 2-17.

Figure 2-17  \hspace{1cm} \text{Comparison between measurement and expression for protected and non-protected profiles (Muñoz-Pérez et al., 1999)}

Figure 2-18 shows a comparison between the best-fitted values of \( A_{rp} \) and predicted values of \( A_{rp} \) using Equation (2.59).

Figure 2-18  \hspace{1cm} \text{Comparison for best fitted values of} \ A_{rp} \ \text{and predicted values of} \ A_{rp} \ \text{using} \ \text{Equation} \ (2.59) \ \text{(Muñoz-Pérez et al., 1999)}
The above is still all based on the assumption of a constant breaker-to-depth ratio. However, this relationship is not adequate in many cases. Larson and Kraus (1989) modified this assumption, by means of Dally’s et al. (1985) dissipation model, Equation (2.60), so wave breaking is not restricted to spilling breakers.

\[ D = \frac{K}{h^2} (F - F_s) \]  

(2.60)

where

- \( K \) = length scale over which the wave energy flux is reduced from its value during breaking, \( F \), to a stable value \( F_s \). [m]
- \( F \) = energy flux during breaking, see Equation (2.61) [W/m]
- \( F_s \) = stable energy flux, see Equation (2.62) [W/m]
- \( \Gamma \) = breaker to depth ratio for reef-protected beaches [-]

An important conclusion stated by Muñoz-Pérez et al. (1999) is that the shape parameter cannot be represented, in all cases, by a simple function of the sediment grain size or fall velocity as proposed by Moore (1982) or Dean (1987). The underlying geology plays an important role in the profile shape. It is remarkable that the hypothesis of uniform dissipation per volume, which leads to the 2/3-power profile, is still consistent for reef-protected beaches.

### 2.4.3 Advantages and disadvantages of the perched beach concept

Possible advantages of perching a beach are:
- Reduction of beach fill material, compared to nourishment only, see Figure 2-19.
- Submerged sills may retain the beach material, where sole nourishment does not resist offshore transport.
- It can be less expensive compared to other beach protections in shallow water, especially when there is a scarcity of sediment.
- This concept can create recreation beaches where this is normally not possible by merely nourishment. E.g. the island of Hawaii, with steep rock shorelines or in situations with little physical space, e.g. by a navigational channel as in IJmuiden.
- The sill concept, see §2.4.1, does not radically modify the near-shore wave environment as some other coastal defence structures do.
- The perched beach concept is aesthetically more pleasant, compared to other hard coastal defence structures, as it is not visible from the beach, see Figure 2-20.
- The use of stone will encourage settlement of several aquatic species, De Ruig and Roelse (1992-a,b).
- In strongly erosive areas such as gully outer bends, where onshore transport hardly takes place in the first place, the negative effect of blocking off the onshore transport is cancelled out. The use of a submerged barrier can only contribute positively, see De Ruig and Roelse (1992-a,b).

![Perched beach on a Caribbean island](source: internet)

Possible disadvantages of the perched beach concept are:
- Development of scour holes at either side of the sill.
- Possible danger to swimmers due to a sudden change in water depth and sharp obstacles, where sole nourishment does not.
- Possible danger to vessels due to a sudden change in water depth, where sole nourishment does not.
- Blocking of the natural onshore sediment transport during fair weather periods, where sole nourishment does not.
- There is a possible impact on the adjacent coast compared to sole nourishment. E.g. water level set-up differences and interruption of the net alongshore transport. Albeit, this is hard to predict.
- There is little quantitative, documented experience and information on which to base a design or an economic comparison with other types of coastal structures, where for other intervention the outcome is better understood. Therefore, the perched beach may have an adverse effect. E.g. the submerged breakwater project in Palm Beach Florida was intended to increase the shore protection, but actually the erosion rate was measured 2.3 times higher and the structure was removed, Browder et al. (1996) and Dean et al. (1997).

### 2.4.4 Inventory of perched beaches

In this paragraph, an inventory is given of perched beaches around the world and their qualities. First, Table 2-1 gives an inventory of studies on perched beaches. Most of these studies are described more
CHAPTER 2. Literature study

We refer extensive study in Appendix D (physical studies) and Appendix E (mathematical studies). Second, Table 2-1 gives an overview of actual built perched beaches and their properties. Despite all efforts, not all quantities were available in literature. The performance of most of these perches beaches is extensively described in Appendix F. For the lion’s share of the physical studies, winter conditions determined the erosion. The most effective measures to reduce sand loss or to reduce effects on the waterline are to adapt the initial profiles to the winter profile, to use adaptive beach fill material and to reduce the wave attack.

Table 2-1  Studies involving perched beaches

<table>
<thead>
<tr>
<th>Location</th>
<th>Year</th>
<th>Length (m)</th>
<th>width sill-shore (m)</th>
<th>Sill Height</th>
<th>Crest width (m)</th>
<th>Seabed level at sill (m)</th>
<th>Beach slope</th>
<th>Grain size (mm)</th>
<th>Tidal range (m)</th>
<th>Wave climate</th>
<th>remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Santa Monica (USA)</td>
<td>1968</td>
<td>600</td>
<td>230</td>
<td>MLLW-7.5</td>
<td>-7.5</td>
<td>30, incl stone apron</td>
<td>1:70</td>
<td></td>
<td></td>
<td>test with swl: MLLW-0 and +1.6</td>
<td>5.4</td>
</tr>
<tr>
<td></td>
<td>1973</td>
<td>100</td>
<td>40</td>
<td>MLLW-3 to -16</td>
<td>5 test depths, MLLW-6 to -18</td>
<td>D$_{178}$= 0.4</td>
<td></td>
<td></td>
<td></td>
<td>H: 1.4 m T: 6.19 s</td>
<td>WES 1:30 scale model</td>
</tr>
<tr>
<td>Niigata West Coast (Japan)</td>
<td>1987</td>
<td>1500</td>
<td></td>
<td>MSL-2.0</td>
<td>40, mound breakwater</td>
<td>MSL-8.0</td>
<td>1:50</td>
<td>D$_{178}$= 0.25</td>
<td>no tide, natural fluctuation 1.8m</td>
<td>H: 1.4 m T: 6.4 s</td>
<td>impermeable jetty’s every 400 m</td>
</tr>
<tr>
<td>Flathead Lake, Montana (USA)</td>
<td>1991</td>
<td>900</td>
<td>8</td>
<td>-1</td>
<td>2</td>
<td>-2</td>
<td>1:12:5</td>
<td>5-25</td>
<td>3.7</td>
<td>H: 0.75-1.25 m T: 3.5-4.5 s</td>
<td>gravel beach</td>
</tr>
<tr>
<td>Tenhondondpolder, Zeeuws Vlaanderen (the Netherlands)</td>
<td>1991-1992</td>
<td>860</td>
<td>175-225</td>
<td>MSL-0.5 to -1.0</td>
<td>MSL-7.0 to -9.0</td>
<td>1.60 to 1.80</td>
<td>0.65</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flemish coast, Knokeke (Belgium)</td>
<td>1998</td>
<td>800</td>
<td>230</td>
<td>MSL-3.0 m</td>
<td>MSL-3.0 m</td>
<td>1.35 to 1.40</td>
<td>4 m with spring tide</td>
<td>gravel sill 40% reduction of losses</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Eastern Beach, Gibraltar</td>
<td>2000</td>
<td>520</td>
<td>150</td>
<td>MSL-0.72 CD-0.20</td>
<td>15</td>
<td>MSL-4.0</td>
<td>1:70</td>
<td>0.220</td>
<td>0.65</td>
<td>H$_{s}$ 3.5 m (1/yr, 10 m)</td>
<td>bottom protect ion 10 m at seaside</td>
</tr>
<tr>
<td>Rubble Tip Area, Gibraltar</td>
<td>2000</td>
<td>520</td>
<td>125</td>
<td>MSL-0.72 CD-0.20</td>
<td>15</td>
<td>MSL-6.5</td>
<td>1:65</td>
<td>0.350</td>
<td>2.8</td>
<td>H$_{s}$ 3.5 m (1/yr, 10 m)</td>
<td>bottom protect ion 10 m at seaside</td>
</tr>
<tr>
<td>Marina East (Singapore)</td>
<td>2000-2002</td>
<td>600</td>
<td>100-150</td>
<td>-2.0 to -3.5</td>
<td>-15 to -20</td>
<td>-10 to -1.20</td>
<td>1:10 to 1.20</td>
<td>pocket beach</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Emmen (the Netherlands)</td>
<td>2002- now</td>
<td>250</td>
<td></td>
<td>NAP-2.0 m</td>
<td>NAP-4.0 to -9.0</td>
<td>1:30</td>
<td></td>
<td></td>
<td>sheet piles</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td>Year</td>
<td>Length (m)</td>
<td>width sill-shore (m)</td>
<td>Sill Height</td>
<td>Crest width (m)</td>
<td>Seabed level at sill (m)</td>
<td>Beach slope</td>
<td>Grain size (mm)</td>
<td>Tidal range (m)</td>
<td>Wave climate</td>
<td>remarks</td>
</tr>
<tr>
<td>-------------------------------</td>
<td>------</td>
<td>------------</td>
<td>----------------------</td>
<td>-------------</td>
<td>-----------------</td>
<td>--------------------------</td>
<td>-------------</td>
<td>-----------------</td>
<td>-----------------</td>
<td>---------------</td>
<td>--------------------------------</td>
</tr>
<tr>
<td>Cape Cod, 4 locations</td>
<td>1978</td>
<td>150-300</td>
<td>MLW 0.6 to +2.1</td>
<td>15</td>
<td>MLW 0 to +1.5</td>
<td>1.2 to 3.0</td>
<td>1 to 1.8</td>
<td>nylon sand bags</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Marina East (Singapore)</td>
<td>1978</td>
<td>3000</td>
<td>77</td>
<td>-0.3</td>
<td>-8.0 to -10.0</td>
<td>1:30 to 1:50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slaughter Beach, Delaware</td>
<td>1979</td>
<td>300</td>
<td>150</td>
<td>MLW 0</td>
<td>-1.2</td>
<td>1:40 to 1:50</td>
<td>Coarse sand</td>
<td>1.5</td>
<td></td>
<td></td>
<td>great alongshore transport</td>
</tr>
<tr>
<td>Emilia Romagna (Italy)</td>
<td>1982</td>
<td>200</td>
<td>75</td>
<td>MSL -1.5</td>
<td>-0.0 to -6.0</td>
<td>1:7 to 1:10</td>
<td></td>
<td>sand filled textile bags (1m³)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Keino-Matsuura (Japan)</td>
<td>1983</td>
<td>80</td>
<td>200</td>
<td>-2.0</td>
<td>-5.0 to -6.0</td>
<td>1:7 to 1:10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>no end groins</td>
</tr>
<tr>
<td>Caorle, Veneto (Italy)</td>
<td>1985</td>
<td>1300</td>
<td>10</td>
<td>MSL -0.2</td>
<td>1.8</td>
<td>MSL -2.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flathead Lake (USA)</td>
<td>1988</td>
<td>110</td>
<td>90</td>
<td>-1.0</td>
<td>1.5</td>
<td>1:10</td>
<td>3m lake level fluctuation</td>
<td>gravel beach, no groins</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lido di Ostia, zone B (Italy)</td>
<td>1989-1991</td>
<td>3000</td>
<td>-1.5 to -2.0</td>
<td>15</td>
<td>-4.0 to -5.0</td>
<td>1:36</td>
<td>0.3 to 1.3</td>
<td>D₅₀= 0.6</td>
<td>&lt; 0.5</td>
<td>Deep water, Hₛ=5m</td>
<td>Connected to end groins at river fibre</td>
</tr>
<tr>
<td>Lido di Ostia, zone C (Italy)</td>
<td>1992</td>
<td>700</td>
<td>MSL -0.5</td>
<td>MSL -3.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No artificial nourishment</td>
<td></td>
</tr>
<tr>
<td>Oostende (Belgium)</td>
<td>1996</td>
<td>174</td>
<td>-2.5 to -3.0</td>
<td>-17 to -20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Southern Island (Singapore)</td>
<td>2002</td>
<td>800</td>
<td>MSL -3.0</td>
<td>1:18</td>
<td></td>
<td>2.8</td>
<td>pocket beach Bay opening 270 m</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Riccione and Cesenatico (Italy)</td>
<td>2000</td>
<td>5000</td>
<td>200</td>
<td>MSL -2.5</td>
<td>MSL -6.0</td>
<td>Sand filled textile bags (1m³)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kemigawa Beach, Tokyo Beaches (Japan)</td>
<td>1980-1991</td>
<td>1300</td>
<td>MSL -1.8</td>
<td>MSL -3.0</td>
<td>1:20</td>
<td>0.160</td>
<td>Scawall SWL= 5.0, 2 arc shaped end groins, 400m long</td>
<td>groins every 350m Sand bags (2m³)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ravenna (Italy)</td>
<td>1992</td>
<td>4000</td>
<td>MSL 0</td>
<td>50</td>
<td></td>
<td></td>
<td>mild wave climate</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alameda Beach, San Francisco (USA)</td>
<td>1993</td>
<td>2000</td>
<td>MSL 0</td>
<td>50</td>
<td></td>
<td></td>
<td>Longard Tubes, above the breaker zone (1.8 m diameter)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The general conclusion that can be drawn from the perched beaches described above is that the performance is unsatisfactory. This is due to excessive loss of beach fill over a short duration. Most of the time, the alongshore transport seemed to be the main cause of the beach erosion.
3 Description of the modelling system

3.1 Introduction

The morphological development of the coast is submissive to complex interaction between currents, waves, sediment transport and bed level variations. Besides this, almost every coastal area is submissive to human interference in the form of structures, dredging or nourishment. During the past decades, so-called process-based morphodynamic models have been developed to simulate these processes and their interactions and to predict the morphological development. Figure 3-1 shows the concept of most morphological models. The essence of these models is the simulation of physical processes, based on primary physical principles, such as conservation of mass, momentum and energy.

![Figure 3-1 Concept of morphological process based models](image)

In this study, a morphodynamic model called Delft3D is used. Delft3D is a software package developed by WL Delft Hydraulics to simulate two-dimensional or three-dimensional natural processes. The Delft3D system structure is shown in Figure 3-2. Several phenomena and their interactions can be simulated in space and time. These phenomena are integrated in eight different modules:
- A hydrodynamics module (Delft3D-FLOW)
- A wave module (Delft3D-WAVE)
- A water quality module (Delft3D-WAQ)
- A particle tracking module (Delft3D-PART)
- A ecological module (Delft3D-ECO)
- A sediment transport module (Delft3D-SED)
- A chemical components module (Delft3D-CHEM)
- A morphodynamic simulation module (Delft3D-MOR)
In this study only the FLOW, WAVE and MOR modules are used and they are described in the next paragraphs.

3.2 Delft3D-MOR

Delft3D-MOR is a steering module that controls and integrates the effects of different processes, such as waves, currents and sediment transport on morphological developments. Each of these processes is dealt with in separate modules. The steering module allows the user to link model inputs and outputs for the model components. The morphological process is modelled as a hierarchical tree structure of processes, as shown in Figure 3-3. Time intervals for the elementary processes are defined and the processes are executed for a fixed number of time steps.

In this study, FLOW and WAVE are operated sequentially, but they are using each other’s results. Instead of also using the transport and bed level variation module the sediment add-on within the FLOW module is used, the “Sediment Online” version. The steering module MORSYS calls the computational modules in a prescribed order, arranges the time process of each module and allows iterations between the modules.

The Delft3D software package is constantly developing. For this study, the 09b version of the Morsys-executable is used. For a more elaborate description of the MOR module, one is referred to Delft3D-MOR user manual (WL | Delft Hydraulics, 2001-c).
3.3 Delft3D-WAVE

The WAVE module is based on either HISWA or SWAN. In this study, SWAN is used for wave simulation. SWAN, an acronym for Simulating WAves Nearshore, is a third generation, spectral wave model that computes the non-steady propagation of short crested waves over an uneven bottom, considering wind action, dissipation due to bottom friction, wave breaking, refraction, shoaling and directional spreading.

The SWAN model takes into account the following physics:
- Wave propagation in time and space, shoaling, refraction due to current and depth, frequency shifting due to currents and non-stationary depth;
- Propagation from laboratory up to global scales;
- Wave generation by wind (not used in this study);
- Dissipation by white-capping, depth-induced breaking and bottom friction;
- Non-linear wave-wave interactions;
- Wave induced set-up;
- Wave blocking by flow;
- Transmission through and reflection from obstacles.

SWAN does not account for:
- Diffraction;
- Scattering reflections.

SWAN computations can be made on a regular and a curvilinear grid in a Cartesian or spherical coordinate system.

The wave conditions (i.e. wave forces based on the energy dissipation rate or the radiation stresses, orbital velocity) calculated in the Delft3D-WAVE module are used as input for the Delft3D-FLOW module, to compute wave driven currents, enhanced turbulence, bed shear stress and stirring up by wave breaking.

For a short description of the SWAN module, one is referred to Appendix G. For a more elaborate description of the Delft3D-WAVE module, one is referred to the Delft3D-WAVE user manual (WL | Delft Hydraulics, 2001-b).

3.4 Delft3D-FLOW

The Delft3D-FLOW module is a multidimensional hydrodynamic simulation program, which calculates non-steady flow and transport phenomena, resulting from tidal and meteorological forcing on a rectangular or a curvilinear, boundary fitted grid. The model can be used for both two-dimensional (depth-averaged) or three-dimensional calculations on coastal, river or estuarine areas where the horizontal length and time scales are significantly larger than the vertical scales. Typical applications of Delft3D-FLOW are simulations of tide and wind driven flows, stratified and density driven flows, river flow, transport of dissolved material and pollutants.

The following physical phenomena are taken into account:
- Tidal forcing;
- The effect of the earth’s rotation (Coriolis force);
- Density driven flows;
- Space and time varying wind and atmospheric pressure;
- Shear-stresses at the bottom;
- Wave induced stresses (radiation stress) and mass fluxes;
- Drying and flooding;
- Turbulence induced mass and momentum fluxes.
Delft3D-FLOW solves the Navier-Stokes equations for an incompressible fluid, under the shallow water and Boussinesq assumption. In the vertical momentum equation the vertical accelerations are neglected, which leads to the hydrostatic pressure equation. The system of partial differential equations for conservation of mass and momentum is solved with a finite difference method on a rectangular, orthogonal curvilinear or spherical grid. The principle variables, such as water level, bottom level and velocities, are arranged in a special way that is known as the staggered grid, see Figure 3-4.

![Figure 3-4 The Delft3D staggered grid](image)

In this study, the continuity equation and the equation of motion are solved in a quasi three-dimensional modelling system. First, the depth averaged continuity and momentum equations are solved. Secondly, the water depth is divided in a number of layers and again the simplified equations as in the first step are applied to those depth layers.

For a short description of the FLOW module, one is referred to Appendix H. For a more elaborate description of the FLOW module, one is referred to the Delft3D-FLOW user manual (WL | Delft Hydraulics, 2001-a).

### 3.5 Sediment Online

This add-on concerns the use of Sediment Online transport morphology in Delft3D-FLOW. There is a feedback of the bottom changes to the hydrodynamic computation and the influence of waves can be taken into account.

The three-dimensional transport of suspended sediment is calculated by solving the three-dimensional advection-diffusion equation for the suspended sediment (mass-balance). For the sediment dispersion the $\kappa$-$\varepsilon$ turbulence model is used.

For a short description of Sediment Online, one is referred to Appendix I. A more extensive description of Delft3D-FLOW and its transport computation can be found in the user manual (WL | Delft Hydraulics 2001-a) and Stelling (1984). It has to be stressed that for this study a development version of Sediment Online is used that is not used for official studies by WL | Delft Hydraulics yet.
3.6 Modelling of a submerged breakwater in Delft3D

3.6.1 Introduction
In both the WAVE and the FLOW module of Delft3D, modelling a submerged breakwater requires adaptations. This is necessary for SWAN, because the bottom grid is often too coarse to resolve the small cross-sectional breakwater area and when the grid is fine enough, the results differ from expectations, see §4.4.1. For FLOW, adaptations are necessary, because for normal computations the breakwater would erode. For sake of simplicity, a uniform bathymetry in alongshore direction is assumed, and we look at the breakwater in cross-sectional perspective.

3.6.2 SWAN
In general, a line structure, such as a breakwater, will affect the wave field in two ways; first, it will reduce the wave height, and second it will cause diffraction around its ends. The model is not able to account for diffraction. In irregular, short crested wave fields, however, it seems that the effect of diffraction is small, except in a region less than one or two wavelengths away from the tip of an obstacle. Therefore, the model can reasonably account for waves around an obstacle if the directional spectrum of incoming waves is not too narrow. Seeing that this study became limited to calibration of a wave flume only, diffraction does not matter.

Since obstacles usually have a cross-sectional area that is too small to be resolved by the bottom grid in SWAN, an obstacle is modelled as a line. There are two ways to simulate the obstacle; first, one can implement a ‘dam’. Here the crest level is defined and the transmission coefficient is determined by Goda’s expression, see Equation (2.1) and (3.1). This means that the transmission coefficient varies with the water level. A minor drawback is the fact that the formulation of Goda does not account for the crest width, through which the transmission is overestimated for wide crests (>5 m), see Appendix K.

\[
K_i = 0.5 \left[ 1 - \sin \left( \frac{\pi}{2\alpha} \left( \frac{R_c}{H_i} + \beta \right) \right) \right], \quad \text{for } -2.45 \leq \frac{R_c}{H_i} \leq 2.45
\]  

(3.1)

where
\[
\alpha = \text{empirical coefficient, depending on the breakwater shape} \quad [-]
\]
\[
\beta = \text{empirical coefficient, depending on the breakwater shape} \quad [-]
\]
\[
R_c = \text{freeboard} \quad [\text{m}]
\]
\[
H_i = \text{incoming wave height} \quad [\text{m}]
\]

The other possibility to simulate a submerged structure is to enforce the transmission coefficient by a so-called ‘sheet’. Here a transmission coefficient is imposed between two grid cells. A potential disadvantage of an imposed transmission coefficient is that it does not change with a different water level, e.g. with a tide. The great advantage of the imposed transmission coefficient is the adjustability of the transmission to the user’s wishes.

In this study, with steady water levels, the latter possibility to enforce the wave transmission is used, viz. the imposed transmission coefficient by the use of ‘sheets’, as the imposed transmission coefficient is nominated in Delft3D-WAVE. From now on, the term ‘sheet’ refers to this imposed transmission coefficient.

Since no data are available on oblique incoming waves, the model assumes that the transmission coefficient does not depend on direction. Another phenomenon that is to be expected is a change in wave frequency since often the process above the dam is highly non-linear. Again there is little information available, so in SWAN it is assumed that the frequencies remain unchanged over an obstacle (only the energy scale of the spectrum is affected and not the spectral shape).
To have more influence on the transmission, dissipation by bottom friction and breaking have to be as small as possible and therefore another bathymetry without breakwater is used. To overcome the difference in depth at both sides of the breakwater, the depth input can either be first more horizontally and landward of the obstacle line more steep or it can be gently sloping from seaward toe to the landward toe, see Figure 3-5. Advantage of the first option is that most of the energy is dissipated by the imposed obstacle and it is therefore more verifiable, and will therefore be used.

![Figure 3-5](image_url) Horizontal and sloping SWAN bathymetries

Now, it can be expected that the enforced transmission releases itself as expected and the calculated transmission is equal to the imposed transmission.

For a correct computation, besides the water level, the updated bathymetry that is calculated in the FLOW module has to be used for SWAN computations. Near the breakwater, one still wants to use the initial horizontal bathymetry. Implementing a land boundary file, called ‘wavemask.ldb’, resolves this. Within the co-ordinates of the land boundary file, the initial SWAN depth is used, i.e. the horizontal bathymetry instead of the updated FLOW bottom, see Figure 3-6. Outside the land boundary, the bottom is used that is calculated by the FLOW module and stored in the communication file.

![Figure 3-6](image_url) Breakwater cross-section with FLOW and SWAN bathymetry

In top view perspective, the SWAN and FLOW bottom and computational grids are located as in Figure 3-7. Because of boundary disturbances, the computational SWAN grid needs to be larger than the area of interest that equals the FLOW bottom and computational grid. If the SWAN bottom grid would be as large as the SWAN computational grid, the bathymetry would only change at the same locations as the FLOW grid. This would result in alongshore discontinuities. Therefore, the SWAN bottom grid is smaller than the FLOW bottom grid. Within the SWAN bottom grid, the bathymetry is used from the FLOW calculations (except within the wavemask land boundary). Outside the SWAN bottom grid the depth value is used of the nearest known depth value. This means that the known values are extended in alongshore direction. In Figure 3-7, this is illustrated by a line with equal depth values.
Then the obstacles, in this case sheets, can be allocated. Locating the obstacle line has to be done from grid point to grid point. Therefore, the resolution of transmission is equal to the computational grid spacing. One can choose the number of sheets, their transmission values and the place on the breakwater. They all have an influence on the hydrodynamics.

**Number of sheets**
To begin with the first alternative, one can choose the number of sheets. One enforced transmission can obtain the same calculated transmission as a series of sheets. See Figure 3-8 and Figure 3-9.
Figure 3-8  Wave transmission with one sheet

Still, one sheet does not simulate any width of the breakwater. Energy dissipation takes place within a few grid cells. A series of sheets can reproduce a more realistic image, see Figure 3-9. The number of sheets is limited by the number of SWAN grid cells and their location is between two SWAN grid cells.

Figure 3-9  Wave transmission with nine sheets

Transmission values
When using a series of sheets, the resulting product of transmission coefficients should equal the measured transmission coefficient or the transmission coefficient calculated with a relation that accounts for the crest width. Every coefficient value can be determined with the use of the local depth, water level and incoming wave height. Though still, the overall transmission coefficient, i.e. the product of each individual transmission coefficient, can differ from the coefficient to be calibrated. Therefore, adaptation of each individual coefficient can be needed by adding a proportional value, bearing in mind the absolute value one. In §4.4.1 the method of Goda (Goda, 1967) is used to determine each individual transmission coefficient.
CHAPTER 3. Description of the modelling system

Sheet location
In reality, there is a distance between breaking and plunging, the latter causing high turbulence. Locating the sheets more onshore for a distance advised by or Basco (1985), Nairn et al. (1992) or Sawaragi (1995) can simulate this transition zone width, see §2.2.2. The sheet location can also regulate the wave set-up significantly. For example, placing nine transmission coefficients instead of five means that the individual transmission values must be higher to obtain the same overall transmission. The value in combination with the local depth determines the set-up. After all, this can be used to fine-tune the wave set-up over the submerged breakwater.

3.6.3 FLOW and Sediment Online
For the FLOW module, simulation of a submerged breakwater needs a spatial resolution high enough for a few grid cells to cover the breakwater. A big difference for the sediment transport is that at the breakwater non-erodible material has to be simulated. By specifying zero available sediment at the breakwater, the bottom is fixed. In Figure 3-10, a part of a cross-section is shown, together with a figure showing the available sediment in kg/m². In this case, 4000 kg/m² means an erodible layer thickness of 2.50 m.

![Figure 3-10](image-url)  
*Breakwater cross-section depth (m) and available sediment (kg/m²)*
4 Model calibration

4.1 Introduction
In this chapter the mathematical model will be calibrated by means of flume measurements conducted for a perched beach in Gibraltar, see Appendix D. For reasons of demarcation only the Eastern Beach tests of the final stage are simulated, though for three different wave conditions. In §4.2 the results of the physical tests are explained. Paragraph 4.3 explains the mathematical model set-up. The actual calibration of several parameters is explained in §4.4, followed by some validity checks in §4.5. This chapter is ended with comments one has to take notice of when comparing mathematical and physical models, §4.6.

4.2 Results of the physical tests
For the Gibraltar Beach development project, several tests are conducted for two locations, see WL | Delft Hydraulics 2000-a,-b,-c,-d,-e. Here only the Eastern Beach tests are simulated from the last stage (WL | Delft Hydraulics, June 2000-e).

Test sequence
Fourteen times, three different wave conditions were carried out. The name, boundary wave condition, sequence and duration of the fourteen executed tests in prototype scale values are shown in Table 4-1. During the calibration, the same sequence and duration as in this table will be simulated.

<table>
<thead>
<tr>
<th>Test name</th>
<th>Wave Condition</th>
<th>H, at MSL-10.5 m (m)</th>
<th>Duration (hr)</th>
<th>Duration (hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T102</td>
<td>1</td>
<td>1.5</td>
<td>0-12</td>
<td>12</td>
</tr>
<tr>
<td>T103</td>
<td>3</td>
<td>3.5</td>
<td>12-18</td>
<td>6</td>
</tr>
<tr>
<td>T105</td>
<td>1</td>
<td>1.5</td>
<td>18-30</td>
<td>12</td>
</tr>
<tr>
<td>T106</td>
<td>2</td>
<td>2.5</td>
<td>30-36</td>
<td>6</td>
</tr>
<tr>
<td>T108</td>
<td>1</td>
<td>1.5</td>
<td>36-48</td>
<td>12</td>
</tr>
<tr>
<td>T109</td>
<td>3</td>
<td>3.5</td>
<td>48-54</td>
<td>6</td>
</tr>
<tr>
<td>T110</td>
<td>1</td>
<td>1.5</td>
<td>54-60</td>
<td>6</td>
</tr>
<tr>
<td>T111</td>
<td>1</td>
<td>1.5</td>
<td>60-66</td>
<td>6</td>
</tr>
<tr>
<td>T112</td>
<td>3</td>
<td>3.5</td>
<td>66-72</td>
<td>6</td>
</tr>
<tr>
<td>T113</td>
<td>1</td>
<td>1.5</td>
<td>72-85.5</td>
<td>13.5</td>
</tr>
<tr>
<td>T114</td>
<td>2</td>
<td>2.5</td>
<td>85.5-91.5</td>
<td>6</td>
</tr>
<tr>
<td>T115</td>
<td>2</td>
<td>2.5</td>
<td>91.5-97.5</td>
<td>6</td>
</tr>
<tr>
<td>T116</td>
<td>2</td>
<td>2.5</td>
<td>97.5-103.5</td>
<td>6</td>
</tr>
<tr>
<td>T117</td>
<td>2</td>
<td>2.5</td>
<td>103.5-109.5</td>
<td>6</td>
</tr>
</tbody>
</table>

Table 4-1 Test sequence and wave condition and duration per test

In this paragraph mean characteristic prototype values for the three different wave conditions are shown. For the three wave conditions, water levels and wave heights are measured in four locations and velocities are measured at two locations, as can be seen in Figure 4-1. For all fourteen individual tests, the values can be seen in Appendix J, Appendix L and Appendix M.
The first location is offshore, the second just in front of the breakwater, the third just behind the breakwater and the fourth is located between the breakwater and the shoreline.

Wave conditions
For the three conditions, wave height and peak period are measured, from which the transmission can be calculated. Table 4-2 shows the mean values per wave condition. For all individual tests, one is referred to Appendix J.

The crest is 15 meter wide and the larger wave conditions ($H_s=2.5\text{ m}$ and $H_s=3.5\text{ m}$) seem to be affected by the 10 meter wide berm. Including the slopes, the submerged breakwater is 40 meter wide. The approaching wavelengths at the breakwater (location 2) are respectively 24, 30, and 35 meter for wave conditions 1, 2 and 3. These values are determined by SWAN computations.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Wave measurements</th>
<th>$K_{12,3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loc.1</td>
<td>Loc.2</td>
</tr>
<tr>
<td>1</td>
<td>$H_s$ (m)</td>
<td>1.60</td>
</tr>
<tr>
<td></td>
<td>$T_p$ (s)</td>
<td>5.5</td>
</tr>
<tr>
<td>2</td>
<td>$H_s$ (m)</td>
<td>2.54</td>
</tr>
<tr>
<td></td>
<td>$T_p$ (s)</td>
<td>7.0</td>
</tr>
<tr>
<td>3</td>
<td>$H_s$ (m)</td>
<td>3.33</td>
</tr>
<tr>
<td></td>
<td>$T_p$ (s)</td>
<td>8.5</td>
</tr>
</tbody>
</table>

Table 4-2  Measured wave heights and periods

The measured transmission coefficients differ a lot from the known models, see Appendix K. Generally, there are two kinds of models that reckon with the crest width. As can be seen in Appendix K, the Seabrook and Hall model (1998) approaches the measured values best.

Water level
The water levels are obtained from the mean signal of the wave measurements. In Table 4-3 one can see the still water level and four mean wave signals, i.e. relative to SWL. For all tests, one is referred to Appendix L.

<table>
<thead>
<tr>
<th>Condition</th>
<th>h-boundary (m rel. to MSL)</th>
<th>$\Delta h_1$ (m)</th>
<th>$\Delta h_2$ (m)</th>
<th>$\Delta h_3$ (m)</th>
<th>$\Delta h_4$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.4</td>
<td>-0.03</td>
<td>-0.04</td>
<td>0.04</td>
<td>0.04</td>
</tr>
<tr>
<td>2</td>
<td>0.5</td>
<td>-0.08</td>
<td>-0.09</td>
<td>0.16</td>
<td>0.16</td>
</tr>
<tr>
<td>3</td>
<td>0.6</td>
<td>-0.14</td>
<td>-0.14</td>
<td>0.25</td>
<td>0.27</td>
</tr>
</tbody>
</table>

Table 4-3  Water levels measured in the wave flume
It is obvious that within the wave flume there is conservation of mass. Wave set-up causes a lower water level at the wave paddle, see Figure 4-2. In the real world and in the mathematical model this does not happen and therefore the boundary water levels are adjusted.

![Figure 4-2](image)

**Figure 4-2 Adjustment of the water level boundary of wave condition 3**

In the prototype scale mathematical simulation, this water level is taken as reference level. E.g. for test condition 3, a still water level of 0.6 meter is simulated, but due to a 0.14 meter lower mean water level at the wave paddle during the physical tests, a water level boundary of 0.46 meter is used as input for the Delft3D model.

The mathematical computation will have to validate the water levels. In Table 4-4 the adjusted water level boundary and the measured water level at the four gauges relative to the adjusted water level boundary are displayed. In addition, the wave set-up at the breakwater, the difference between water level in location 2 and 3, is shown.

<table>
<thead>
<tr>
<th>Condition</th>
<th>h-boundary adjusted (m rel. to MSL)</th>
<th>$\Delta h_1'$ (m)</th>
<th>$\Delta h_2'$ (m)</th>
<th>$\Delta h_3'$ (m)</th>
<th>$\Delta h_4'$ (m)</th>
<th>Set-up$_{2,3}$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.370</td>
<td>0.00</td>
<td>-0.01</td>
<td>0.07</td>
<td>0.07</td>
<td>0.08</td>
</tr>
<tr>
<td>2</td>
<td>0.418</td>
<td>0.00</td>
<td>-0.01</td>
<td>0.24</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>3</td>
<td>0.455</td>
<td>0.00</td>
<td>0.00</td>
<td>0.39</td>
<td>0.41</td>
<td>0.39</td>
</tr>
</tbody>
</table>

**Table 4-4 Adjusted water level for in the mathematical study**

**Velocities**

Just behind the breakwater, velocity is measured at two depths; at mid-depth and just above the bottom, see Figure 4-3. Table 4-5 shows the mean values for each wave condition. For all individual tests, see Appendix M. At the first test runs velocity was not yet measured, but it is considered valid to assume that velocities for these test are more or less equal for the same wave conditions but a slightly changed bathymetry.
Sedimentation erosion patterns and offshore losses.
Figure 4-4 depicts the measured initial bathymetry of the first test and the end bathymetry of the last test of the sequence of tests, see Table 4-1. Some phenomena that stand out are, discussed in offshore direction:
- Some deposition above the water line. Probably this onshore sedimentation is overestimated in the physical model by model-effects (De Vroeg, personal communications);
- Substantial erosion just in front of the shore, resulting in a steep upper beach slope;
- A little bit sedimentation between the breakwater and the shore, forming a sort of bar;
- The measurements show a change of the hard toe of the breakwater. In fact, for the last tests, the toe is shortened on purpose;
- High deposition rates just seaward of the breakwater;
- Considering the end bathymetry, the beach is barely perched by the breakwater. The slopes on both sides of the breakwater seem to be little different and lie in the length of each other.

<table>
<thead>
<tr>
<th>Test condition</th>
<th>$H_s$ at MSL-10.5 m (m)</th>
<th>$T_p$ (s)</th>
<th>Set-up (m w.r.t. MSL.)</th>
<th>Offshore losses ($m^3$/m/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.5</td>
<td>5.5</td>
<td>0.4</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>7.0</td>
<td>0.5</td>
<td>0.03</td>
</tr>
<tr>
<td>3</td>
<td>3.5</td>
<td>8.5</td>
<td>0.6</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Table 4-6 Mean offshore losses per wave condition
4.3 Mathematical model set-up

4.3.1 Delft3D set-up

For the FLOW module, a grid is chosen with wider grid cells at the deeper boundary and smaller ones near the shore. The breakwater had to cover several grid cells. The grid is made so that three cells fit in the 15-meter wide crest. With a maximum smoothness (difference in cell width between two adjacent cells) of 5% at the offshore boundary 25-meter wide grid cells were possible. In alongshore direction, six cells are used to first simulate a more or less 2DV situation. The FLOW computational grid and bathymetry in cross-shore direction are shown in Figure 4-5. In cross-shore direction, 98 grid cells are used, varying from 5 to 25 meter. In alongshore direction six cells are used, each 25 m long, resulting in a 150 m wide beach. In vertical direction, 10 layers are used to simulate in three dimensions. Appendix O describes the individual, relative layer thickness as well as other FLOW input parameters in the input files. Offshore a water level boundary is enforced. The upper and lower boundaries are closed, to simulate the flume boundaries.

![FLOW grid and cross-shore bathymetry](image)

The bathymetry data set contains over 9000 points in cross-shore direction that had to be translated to the grid depth points. This is done by averaging the measured depth values around a depth point. Appendix N also shows the numerical simulation of the breakwater.

Concerning the sediment transport, the same sediment distribution as Figure 3-10 is used, viz. zero available sediment at the breakwater. For other transport parameters, one is referred to Appendix P and Appendix Q.
As described in §3.6.2, for SWAN the computational grid has to be large enough so that boundary disturbances do not exert influence on the area of interest. To make it easy to consider a wider beach at a later stage, the computational grid is chosen one kilometre wide. For the depth values, SWAN uses the computed FLOW depth values. Since the SWAN grid is wider, outside the FLOW grid the initial unchanged depth would be used for the SWAN computation. This is solved by using a smaller SWAN bottom grid than the FLOW grid. When no depth values are available, the nearest depth values are used, so the updated FLOW bathymetry reaches over the entire computational SWAN grid, see Figure 4-6.

Concerning the breakwater simulation, the same method is used as described in §3.6.2, but the exact values are given in §4.4. In Appendix R, the initial input parameters for the WAVE computation are shown in the SWAN input files, which are chiefly default values, except the directional spreading that is set on one degree to simulate long crested waves as in the flume.

Figure 4-6    SWAN and FLOW bottom- and computational grids
4.3.2 Overall set-up

The final model set-up has evolved from a simple to a complex model. Only the latter is discussed here. Before executing a run, a separate directory is created with three subdirectories, called ‘batch’, ‘mdf’ and ‘flow’. The batch subdirectory contains a batch file called ‘runcond.bat’. The ‘mdf’ subdirectory contains the general FLOW en WAVE input files and the FLOW subdirectory contains the attribute files for both FLOW and WAVE computations, see Figure 4-8.

The actual computation gets going by running a Matlab script, ‘run003.m’, see Appendix S and Figure 4-7. The use of Matlab is necessary to convert read data of a simulation to be used in the next simulation, which will be explained later. First, the Matlab working directory is switched to the main directory of the simulation. Two other subdirectories are made, called ‘run’ and ‘output’, see Figure 4-8.

From Matlab the batch file (‘runcond.bat’) is called, together with 28 specific parameters. First all the input and attribute files are copied to the ‘run’ subdirectory. Within the input files, the parameters are changed into values by the modification file called ‘mod.exe’. The subdirectory is now ready to execute.

A series of WAVE and FLOW computations is made, steered by the morsys executable. Hereby the WAVE module uses the currents, water level and bathymetry from the FLOW module and the FLOW module uses the results of the WAVE computation, both stored on a communication-file. At the end of this, the batch file creates a subdirectory in the output subdirectory with the name of the test condition as described in Table 4-1. The input and output files are moved from the ‘run’ subdirectory and stored in the specific output directory.

With the batch file being finished, Matlab continues executing the script. Another Matlab script is called upon that extracts the depth values in the water level points from the map output file and saves them in a depth file, ‘dsdd.m’, see Appendix S. In addition, the available sediment is extracted and saved to use in the next test condition.

For the next test condition, also an initial condition file is necessary, created by a Matlab file called ‘inifile.m’, see Appendix S. In this file, the water levels are stored. Now the cycle is completed for the first test. Again, the batch file is called upon, followed by the other Matlab script until the whole test series are executed as in the physical model, see Table 4-1. At the end, the main directory looks as in Figure 4-8.
4.4 Model calibration

In order to use the proper input parameters, the mathematical model is calibrated, verified and validated with the physical test results, conducted by WL | Delft Hydraulics. Before the Sediment Online add-on is activated, the hydrodynamics are calibrated. Three different wave conditions are distinguished. For the wave height, water levels and velocities the mean values are used to calibrate the hydrodynamics, as expressed in Appendix J, Appendix L and Appendix M.

One can distinguish several hydrodynamic aspects. In this study, the following aspects have been investigated:

- Wave transmission; due to the bathymetry and the submerged breakwater, incoming waves tend to shoal and break, discussed in §4.4.1 and §4.4.3;
- Water level; the wave force gradient at the submerged breakwater is equalised by a significant head difference, discussed in §4.4.2;
- Velocity field, discussed in §4.4.4; although no tide or currents are imposed, the waves and water level gradients tend to enforce a current. At the area of interest, the alongshore currents have to be negligible;
- With the best-obtained conditions, sediment transports are investigated, discussed in §4.4.5.

For the wave transmission calibration, water level calibration and velocity calibration a fixed bed is used, i.e. the initial bathymetry and de-activated bottom updating. Hereby it is assumed that the differences between the initial bathymetry and the mean bathymetry per wave condition would result in negligible differences. Another assumption is that the conditions are uniform in alongshore direction. This allows us to average all values in alongshore direction and to compare them with the measurements. Concerning the transport patterns, also a uniform profile in alongshore direction is assumed and therefore averaged cross-sections are compared. All these assumptions are verified in a later stage.

The next example clearly shows that a warm-up time is needed before the right wave heights and velocities are calculated. The WAVE and FLOW module use each other’s output for their own computation. The first WAVE computation uses a uniform water level without wave set-up and set-down, the blue line in the upper figure. These ‘wrong’ values (the red line in the lower figure) are used to compute the water level (red line in the upper figure) and currents in the FLOW module. Again a new wave field is computed which is used in the next FLOW computation (both green lines). From the third WAVE and FLOW computation on there are negligible changes. This is an important fact for calibration. Three iterations have to be made before the output is stable. Later bottom updating is therefore activated from the third iteration on. See Appendix P, in ‘morph’ input file: MORSTT = 30 (minutes after ITDATE to begin bed updating).
### 4.4.1 Wave transmission

The main obstacle to simulate is the submerged breakwater. When the same bathymetry for SWAN is used as for FLOW, the wave height decay looks as in Figure 4-10. This figure and the next four figures show the wave height development in cross-shore direction for three different wave conditions at the location of the breakwater, depicted in the lowest plot. Besides the graphs, the measured and calculated wave heights at locations two and three and the measured and calculated wave transmission are given in words. Back to Figure 4-10, using the same bathymetry for SWAN as for FLOW, no ‘wavemask.lbd’ file is used and no numerical obstacles are implemented as prescribed in §3.6. The transmission in this case gives unreliable values, just like the water level alteration, see Appendix T. The inconsistency of the transmission error, i.e. under- and overestimation, is probably due to different water levels since all the other parameters are equal. This example underscores the need for a different schematisation as described in §3.6.2 to simulate the wave transmission.
CHAPTER 5. Sensitivity analysis

Figure 4-10  Wave transmission for all conditions, with equal bathymetry for FLOW and SWAN

With SWAN, it is best to model an obstacle as a line since obstacles usually have a cross-sectional area that is too small to be resolved by the bottom grid. As described in §3.6.2 one can simulate an obstacle by a ‘dam’ (transmission coefficient is calculated for every water level for the imposed crest height) or a ‘sheet’ (imposed transmission coefficient). Since the water level does not change in time, the sheet option is used in this study.

The next figure, Figure 4-11, shows a single sheet with a horizontal deep bottom at the breakwater. Hereby the boundary water level is chosen so that in front of the breakwater the water level equals the measured value, see Appendix U.
Both the dam and sheet option do not consider the crest width. One could place several sheets after another with the same resulting transmission coefficient as measured in the tests. A series of transmission coefficients is calculated for all three wave conditions by means of an Excel spreadsheet. For every one of the nine SWAN grid points on the breakwater, the transmission coefficient is calculated with the formulation of Goda (Equation (2.1)). The transmission coefficient at one grid point is calculated from the local water level and bottom depth and the incoming wave height from the previous grid cell. Doing this resulted in transmission coefficients as depicted in Figure 4-12 and in an underestimated overall transmission. This deviation is not completely surprisingly, since every additional sheet will always reduce the overall transmission coefficient, but in this way a representative wave height decrease in space is obtained that only has to be adjusted.
To obtain the same wave reduction profile as measured, a value is added to every transmission value below one, proportional to the former deviating value. Now the same transmission can be obtained as measured, see Appendix V and Figure 4-13. It is clear to see that the smallest wave condition is not affected by the berm as the transmission coefficients of the first four sheets is one. Figure 4-13 also shows the water level values at location two and three, noted as $h_{loc.2}$ and $h_{loc.3}$. Although the right wave transmission is obtained, the water levels still differ from measurements. This will be discussed in the next paragraph.
Figure 4-13  Wave transmission for all conditions, using nine adapted sheet values
4.4.2 Wave set-up

With the right wave transmission obtained, there is still a slight difference in wave set-up, see Appendix V. Enforcing the wave reduction over a smaller distance can enlarge the set-up, see §2.2.4. This is because the radiation stress gradient ($\delta S_{xx}/dx$) exerts its force on a shallower or deeper part of the breakwater. The changed water level however has an influence on the transmission itself. This resulted in an iterative process of wave height and water level calibration by slight adjustments of the transmission coefficients. At the end, the water level at both sides of the breakwater can be the same as the measured values. For the used transmission coefficient and the wave height decay, see Figure 4-14. For the obtained water level development, one is referred to Appendix W.
4.4.3 Remaining wave propagation

With the right wave height reduction over the breakwater obtained, there is still a difference in wave propagation between the offshore boundary and the breakwater and between the breakwater and the shoreline, compared to the measurements, see Appendix W. This is corrected by adjustment of the bottom friction factor and the breaker parameter.

For the bottom friction factor $C_{\text{bottom}}$ see Equation (A.21) in Appendix G, the formulation of Jonswap is used. In literature, the Jonswap friction factor shows a great spreading, see Hasselmann et al. (1973).

The effect of a higher breaker parameter results in a higher wave height in the breaker zone, where as a higher friction coefficient results in a greater wave height reduction outside the breaker zone.

After some iteration the transmission, the wave set-up and the wave height decrease onshore can be obtained as measured, see Appendix W. The used values for the final bottom friction and the breaker parameter are shown in Table 4-7.

<table>
<thead>
<tr>
<th>Test condition</th>
<th>$H_{\text{sig}}$ (m)</th>
<th>Friction factor</th>
<th>Breaker parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.5</td>
<td>0.62</td>
<td>0.73</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>0.49</td>
<td>0.74</td>
</tr>
<tr>
<td>3</td>
<td>3.5</td>
<td>0.80</td>
<td>0.84</td>
</tr>
</tbody>
</table>

Table 4-7 Bottom friction factors according Jonswap and breaker parameters

The inconsistency between the bottom friction factor and the wave condition could be explained by the fact that during the calibration procedure the measured wave heights and water levels are averaged for every wave condition to be compared with computations with the initial bathymetry for every wave condition, see §4.4, page 44. For example, wave condition two is mainly executed at the end of the test series, see Table 4-1, and most of the wave height measurements took place with a different bathymetry. The mean bathymetry during this test condition differs from the initial bathymetry in a way that at the locations two, three and four sediment is settled, see Figure 4-15.

Concerning the breaker parameter the higher value for wave condition 3 is striking. This is probably also attributed to the difference between the initial bathymetry and the mean bathymetry for this wave condition.
4.4.4 Transition zone and velocities

To simulate the transition zone, see § 2.2.2, the series of sheets can be moved onshore, for one, two or three grid cells, see Figure 4-16.

This movement will have an effect on the wave set-up and so again by iteration this has to be adjusted. A marginal note needed to state here is that the length of the displacement of the sheets is limited by the desire to obtain the right wave set-up. Moving all sheets beyond the third water level gauge will obviously never result in a wave set-up that is measured at water level gauge number three, as the wave set-up is indirectly caused by the wave reduction by the sheets.

The calculated velocities behind the breakwater increase with an increasing displacement, see Table 4-8. In Table 4-8 measured and calculated velocity values are given for the normal situation and for the situation that the sheets are moved respectively 1, 2 and 3 grid points to the shore, for location 1 and 2 as shown in Figure 4-17. Appendix X shows the wave height decay and water level variation with the sheets moved one cell in onshore direction, and Appendix Y for a movement of two cell, i.e. 10 meters.
CHAPTER 5. Sensitivity analysis

<table>
<thead>
<tr>
<th>Test condition</th>
<th>H_{sig} (m)</th>
<th>Location</th>
<th>Measured velocity (m/s)</th>
<th>Calculated landward velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>1.5</td>
<td>1 (mid-depth)</td>
<td>-0.12</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 (above bottom)</td>
<td>-0.08</td>
<td>0.06</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>1 (mid-depth)</td>
<td>-0.19</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 (above bottom)</td>
<td>-0.15</td>
<td>0.04</td>
</tr>
<tr>
<td>3</td>
<td>3.5</td>
<td>1 (mid-depth)</td>
<td>-0.26</td>
<td>0.03</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 (above bottom)</td>
<td>-0.17</td>
<td>0.10</td>
</tr>
</tbody>
</table>

Table 4-8  Measured and calculated landward velocities for sheets moved respectively 0, 1, 2 and 3 grid cells onshore (n.a. = not available due to numerical instability)

One can see that the direction agrees quite well and the magnitude differs, but approaches the measurements best for movement of sheets of one or two grids cells. This movement of two and one grid cell for respectively condition 1 and conditions 2 and 3 will be used for a first movable bed calculation with the movable bed computations, see §4.4.5. There still is a difference between the locations of the highest velocities. Measurements give higher offshore-directed velocities at half depth where calculations give higher offshore-directed velocities near the bottom.

When looking at the calculated cross-sectional velocity field, see Figure 4-18, a few phenomena stand out. First, there seem to be circulation patterns in front of the breakwater.

Figure 4-18  Cross-sectional velocity field

This can be explained by the model effect the wave forcing terms only exert influence on the top layer, see Appendix H. In the breaker zone, this is correct, but outside the breaker zone the wave forces should be imposed uniform over the entire depth. Figure 4-19 shows the accordance of the signs of calculated wave forcing terms and velocities in the top layer. The wave-induced forcing terms, \( F_x \) and \( F_y \), are given by the gradients of the radiation stress tensor \( S \), see Equations (4.1) and (4.2).

\[
F_x = -\frac{\partial S_{xx}}{\partial x} - \frac{\partial S_{xy}}{\partial y} \quad \text{(4.1)}
\]

\[
F_y = -\frac{\partial S_{yx}}{\partial x} - \frac{\partial S_{yy}}{\partial y} \quad \text{(4.2)}
\]

where
- \( S_{xx} \) = radiation stress acting on a plane parallel to the coast [N/m]
- \( S_{yx} \) = radiation stress acting on a plane perpendicular to the coast [N/m]
- \( S_{xy} \) = radiation stress acting on a plane parallel to the coast [N/m]
- \( S_{yy} \) = radiation stress acting on a plane perpendicular to the coast [N/m]
In Figure 4-19 one can easily define different zones, which will be discussed from left to right. First, until \( x = 1270 \) m, the wave height decreases due to bottom friction as the only influence on the energy balance. This results in an onshore directed current in the top layer, as depicted by the blue arrows. From \( x = 1270 \) m to \( x = 1540 \) m, the wave height increases due to shoaling, resulting in an offshore directed current in the top layer. The zone from \( x = 1540 \) to \( x = 1605 \) m can be defined as a breaking zone with strong onshore currents in the top layer. From \( x = 1605 \) m to \( x = 1640 \) m the wave height increases again due to shoaling and from \( x = 1640 \) m to the shoreline can be defined as the breaker zone, with strong onshore directed currents in the top layer.

As shown by Dingemans et al. (1987), the use of gradients of radiation stresses in numerical models can result in spurious currents. In this study the sign of the driving forces switches several times, see Figure 4-19, resulting in several circulation cells, see Figure 4-20.
When looking at the calculated cross-shore velocity pattern above the breakwater crest, shown in Figure 4-21, it is according to what one can expect; onshore directed in the top layer and offshore directed at the bottom.

The depth-averaged velocities are shown in Figure 4-22. As expected for the fixed uniform bottom, no significant depth averaged velocities in both alongshore and cross-shore direction are present.
### 4.4.5 Sediment transport

The first computation with bottom updating is conducted with an onshore shift of the sheets of one cell, as discussed in §4.4.4. A morphological time scale of six is used to reduce the simulation time. After every 15 minutes, that is to say 1.5 hours in real time, SWAN computes a new wave field. Bottom updating starts after the third WAVE computation, i.e. after the warm-up time, see §4.4. For every test, the averaged transport rates are computed and averaged for each wave condition. From now on offshore losses are defined as the positive transports over the submerged breakwater. There are several ways to obtain transport values, but in this study transports calculated by the difference in depth are used, see Appendix Z. Together with the measured and computed initial and end bathymetries, they are shown in Figure 4-23.

On the left side, the measured and calculated offshore losses (m³/m/hr, positive offshore directed!) are shown for the three wave conditions and their time average. In addition, a production term is given (m³/m/hr) which will be explained later. On the right, the initial bathymetry and both the calculated and measured bathymetries at the end of the total test series, see Table 4-1, are printed in cross-shore direction, for the most interesting area between the breakwater and the shoreline.

![Figure 4-23 Measured and computed transport rates (m³/m/hr), production rate and initial and end bathymetries](image)

When looking at the transport rates and the resulting bathymetry a few phenomena stand out:

- First, the shape of erosion around the waterline agrees very well with measurements. This is owed to the possibility of erosion of dry cells;
- Although in the physical model sedimentation takes place above the waterline, this is still not possible in Delft3D. Anyhow, the large amount of sedimentation above the waterline is likely to be a model effect;
- The slope agrees well with the measurements for the upper part of the perched beach.
- Instead of the measured offshore transport, the computed transports are directed onshore. This explains the sedimentation at the toe of the breakwater with the physical tests where in the mathematical tests there is some deposition of sediment between the breakwater and the water line. An explanation of this location of deposition can be found in the same circulation cells as explained in §4.4.4. The sediment seems to deposit between two circulation cells that circulate opposite to each other, see Figure 4-32;
- Although the directions differ, the correlation between wave height and transport rate corresponds for both measurements and computations.

Concerning the transport calibration, investigation will now be divided into four main points; mass balance, erosion of the dry cells, transport direction and magnitude over the breakwater and the erosion between the breakwater and the shore.
Mass balance
The sediment mass balance is checked by comparing the product of the depth and cell area at the beginning and the end of the simulation of the total test series, see Appendix AA. After a check on the conservation of sediment mass, it appeared that a more than significant amount of sediment is added during the computations, until now for unknown reasons.

For the first computation, see Figure 4-23, the mean production was 0.16 m³/m/hr. The next example emphasizes the sensibility that comes along with this artificial production. If it would be correct to assume that this production is subsistent to the change in bottom level, it could be expected that this production of sediment takes place between the breakwater and the shore. As explained in this paragraph, the cross-shore transports are calculated by integration of bottom differences instead of the calculated transports. Keeping this in mind would imply that the cross-shore transport would change of sign when this production is subtracted from the cross-shore transport. In this case, the mean calculated onshore transport is 0.09 m³/m/hr, minus the production of 0.16 m³/m/hr would result in an offshore-directed transport of 0.07 m³/m/hr. This is more approximating the measured erosion value of 0.05 m³/m/hr, see Table 4-9. Still, this is all under the assumption that all of the production takes place between the breakwater and the shore. Anyhow, it emphasizes the sensibility of the production to the transport direction.

<table>
<thead>
<tr>
<th>calculated transport</th>
<th>-0.09</th>
<th>m³/m/hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>production</td>
<td>0.16</td>
<td>m³/m/hr</td>
</tr>
<tr>
<td>calculated transport + production</td>
<td>0.07</td>
<td>m³/m/hr</td>
</tr>
<tr>
<td>measured transport</td>
<td>0.05</td>
<td>m³/m/hr</td>
</tr>
</tbody>
</table>

Table 4-9  Calculated and measured offshore transport (positive) versus production and calculated transport and production

When observing the whole system in relation to the artificial sand mass production, one can state the next facts. In alongshore direction the system is closed, so a net transport in that direction can be excluded. Onshore, transport rates are impossible due to the dry cells. The only place where sand could enter the system naturally is at the offshore open boundary. In the 10 meter deep water, the waves have little effect on the sediment and transport through this boundary also appeared to be negligible small, see Figure 4-24. Figure 4-24 shows the mean transport rate at the boundary, the mean transport rate over the submerged breakwater and the increase of the sediment balance for the whole system, for the offshore part of the system and for the area between the breakwater and the shore.

Figure 4-24 shows some contradictory matters; concerning the actual bathymetry changes, the sediment balance shows an increase of 0.09 m³/m/hr for the area between the breakwater and the shore where as the cross-shore transport rate based on concentrations and velocities is 0.02 m³/m/hr, onshore directed. Assuming the cross-shore transport over the breakwater to be correct, the mass balance increases with 0.07 m³/m/hr between the breakwater and the shore and with 0.09 m³/m/hr at the other side of the breakwater. Reckoning with the different widths where the artificial production takes place, this would imply a relation between erosion-sedimentation rates and the sediment balance increase rate.
CHAPTER 5. Sensitivity analysis

In Appendix DD, the influence of different settings of the computational time step, grid size and morphological time step on the production and the end bathymetry is investigated. Table 4-10 shows different production rates of sediment for different numerical settings, i.e. different time steps, mean grid sizes or morphological time scales.

<table>
<thead>
<tr>
<th>run name</th>
<th>Time step (sec)</th>
<th>Mean grid size (m)</th>
<th>Morphological time step (-)</th>
<th>Net production (m³/m/hr)</th>
<th>Calculated offshore losses (m³/m/hr)</th>
<th>Measured offshore losses (m³/m/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Norm001</td>
<td>6.0</td>
<td>5.0</td>
<td>6</td>
<td>0.17</td>
<td>-0.13</td>
<td>0.05</td>
</tr>
<tr>
<td>Norm002</td>
<td>3.0</td>
<td>5.0</td>
<td>6</td>
<td>0.17</td>
<td>-0.14</td>
<td>0.05</td>
</tr>
<tr>
<td>Norm003</td>
<td>1.5</td>
<td>5.0</td>
<td>6</td>
<td>0.12</td>
<td>-0.08</td>
<td>0.05</td>
</tr>
<tr>
<td>Norm0025</td>
<td>3.0</td>
<td>5.0</td>
<td>1</td>
<td>0.07</td>
<td>-0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>Norm0026</td>
<td>1.5</td>
<td>5.0</td>
<td>1</td>
<td>-0.01</td>
<td>0.01</td>
<td>0.05</td>
</tr>
<tr>
<td>Half001</td>
<td>3.0</td>
<td>2.5</td>
<td>6</td>
<td>0.18</td>
<td>-0.07</td>
<td>0.05</td>
</tr>
<tr>
<td>Half002</td>
<td>1.5</td>
<td>2.5</td>
<td>6</td>
<td>0.15</td>
<td>-0.20</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Table 4-10 Influence of reducing the computational time step, grid size and morphological time step on the mass balance

As can be seen in Table 4-10 the net production of sediment reduces with reduction of the computational time step, grid size and morphological time step, but not below significance, compared to the calculated offshore losses.

A possible practical solution to the production could be found by comparing the erosion sedimentation patterns of two extremities, i.e. the runs with the highest and the lowest production (norm001 and norm0026). A manual correction for the artificial production could be imposed if the location of the production is known. When reduction of the artificial production stretches over the entire cross-shore profile one could impose the reduction manually after every test, to obtain conservation of mass. When the reduction of artificial production takes place due to low erosion, the correction has to be imposed there, see Figure 4-25.
Dropping these speculations, Figure 4-26 shows the cumulative erosion sedimentation for every wave condition and for both normal mass production (run norm001, plotted in blue) and the one with a reduced net production (run norm0026 plotted in green). The difference in erosion of these runs is given in red, see Figure 4-26. In this figure, the area between the breakwater and the shore is enlarged, because most of the erosion and sedimentation occurs there and therefore gives a more distinct view.
From Figure 4-26 one can see that the red difference line does not show a clear pattern in cross-shore direction, so the option of manual correction can not be used to deal with the mass balance problem. One thing that became clear when investigating the increase of the sand balance in time, was that generally over half of the production takes place during wave condition 3, see Appendix BB. With wave condition 3 having the largest transport rates, it can be stated that there is a relation between the transport and the artificial production of sand.

Therefore, the influence of several numerical, empirical and transport parameters is investigated for wave condition 3, but none of them seemed to be fully responsible for the production of sand, see Appendix CC. Considering these investigations, there is still no explanation of the production of sediment at this stage.

**Transport direction**

Besides the mass balance problem, the direction of the total transport over the submerged breakwater differs between the measurements and computation. In this section, the sediment transport rates that will be discussed are the sediment transport rates from concentrations and velocities, since without mass balance problems this would equal the sediment transport calculated by bottom changes, as done before.

Initially, the advection transport solely seemed to be offshore directed, see Figure 4-27, but the total transport, i.e. advection and diffusion transport, was onshore directed.

![Velocity, concentration and advection transport distribution at the breakwater crest](image)

*Figure 4-27 Velocity, concentration and advection transport distribution at the breakwater crest*

Typical calibration parameters for sediment transport are the viscosity and diffusivity parameters, see §5.3.1. The horizontal eddy diffusivity term seemed to be the key to offshore-directed transport. In a way, one could expect this since turbulence behind the breakwater is not simulated good enough yet. Increasing the horizontal eddy diffusivity term for wave condition 3 from 10 m²/s to 30 m²/s switched the sign of the transports positive, i.e. offshore directed as measured, see Figure 4-28. This figure shows the suspended (ssuu) and bed load (sбуu) transport rates over the submerged breakwater for wave condition 3, being the dominant wave condition concerning transport rates.

![Suspended transport rates and bed load transport rates for different horizontal eddy diffusivity parameters](image)

*Figure 4-28 Suspended transport rates and bed load transport rates for different horizontal eddy diffusivity parameters*
Unfortunately, increasing the horizontal eddy diffusivity term only shows offshore-directed transport initially. Appendix DD shows the calculated transport rates per wave condition for several different settings, but increasing the horizontal eddy diffusivity term did not result in mean offshore transport for any wave condition. Again, the production of sediment was the dominant factor.

For numerical simulation reasons, the bottom roughness at the breakwater could be of influence on the transport direction, see §5.3.2 and Appendix EE. In this case, the onshore transport rates reduced, but did not switch of direction by changes of the bottom roughness at the submerged breakwater.

**Erosion of dry cells onshore**

Although sedimentation above the waterline can not be simulated yet, erosion of dry cells above the waterline can, using a factor THETSD. This factor, see Appendix P, determines the fraction of erosion of the last wet computational cell that is assigned to the adjacent dry cell. This factor is user defined and lies between 0.0 and 1.0.

![Figure 4-29 Erosion of dry cells for different THETSD values](image)

Figure 4-29 shows a blow-up of the area around the waterline. Besides the mean water level and the measured initial and end bathymetry, the calculated end bathymetries are shown for different THETSD values. A remarkable phenomenon is that the erosion of dry cells is more or less equal for THETSD values of 0.4 and 1.0 where as 0.8 shows the most erosion. This cannot be explained on the basis of the THETSD value. Probably there is another differing parameter. Besides the general trend of increasing erosion of dry cells with increasing THETSD values, the measured erosion around the shoreline is still very superficial. Not only the THETSD-value determines the erosion around the shoreline, but also diffusivity and viscosity terms, see §5.3.1. Appendix DD shows the end bathymetries for several different settings of THETSD- and diffusivity values. Following these graphs, a combination of a THETSD value of 1.0 and a horizontal diffusivity value of 60 $m^2/s$ gives the best results, see Figure 4-30 and Figure A 111.

![Figure 4-30 Measured and calculated end bathymetry with THETSD=1.0 and a horizontal eddy diffusivity of 60 $m^2/s$.](image)
**Sedimentation and erosion between the breakwater and the shore**
The sedimentation and erosion between the breakwater and the shore is mainly determined by the supply of sediment by erosion of the dry cell at one side, by the offshore transport over the breakwater from the other side and by the velocity patterns between the breakwater and the shore, see Figure 4-31.

![Determining factors for sedimentation and erosion between breakwater and shore](image)

**Figure 4-31**  
Determining factors for sedimentation and erosion between breakwater and shore

The supply of sediment by the erosion around the waterline can agree with the measurements as discussed above. Therefore, it is assumed that this is not the limiting factor when adjusting the sedimentation-erosion pattern between the breakwater and the shore. The cross-shore transport rates are still hard to calibrate. The actual transport rates between the breakwater and the shore are particularly determined by the velocity patterns.

![Initial and end bathymetries, velocity field and two opposite circulation cells](image)

**Figure 4-32**  
Initial and end bathymetries, velocity field and two opposite circulation cells

As discussed in §4.4.4, spurious circulation cells develop, which lead to sedimentation between the two counter-circulating cells, see Figure 4-32. Without another way of simulating wave driven currents the sedimentation between the breakwater and the shore does not seem to be adjustable. A dependence of a wave height to depth ratio to the kind of force distribution could help solve this problem; in the breaker zone the forces exert their influence in the top layer and outside the breaker zone the forces are vertically distributed over the entire depth.
4.5 Model validity checks

To be sure if the model is not conflicting with basic physical laws and unit demands, some checks are made. In the next paragraphs, the output is checked for negative mass values, conservation of mass, check on the exhaustion of available sediment and a check for alongshore uniformity.

4.5.1 Check on negative mass values

Two different mass values are distinguished; available sediment at the bottom and concentration values of suspended sediment. During this study, soon the available mass values at the bottom became negative. A succeeding executable version was made to check on these negative values. This adaptation was effective immediately, but later on, still some negative values appeared to occur, although smaller, negligible values.

For the latest simulations, the number of occurrences of negative available sediment values is counted for the sum of all grid points and time steps, see Appendix DD.

For every simulation, a check is made for negative concentration values and negative available sediment values, for every output time step and every grid cell. In Appendix DD one can see the results of these counts. First, the solution for the negative concentration was searched in the Forester Filter as described in the FLOW user manual, (WL | Delft Hydraulics, 2000-f). This Forester Filter adds numerical diffusion where the sediment concentrations are negative. Albeit an activated Forester Filter, still negative concentrations were present.

So far, for both the occurring negative concentrations and negative available sediment values, there is no explanation.

4.5.2 Check on the conservation of mass

Already in §4.4.5 the lack of conservation of mass was detected. Additionally, in Appendix BB the total mass is plotted in time. Here it appeared that the production of mass takes place linearly in time during the simulation and faster for the heaviest wave conditions. Also the influence of several numerical, empirical and transport parameters on sand production is investigated, but none of them seemed fully responsible for the production of sand, see Appendix CC. In addition, no differences in sediment take place between two wave conditions, as it was with the former software versions, see Appendix FF.

4.5.3 Check on the exhaustion of available sediment

Besides the non-erodible area of the breakwater, the available sediment may not be exhausted during the simulation. Especially around the water line most of the erosion occurs. A first sediment thickness of 4000 kg/m² (i.e. 2.50 meters) exhausted with some simulations and was therefore increased to 10,000 kg/m² (i.e. 6.25 meters). This solved the problem.

4.5.4 Check on uniformity in alongshore direction

Until now, uniformity is assumed in alongshore direction. Therefore, all values were averaged in alongshore direction for all six grid cells. In Figure 4-33 the velocity field and the updated bathymetry are shown in a view from above.
As one can see from Figure 4-33, two circulation cells are formed. This looks like an intensifying situation; the difference in set-up due to the alongshore depth variation induces alongshore currents and these currents will intensify the alongshore variation. However, this vicious circle has to start with an initial disturbance. This initial disturbance will be investigated by the situation in alongshore direction, somewhere in the breaker zone, see Figure 4-34. Figure 4-34 shows the significant wave height and water depth from the wave field computation (upper two plots) and the water level and bottom depth distribution in alongshore direction from the FLOW computation. Every graph contains two lines; the first one is numbered with a one and depicts the situation just before bottom updating and the second one depicts the situation with just ten minutes of bottom updating.

Initially, before bottom updating, the SWAN wave height and water depth are distributed uniform in alongshore direction and the FLOW bottom is obviously also uniform, see the lines numbered with an one. The water level, however, showed little fluctuations in alongshore direction, caused by boundary adjustments. It is probably this disturbance that is the origin of greater disturbances. After ten minutes of bottom updating, the FLOW bottom is changed. This changes the water depth for SWAN and this has an influence on the wave height distribution, see lines numbered with a two.

Something else that can be seen from Figure 4-34 is the uniform water depth for SWAN outside the bottom grid. As said in §3.6.2, the water depth is copied from the nearest known value, resulting in alongshore uniform bathymetry, resulting in its turn for uniform wave conditions between the FLOW grid and the SWAN boundary disturbances.
Eventually, the wave field in top view perspective looks as in Figure 4-35; disturbances within the FLOW grid by the altered bottom and at the boundaries by boundary disturbances. The boundary disturbance area differs from Figure 3-7 in a sense that it is smaller. That is due to the small directional spreading of one degree for this simulation.
4.5.5 Deviation between depths and wave heights

As discussed in §4.4, the initial bottom is used for the wave height calibration of the wave conditions. This is done for sake of research time reduction. Nevertheless, it would be better to use the mean measured bathymetry of every wave condition or even the mean measured bathymetry for every one of the 14 tests. In this paragraph the deviation in bathymetry and wave height is investigated. First, Figure 4-36 shows the development in time of the depths at the four locations. Between arrows, the wave condition is given at the right point of time.

![Figure 4-36](image)

Concerning these depth, the depth at location two changes significantly during the second test and the depth at location four changes significantly during the first half of tests. For wave condition one this means that the depth changes more and more for the last test. For wave condition two and three, occurring later in the test series, the depth changes less in time, but the depth changes with the initial depth that is used during the calibration.

What the effect is of this deviating depths on the wave heights can be seen in Figure 4-37. This figure depicts the wave height development in time for the three wave conditions and at each of the four measurement locations. The calibrated wave height using the initial depth is given, which is the same as the mean measured wave height with changing depth. Subsequently, the mean calculated wave height is given. The difference between these wave heights is a measure for the fault of the assumption to use the initial depth for the calibration.

![Figure 4-37](image)

All in all, the differences are small, i.e. less than 5% deviation of the calculated mean wave heights to the real measured and calibrated wave heights. When comparing this deviation with the differences in velocities between measurements and calculations, the assumption is acceptable to use a single initial bathymetry for the calibration,
Figure 4-37  Wave height development in time for the three wave conditions
4.6 Differences between the mathematical and physical model

One can point out simplifications and model effects of both the physical model and the mathematical model that have to be realised when comparing the results.

With regard to the physical model, the following points stand out:
- Inaccuracies in measurements; when scaling the model values to prototype values a small measurement fault becomes larger in absolute sense. For the physical model, three cross-section bathymetries are averaged to compare with the mean calculated cross-section, using every cell;
- The breakwater is not for 100% impermeable, as it is simulated in Delft3D; The amount of transport due to this permeability is still hard to predict;
- In reality a process of sediment sorting can occur; Due to a spreading in the sediment diameter, different grains react different on some processes, resulting in a different transport;
- The accretion above the waterline is overestimated in the physical model;
- In the physical model, waves are driven by the paddle and subsequently only bottom friction and wave breaking urge on the energy balance. In reality these wind waves and their energy balance are affected by wind growth, whitecapping and quadruplets interaction. Though deactivated in the mathematical model as well, this does not represent the reality;
- As in the WL | Delft Hydraulics report (2000-e), the physical test is scaled to prototype circumstances for the calibration of the mathematical model. During this scaling of scale model values to prototype values, scaling rules are used that are probably not perfect, especially with respect to sand size.

With regard to the mathematical model, the following points stand out:
- Sedimentation above the water line is yet not possible with Delft3D.
- The distribution of wave forces should be distributed over the vertical in the shoaling zone, but they are imposed in the top layer.
- Wave breaking will start on the breakwater, but part of the turbulent water movement of the breaking will take place just landward of the breakwater. This increased turbulence is not included in the model. As a result, just landward of the structure sediment concentrations are likely to be somewhat underestimated and sediment deposition just behind the breakwater may be over-estimated.
- Reflection is not simulated in SWAN.
5 Sensitivity analysis

5.1 Introduction
In this chapter, the sensitivity is examined of several numerical and empirical input parameters on the end bathymetry. As numerical parameters, time step, grid size and wave-field updating interval are distinguished in §5.2. The empirical parameters that are examined in §5.3 are the viscosity and diffusivity parameters, the bottom roughness and transport parameters. In §5.4 the sensitivity of hydraulic parameters is investigated. A critical comment that one has to face is the fact that the sensitivity on the end bathymetry is analysed, whilst the calculation of the end bathymetry is not satisfactory due to the artificial production. Therefore, it might be that the investigated parameter has more influence on the production of sand than it would have on the transports with a model without a lack of mass conservation.

5.2 Numerical aspects

5.2.1 Time step
Halving the numerical time step should not have to influence the results. When doing so, comparing two identical simulations with different time steps, the only aspect that stands out concerning the end bathymetry, see Figure 5-1, is some extra erosion around the water line. Yet no explanation can be attributed to this phenomenon, but the effect is nevertheless negligible.

![Influence of halving the time step on end bathymetry](image)

*Figure 5-1 Infl unce of halving the time step on end bathymetry*
5.2.2 Grid size
When halving the FLOW computational grid size, the effect on the end bathymetry is negligible as can be seen in Figure 5-2. The main difference can be seen around the shore line, but that is due to a different MORFAC factor, see §4.4.5.

![Figure 5-2 Effect of grid size halving on end bathymetries](image)

5.2.3 Number of iterations
As said in paragraph 4.3 the WAVE and FLOW modules use each other’s output. SWAN uses the bathymetry and water level calculated in FLOW and FLOW uses the SWAN output. With a fixed bed, equilibrium will be established after a few WAVE-FLOW successions, as no circumstance changes. When the bathymetry changes, this will have an effect on the wave conditions. One can choose the interval time of a new wave field calculation.

In the next example, first an interval time of 15 minutes is used and then an interval of 7.5 minutes and 30 minutes with the same morphological time factor six. Figure 5-3 shows the initial bathymetry and end bathymetries for the different intervals of new wave field calculations. Unfortunately, the two intervals of 7.5 and 30 minutes became unstable during the computation and therefore the influence of the number of iterations is hard to determine.

![Figure 5-3 Effect of wave field updating interval on end bathymetries](image)
5.2.4 Influence of the morphological time factor

For reduction of the calculation time one can determine a morphological time factor, ‘morfac’ in the FLOW input file ‘morph.inp’, see Appendix CC. All the bathymetry changes are multiplied by this factor.

Reducing this factor and increasing the simulation time by the same proportion should result in equal end bathymetries. Figure 5-4 shows the initial and end bathymetries for a ‘morfac’ factor 1 and 6. Again, the greatest differences in the end bathymetry can be seen around the water line.

![Effect of morphological time factor on the end bathymetries](image)

Figure 5-4 Effect of morphological time factor on the end bathymetries

5.2.5 Transition between two wave conditions

For all the executed runs, the transition between two wave conditions is a bit different from in the physical tests. For the physical tests, it takes some time from the wave paddle start for the sediment to reach a certain equilibrium concentration. When the wave paddle stops the suspended sediment settles and the new bathymetry is measured, see Figure 5-5. Figure 5-5 shows two plots; the upper one depicts the volume of suspended sediment in time that goes to equilibrium after some time and goes back to zero between two wave conditions. The lower one depicts the total volume of sand at the bottom in time. Conservation of mass tells that the sum of these two values is constant in time.
For the mathematical model, a warm-up time is needed for the velocities and wave conditions to play in, see §4.4. Therefore, the bottom updating is activated after the third wave field calculation. For sake of simplicity, it is assumed that the volume of suspended sediment does not change significantly during the bottom updating. This assumption simply enables to extract the updated bathymetry at the end of the simulation while there is still sediment in suspension. The next wave condition starts with clear water, i.e. no sediment is suspension at the first time step, see the blue line in Figure 5-6.
As long as the volume of sediment in suspension does not change during the bottom updating one can expect the next run to start with an equal amount of sand, blue line in Figure 5-6. If this assumption is not correct and the amount of sediment in suspension changes in time, sediment is either created or lost. If the volume of sediment in suspension decreases, this sediment settles at the bottom. The next run uses this higher amount of available sediment and sand is created, see the yellow line in Figure 5-6. If the sediment in suspension increases, the opposite occurs, purple line in Figure 5-6.

Leaving this theory behind, Figure 5-7 shows the actual calculated suspended sediment and available sediment at the bottom, for four concessive wave conditions. For every wave condition, first there is a warm-up time, after which the bathymetry is up-dated. To start with the suspended sediment of the first wave condition, the volume of suspended sediment starts with zero and reaches equilibrium very soon. The next condition starts with ‘clean water’ again, i.e. an initial condition of zero sediment in suspension. For all the wave conditions, it can be said that the volume of suspended sediment does not change during the period of bottom updating. Concerning the sediment at the bottom, two values are traced out. First the volume of available sediment and second the volume of sand above m.s.l. -12.5 m, see Appendix CC. Both volumes do not change during the warm-up time, but do increase during the bottom updating. This is due to the numerical production of sediment, see §4.4.5.
To investigate the influence of the type of transition between two wave conditions, another run is conducted where each wave condition is followed by a computation without wave activity. The suspended sediment is extracted from the previous run and used in an initial condition file. A harmful necessity is that the bottom updating has to start from the first time step on. Now the theoretical development of the volumes of sand in suspension and at the bottom in time should look like the real physics, see Figure 5-5.

Figure 5-8 shows actual calculated values. Again the volume of sediment in suspension and the volumes of the sediment at the bottom. Concerning the volume of sediment in suspension, again a quick establishment of equilibrium can be seen, followed by a decrease to zero during the zero wave condition. The volumes of sediment at the bottom seem to decrease during the pick-up of sediment and increase during the settlement of sediment. Still, there is a net increase of sediment during the simulation.
Due to the production of sediment during the simulation, the influence of the transition between two wave conditions is hard to explain, but it can be said that during the period of bottom updating, the volume of suspended sediment hardly changes, see Figure 5-7. Therefore, it is accepted to extract the bathymetry at the end of a wave condition, without the use of an extra simulation period without wave activity.
5.3 Empirical parameters

5.3.1 Viscosity and diffusivity parameters

In case of the used k-ε turbulence model, the uniform values for horizontal and vertical diffusivity and viscosity as variable parameters in the FLOW input-file are used as a background minimum value. In this paragraph, the influence of these parameters is checked up on. Their influence on velocities, concentrations and transports is checked for both lower and higher values than the default values. The default values are shown in Table 5-1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal eddy viscosity</td>
<td>m²/s</td>
<td>10</td>
</tr>
<tr>
<td>Horizontal eddy diffusivity</td>
<td>m²/s</td>
<td>10</td>
</tr>
<tr>
<td>Vertical eddy viscosity</td>
<td>m²/s</td>
<td>1.0E-6</td>
</tr>
<tr>
<td>Vertical eddy diffusivity</td>
<td>m²/s</td>
<td>1.0E-6</td>
</tr>
</tbody>
</table>

Table 5-1 Default viscosity and diffusivity values

Vertical eddy viscosity
The vertical eddy viscosity is determined by k and ε. The uniform value is used as a background minimum value, to damp out short oscillations.

Vertical eddy diffusivity
The vertical eddy diffusivity is derived from the eddy viscosity, according to Equation (5.1).

\[ D_v = \frac{\nu_v}{\sigma_c} \]  

where

- \( D_v \) = vertical eddy diffusivity [m²/s]
- \( \nu_v \) = vertical eddy viscosity [m²/s]
- \( \sigma_c \) = Prandtl-Schmidt number [-]

The uniform value is used as a background minimum value.

Horizontal eddy diffusivity
The horizontal eddy diffusivity parameter is a typical calibration parameter. The horizontal eddy diffusivity parameter mainly exerts influence on the concentration distribution, by means of the advection-diffusion equation, see Appendix I. Physically, the horizontal eddy diffusivity distributes the sediment horizontally. Figure 5-9 shows different concentrations for different horizontal eddy diffusivity values. Especially for higher values, there is an increase in concentration values.

The total influence of the horizontal eddy diffusivity on velocities, concentrations and advection transport above the breakwater can be seen in Figure 5-10. Besides the velocities, concentrations and advection transport as a function of the depth above the breakwater crest, values of suspended (ssuu) and bed load (sbuu) transport are depicted in every upper left corner.
CHAPTER 5. Sensitivity analysis

Figure 5-9  Influence of horizontal eddy diffusivity parameter on the concentration distribution
As can be seen from Figure 5-10, there is a correlation between the horizontal eddy diffusivity and the concentration. With the velocities not influenced by the horizontal eddy diffusivity, this correlation also counts for the transports. For higher values of the horizontal eddy diffusivity, the total cross-shore transport switched direction to offshore (positive value). The higher eddy diffusivity values seem to model turbulence behind the breakwater quite well.
**Horizontal eddy viscosity**
The horizontal eddy viscosity parameter also is a typical calibration parameter. The horizontal eddy viscosity directly influences the momentum equation, see Appendix H. The influence on the velocity distribution becomes negligible for smaller horizontal eddy viscosity values, as can been seen in Figure 5-12. The influence of the horizontal eddy viscosity on the concentration distribution is also very little, see Figure 5-11, and therefore the influence on transport above the breakwater is small for horizontal eddy viscosity values, see Figure 5-12.

![Figure 5-11 Influence of horizontal eddy viscosity parameter on the concentration distribution](image)
CHAPTER 5. Sensitivity analysis

Figure 5-12  Velocity, concentration and advection transport distribution above the breakwater for different horizontal eddy viscosity values

All in all both the sensitivity of changes of the horizontal eddy viscosity values on advection transport and suspended transport is negligible small.
5.3.2 Influence of bottom roughness on sediment transport

As explained in Appendix EE, the reference concentration height is determined by the roughness height. Below this height, the concentration is assumed equal to the reference concentration. In combination with a fine \( \sigma \)-grid, i.e. many layers in vertical direction, and a small water depth, the reference height can cover several layers.

![Figure 5-13 Influence of Manning roughness coefficient on transport](image)

In this case, a decrease of the Manning roughness coefficient causes little reduction of the onshore transport and does not change the transport direction, see Appendix EE.

5.3.3 Sediment transport parameters

The influence of all the transport parameters is investigated on the sand production and can be seen in Appendix CC. The influence on the sediment transport is hard to investigate, since the production term is dominant.
CHAPTER 5. Sensitivity analysis

5.4 Hydraulic parameters

5.4.1 Spectral resolution

For the initial WAVE input, a sector of 255 to 285 degrees is used as directional space to reduce computational time. This should not give any differences compared to a 360 degree circle as directional space with the use of a wave direction of 270 degree and a directional spreading of 1 degree. This assumption is investigated by an additional simulation with a circle as directional space. The influence on the wave height decay and the wave transmission can be seen in Figure 5-14. The wave height decay for both directional space settings is exactly equal, as one could expect, but therefore hard to distinguish in the graphs of Figure 5-14.

![Wave height decay with 255-285° sector](image)

Figure 5-14 Wave height decay with a circular directional space

5.4.2 Directional spreading

As with the physical model, all the computational simulations should be conducted with long crested waves, i.e. without directional spreading. Soon it appeared that a directional spreading of 1 degree was a minimum value. As discussed in §4.4, perfect results could be obtained with this setting. However, in reality, the wave energy mostly has a directional spreading of 30°. Figure 5-15 shows the wave height decay with a directional spreading of 30 degrees. It has to be said that for this computation a circular directional space is used, because the sector between 255 and 285 degree would contain only a part of the energy. Again, no difference in wave height decay can be found, as can be seen by the equal situation of the graphs of Figure 5-15.
As expected, the boundary disturbances are greater for this wider directional spreading, see Figure 5-16, compared to Figure 4-35. For this investigation on the sensitivity of the directional spreading, a fixed bottom is used. This explains why no disturbances can be found within the FLOW grid, where as in Figure 4-35 can.

Figure 5-15  Wave height decay at cross-section y=1500, with a directional spreading of $30^\circ$

Figure 5-16  Wave field with a directional spreading of $30^\circ$
6 Conclusions and recommendations

6.1 Conclusions
The conclusions are separated in conclusions drawn from the literature study, the calibration process and the sensitivity analysis on non-uniformities.

6.1.1 Conclusions on the literature study
Concerning the location and the shape of the submerged structure, the opinions differ as the conditions differ for the different studies. Concerning the perched beach profile, a Dean’s equilibrium beach profile can be used, determined by the entering wave energy flux. However, the beach parameter is smaller for reduced wave energy resulting in a steeper profile. For the lion’s share of the physical studies, winter conditions determined the erosion. The most effective measures are to adapt the initial profile to the winter profile, to use adaptive beach fill material and to reduce the wave attack. Still mostly the perched beach physical test results are not generally applicable, but the studies on the performances of perched beaches can help with a preliminary design. For the final design, dedicated physical and mathematical tests are recommended. The mathematical studies generally describe a steeper slope behind the breakwater than the beach profile without a submerged breakwater. Since the 2DV tests did not deal with alongshore currents, the performance of actual built perched beaches was often different than expected. The importance of groins is recognised, because in many cases the alongshore transport appeared to be dominant for the erosion.

6.1.2 Conclusions on the calibration process and Delft3D
The used Delft3D modules are state of the art simulation tools for morphological simulation. Most of the hydraulic features can be simulated well, but others need to be reviewed and adjusted for future versions. The relevant phenomena will be discussed separately.

Wave height reduction
The wave height reduction over the breakwater can be simulated and adapted very accurate, equal to the measurements. Using a series of obstacles makes it possible to simulate the crest width for the wave computation. This is, of course only possible for relatively small grid size, as used in this study. In case of a computational simulation of a submerged breakwater without physical measurements of the wave transmission, the transmission can be obtained with the use of Seabrook and Hall’s formulation.

Wave set-up
The wave set-up calibration can be controlled by spreading apart or placing together the sheets and their imposing transmission values, see §4.4.2. Without measurements one can use an analytical computation to calculate the set-up.

Transition zone and velocities
Shifting the obstacles in onshore direction can delay the wave forces that enforce the currents in the top layer. The forcing of currents by radiation stress gradients causes a few spurious circulation cells, but at the location of the velocity gauges both direction and order of magnitude of the flow agree quite well, see §4.4.4.
CHAPTER 6. Conclusions and recommendations

**Sediment transport**
The cross-shore transport over the submerged structure can be obtained from the calculated output in several ways, see Appendix Z. The cross-shore transport over the submerged structure, directly from the output files, is mainly onshore directed, but adjustment of the horizontal eddy diffusivity can change direction as it represents the turbulence near the breakwater. Unfortunately, the direction of a whole simulation series is still onshore directed. Even with some offshore transport during some individual tests, the bed level rises between the breakwater and the shore, due to a numerical production of sediment. This was partly due to the fact that a version of Sediment Online is used in which cut-off values for small bottom changes caused sand production and partly due to working with a morphological factor. It could be possible to look at the cross-shore transports from the trim file only, see Appendix DD, and refrain from the preservation of mass laws, but it would be contradictory with any fair calibration.

The erosion around the shoreline and the erosion of dry cells can be simulated fairly well thanks to the THETSD value, see §4.4.5.

The erosion and sedimentation between structure and the shoreline is different than the measurements due to the spurious circulation cells, see §4.4.5. The eroded sediment from the water line area seemed to deposit between the circulation cells, preventing it from reaching the breakwater where it possibly could result in offshore transport.

**Mass balance problem**
Soon it appeared that the volume of sediment increased during the simulations. This increase is linear in time and correlates with the wave height. The influence of every parameter on this production is investigated. Although differences in contribution of the production exist between different parameters and the time step and morphological factor reduced the production the most, the production term did not reduce below significance. Later on, negative concentration values and available sediment values are found, but this would only reduce the mass balance. With only positive mass values, the mass balance would increase even more, though little compared to the already occurring increase.

During the completion of the final version of this report, the source of the mass production became known. The cause is found in cut-off values for small bottom changes and by the fact that when using the morphological factor, this cutting off occurs more often. The first cause is recently put right in a new version of Sediment Online and the second cause can be avoided when using a morphological factor of one.

**6.1.3 Conclusions on the investigation of influences of non-uniformities**
At the end, the calibration of the model took up more time than planned. Therefore, it seemed inevitable not to start the investigation of the influences of non-uniformities during the calibration process. Continuation of the thesis investigation is still very interesting. Even if the calibration in cross-shore direction can be hard, when involving alongshore currents, other effects can become dominant and pleasant results can occur.
6.2 Recommendations

Although some essential problems have been encountered, see Appendix FF, there are still other challenges to encounter. There is a need for further investigation of the mass balance problem in Delft3D and the problems of negative concentrations and negative masses have to be unravelled. As said in §6.1.2, WL Delft Hydraulics is working on it. The causes are known and new versions are developed.

Spreading of the current driving forces in Delft3D or relating the force to a wave height to depth ratio is advised to prevent artificial circulation cells. Future developments of Delft3D will couple the driving force to a wave-to-depth ratio. Because of this, currents will only be forced in the breaker zone and not in the shoaling zone. This would prevent the spurious circulation cells. (Roelvink, personal communications).

When the improvements are implemented in the Delft3D software package, it is interesting to investigate the model on other physical perched beaches studies, as described in Appendix D. The Lido di Ostia physical tests are interesting in a way to see if the present-day software with three-dimensional computations would have resulted in another design. A second interesting case to investigate is the Flemish coast project, because here the tide also plays a part in the beach development, where as most studies do not.

It is recommended to investigate on the perched beach concept further on. The perched beach in IJmuiden, see Appendix A, lends itself perfectly for studies on this subject because of the relative simple conditions. The waves approach the beach from the same direction all the time. Ever since the construction, the beach profile is measured periodically. In addition, when using prototype measurements, one can avoid the scaling errors.
References

d’Agremond, K., Van der Meer, J.W., De Jong, R.J., 1996
Wave transmission at low crested structures
Proc. of the 25th Int. Conf. on Coastal Eng. ASCE vol. 2, pp. 2418-2427

Ahrens, J.P., 1987
Characteristics of reef breakwaters

Andrews, D.G., McIntyre, M.E., 1978
An exact theory of non-linear waves on a Langrangian-mean flow

Arai, Y., Tamura, M., 1987
Erosion control measures and beach restoration plan at the Niigata west coast
ASCE Coastal Zone 1987, Conference Proceedings, pp. 4545-4555

Basco, D.R., 1985
A Qualitative Description of Wave Breaking.
Journal of Waterways, Port, Coastal and Ocean Engineering, ASCE, vol 111 (2), pp. 171-188

Basco, D.R., Yamashita., T., 1986
Toward a simple model of the wave breaking transition region in surfzones
Proc. of the 20th Int. Conf. on Coastal Engineering, ASCE, pp. 955-970

Bascom, W., 1959
The relationship between sand size and beach-face slope

Energy loss and set-up due to wave breaking of random waves

Berenguer, J.M., Enriquez, J., 1988
Design of pocket beaches: the Spanish case
Proc. of the 21st Int. Conf. on Coastal Eng., ASCE, pp. 1411-1425

Birkemeier, W.A., 1985
Field data on seaward limit of profile change
Journal of Waterways, Port, Coastal and Ocean Engineering, vol 111 (3), pp 598-602, ASCE

Browder, A.E, Dean, R.G., Chen, R., 1996
Performance of a submerged breakwater for shore protection
Proc. of the 25th Int. Conf. on Coastal Eng., ASCE, vol 2, pp. 2312-2324-

Bruun, P., 1954
Coast erosion and the development of beach profiles
Beach Erosion Board, Technical memorandum, No. 44

Chatham, C.E., 1972
Movable-bed model studies of perched beach concept
Proc. of the 13th Int. Conf. on Coastal Eng., ASCE, pp. 1197-1216
Chatham, C.E., Davidson, D.D., Whalin, R.W., 1973
Study of beach widening by the perched beach concept, Santa Monica Bay, California
U.S. Army Engineer Waterways Experimental Station, Technical report H-73-8

Chia, W.S., 2001
Conceptual design of a pocket beach in deep water at Marina East, Singapore
MSc thesis, IHE Delft

Dally, W., Dean, R.G. Dalrymple, R., 1985
Wave height variation across beaches of arbitrary profile
J. Geophys. Research, vol. 90 No. C6, pp. 11,917-11,927

Daemrich, K.F., Kahle, W., Partenschky, H.W., 1985
Schutzwirkung von Unterwasserwellenbreckern unter dem Einfluss unregelmassiger Seegangswellen
Hannover

Dean, R.G., 1977
Equilibrium beach profiles: US Atlantic and Gulf Coast
Dep. Of Civ. Eg. , Ocean Eng. Report no 12, University of Delaware, Newark, DE

Dean, R.G., 1986
Coastal armouring, effects, principles and mitigation
Proc. of the 20th Int. Conf. on Coastal Eng., ASCE, pp. 1843-1857

Dean, R.G., 1987
Coastal sediment processes: toward engineering solutions
Coastal Sediments, 1987, pp. 1-24

Dean, R.G., 1988
Evaluation of shore protection structures including beach nourishment
In: Short course on planning and designing maritime structures, Malaga, Spain, June 1988

Dean, R.G., Chen, R., Browder, A.E., 1997
Full scale monitoring study of a submerged breakwater, Palm beach Florida, USA
Coastal Engineering vol. 29 Elsevier, pp. 291-315

Deguchi, I, Sawaragi, T., 1986
Beach fills at two coasts of different configuration
Proc. of the 20th Int. Conf. on Coastal. Eng. ASCE vol. 2, pp. 1032-1046

Computation of the driving forces of wave induced currents
Coastal Engineering vol. 11, 1987, pp. 539-563

Performance of a perched beach – Slaughter beach, Delaware
Coastal Sediments, 1987, pp. 1385-1398

Dunham, J.W., 1968
Proposed Santa Monica Causeway project
Proceedings ASCE, Journal of the Waterways and Harbors Division, pp.425-436

Eldeberky, Y., Battjes, J.A., 1996
Spectral modelling of wave breaking: Application to Boussinesq Equations
Ferrante, A., Franco, M, Boer, S., 1992
Modelling and monitoring of a perched beach at Lido di Ostia (Rome)

Fredsøe, J., Deigaard, R., 1992
Mechanics of coastal sediment transport
Advanced series on ocean engineering – vol. 3, World Scientific Publishing

Goda, Y., Takeda, H., Moriya, Y., 1967
Laboratory investigation of wave transmission over breakwaters

González, M., Medina, R., Losada, M.S., 1999
Equilibrium beach profile model for perched beaches
Coastal Engineering vol. 36 (4) 1999, pp. 343-357

Effect of submerged breakwaters on profile development
Proc. of the 25th Int. Conf. on Coastal Eng., ASCE, vol. 2, pp. 2428-2441

Gunyakti, A., 1987
Beach preservation by means of offshore submerged mound of dredged materials
Coastal Zone 1987, pp. 2461-2471

Gutman, A.L., 1979
Low-cost shoreline protection in Massachusetts
Coastal Structures 1971, pp. 373-387

Gienapp, H., Hasselmann, D.E., Kruseman, P., Meerburg, A., Müller, P., Olbers, D.J., Richter, K.,
Sell, W., Walden, H., 1973
Measurements of wind-wave growth and swell decay during the Joint North Sea Wave Project
Dtsch. Hydrogr. Z. Suppl., 12, A8

Hearn, J.K., 1987
An analysis of stability of and wave modification due to low crested, sacrificial breakwaters
MSc thesis, University of Florida
ULF/COEL/MP-87/1

Hsu, J.R.C., Evans, C., 1989
Parabolic bay shapes and applications

Hsu, J.R.C., Silvester, R., Xia, Y.M. 1989
Static equilibrium bays: new relationships
Journal of Waterways, Port, Coastal and Ocean Engineering, ASCE, 155 (3), pp. 285-298

Hughes, S.A., Chiu, T.J., 1978
The variation of beach profiles when approximated by a theoretical curve
MSc Thesis, University of Florida

Engineering methods for predicting beach profile slopes
REFERENCES

Lamberti, A., Mancinelli, A., 1996
Italian experience on submerged barriers as beach defence structures
Proc. of the 25th Int. Conf. on Coastal Eng., ASCE, vol. 2, pp. 2352-2365

Larson, M., Kraus, N.C., 1989
Numerical model to simulate storm-induced beach change
Army Corps of Eng., Waterway Experiment Station, Technical report CERC-89-9

Liberatore, G., 1992
Design and reliability of coastal structures
Proc. of the short course on the design and reliability of coastal structures, Venice, 1992

Moore, B., 1982
Beach profile evolution in response to changes in water level and wave heights
MSc Thesis, University of Delaware

Moreno, L.J., Kraus, N.C., 1999
Equilibrium shape of headland-bay beaches for engineering design
Coastal Sediments 1999, ASCE, vol. 1 pp. 860-875

Muñóez-Pérez, J.J., Tejodor, L., Medina, R., 1999
Equilibrium beach profile model for reef-protected beaches
Journal of Coastal Research vol. 15 (4), pp. 950-957

Transition zone width and implications for modelling hydrodynamics
Proc. 22nd Int. Conf. on Coastal Eng., Delft 1990, ASCE, pp.68-82

Nelson, R.C., 1994
Depth limited design wave heights in very flat regions
Coastal Engineering, vol. 23, pp. 43-59

Onderwater, M.C., 2002-a
Stabiliteit hangend strand IJmuiden, concept rapport
Alkyon Hydraulic Consultancy and Research, March 2002

Onderwater, M.C., 2002-b
Stabiliteit hangend strand IJmuiden, definitief concept rapport
Alkyon Hydraulic Consultancy and Research, May 2002

Pilarczyk, K.W., Zeidler, R.B., 1996
Offshore breakwaters and shore evolution control
A.A. Balkema, Rotterdam, the Netherlands

Powell, K.A., Allsop, N.W.H., 1985
Low-crest breakwaters, hydraulic behaviour and stability
Hydraulics Research report No. SR 57, Wallingford

Ruig, J.H.M. de, Roelse, P., 1992-a
A feasibility study of a perched beach concept in the Netherlands
Proc. of the 23rd Int. Conf. on Coastal Eng., ASCE, pp. 2581-2598

Ruig, J.H.M. de, Roelse, P., 1992-b
Sawaragi, T., Deguchi, I., Park, S., 1988
Experimental study on the function of submerged breakwaters to control cross-shore sediment transport on artificially nourished beaches
Coastal Engineering in Japan, vol. 31, No. 1, 1988

Sawaragi, T., 1995
Coastal Engineering – Waves, beaches, wave-structure interactions
Elsevier science, the Netherlands

Seabrook, S.T., Hall, K.R., 1998
Wave transmission at submerged rubble mound breakwaters

Seelig, W.N., 1979
Effects of breakwaters on waves: laboratory tests of wave transmission by overtopping

Shore Protection Manual, 1984

Sorensen, R.M., 1987
Evaluation of perched beach concept for shore stabilization
Coastal and port engineering in developing countries 1987, vol 1, pp. 318-326

Sorensen, R.M., Beil, N.J., 1988
Perched beach profile response to wave action
Proc. of the 21st Int. Conf. on Coastal Eng., ASCE, 1988, pp. 1482-1492

Stelling, G.S., 1984
On the construction of computational methods for shallow water flow problems
Rijkswaterstaat Communications, No. 35, The Hague, Rijkswaterstaat, 1984

Sunamura, T., Horikawa, K., 1974
Two-dimensional beach transformation due to waves
Proc. of the 14th Int. Conf. on Coastal Eng., Copenhagen, Denmark, ASCE, pp. 920-938

Sunamura, T., 1984
Quantitative prediction of beach-face slopes
Geol. Soc. of America Bulletin, Vol. 95

Svendsen, I.A., 1984
Wave heights and set-up in a surf zone
Coastal Engineering, No. 8, 1984, pp. 303-329

Tan, S.K., Chiew, Y.M., 1994
Analysis of bayed beaches in static equilibrium
Journal of waterways, Port, Coastal and Ocean Engineering, ASCE, vol. 120 (2), pp. 145-153

Tomasicchio, E., 1996
Submerged breakwaters for the defence of the shoreline at Ostia: Field experiences and comparison
Proc. of the 25th Int. Conf. on Coastal Eng., ASCE, vol. 2, pp. 2404-2417
Van der Meer, J.W., 1991
Stability and transmission at low crested structures
WL | Delft Hydraulics report 453

Van Rijn, L.C., 1993
Principles of sediment transport in rivers, estuaries and coastal seas.
Amsterdam, Aqua publications

Van Rijn, L.C., 1998
Principles of coastal morphology
Aqua publications, the Netherlands

Velden van der, E.T.J.M., 1989
Coastal engineering
Lecture notes CT5309, Delft University of Technology

Vellinga, P., 1984
A tentative description of a universal erosion profile for sandy beaches and rock beaches
Coastal Engineering, vol. 8 1984, pp. 172-184

Visser, P.J., 1984
Uniform longshore current measurements and calculations
Proc. of the 19th Int. Conf. on coastal Eng., ASCE. pp. 2192-2207

Walton, T., 1998

Perched beaches and submerged breakwaters

Wiegel, R.L., 1964
Oceanographic engineering
Prentic-Hall, Englewood Cliffs

Winter T., 1993
Profile development of a nourishment behind a submerged breakwater
master’s thesis at the Delft University of Technology

WL | Delft Hydraulics, 1983
Lido di Dante, Morphological behaviour of beach fill with underwater dam
WL | Delft Hydraulics report H 1891, January 1989

WL | Delft Hydraulics, 1989
Coastal protection plan Lido di Ostia
WL | Delft Hydraulics report H 891, April 1989

WL | Delft Hydraulics, 1990
Beach nourishment schemes for the coast of Riccione and Cesenatico
WL | Delft Hydraulics report H 725, December 1990

WL | Delft Hydraulics, 1991-a
Praktijkproef hangend strand, voorstudie
WL | Delft Hydraulics report H 1291, February 1991
WL Delft Hydraulics, 1991-b
Praktijkproef hangend strand, brainstorm
WL | Delft Hydraulics report H 1291, July 1991

WL Delft Hydraulics, 1998
Derde Haven IJmuiden; Golfdoordringingsberekeningen
WL | Delft Hydraulics report H 3288, April 1998

WL | Delft Hydraulics, 2000-a
Gibraltar beach development
Volume I: report on desk study
WL | Delft Hydraulics report H3590, June 2000

WL | Delft Hydraulics, 2000-b
Gibraltar beach development
Volume II: report on physical model testing beach schemes
WL | Delft Hydraulics report H3590, June 2000

WL | Delft Hydraulics, 2000-c
Gibraltar beach development
Volume III: report on schematic physical model tests on wave transmission
WL | Delft Hydraulics report H3590, June 2000

WL | Delft Hydraulics, 2000-d
Gibraltar beach development
Volume IV: report on additional wave climate study
WL | Delft Hydraulics report H3590, June 2000

WL | Delft Hydraulics, 2000-e
Gibraltar beach development
Volume V: report on physical model testing of beach schemes. Wide crested breakwater option
WL | Delft Hydraulics report H3590, June 2000

WL | Delft Hydraulics, 2000-a
User Manual Delft3D-FLOW, version 2.00

WL | Delft Hydraulics, 2001-b
User Manual Delft3D-WAVE, version 3.06

WL | Delft Hydraulics, 2001-c
User Manual Delft3D-MOR
Pocket perched beaches
Computational modelling and calibration in Delft3D

APPENDICES

Status: FINAL

MSc Thesis
F.J.H. Olijslagers
September 2003

Raadgevend Ingenieursbureau Lievense

wl | delft hydraulics

TU Delft
Delft University of Technology
Appendix A. Location of the project area

In addition to §1.2, this appendix describes the project area of the IJmuiden Third Harbour, being the occasion for this study. Within the breakwaters of the port of IJmuiden, the Third Harbour project is marked in red in Figure A 1. The layout of the Third Harbour and the perched beach is shown in Figure A 2. In March 2003, the Third Harbour and the perched beach were almost fully constructed as shown in the air photograph of Figure A 3. A detailed layout of the perched beach is given in Figure A 4 and a cross-section of the perched beach in Figure A 5. A photo of the actual pocket perched beach is shown in Figure A 6 for March 2003 and in Figure A 7 for May 2003. The latter also shows the recreational houses on the elevation.
Figure A 2  
IJmuiden Third Harbour design layout

Figure A 3  
IJmuiden Third Harbour and pocket perched beach during construction, March 2003
Figure A 4  IJmuiden perched beach, detailed layout
Figure A 5  IJmuiden perched beach cross-section D-D

Figure A 6  IJmuiden pocket perched beach, beach profile, March 2003

Figure A 7  IJmuiden perched beach with recreational houses, May 2003
Appendix B. Non uniformity in IJmuiden

The next simple computations show that non-uniformity of the hydraulic conditions in IJmuiden can play an important role. As discussed in §1.2, the unpredictable consequences of non-uniformity on this beach and on the performances of actual built beaches initially was the reason to investigate on this matter.

Geometry and hydraulic conditions
The local geometry and hydraulic conditions are illustrated in Figure A 8. Onderwater (2002a and 2002b) describes wave intrusion between IJmuiden’s breakwaters as calculated by WL | Delft Hydraulics (1998). In front of the submerged sheet piles, a non-uniform wave climate was found. For a 10/1-year storm, at the north side, a significant wave height of 0.75 meter approaches the perched beach under an angle of 6 degrees, see Figure A 8. At the opposite side of the 250-meter long submerged sheet piles, a significant wave of 0.85 meter approaches the shore with an angle of 16 degrees. Assuming an initial beach parallel to the submerged sheet piles and concerning the wave direction, one could expect a southward directed current. Concerning the wave height differences, one could expect northward directed currents, based on a difference in wave set-up.

Alongshore currents
This wave field generates two driving forces for alongshore currents:
The first driving force is generated by the oblique wave angle forcing a current to the south. Hereby, simplified single wave condition is used, with a direction of 310 degree at the sheet piles, entering from deep water.

\[
\frac{dS_y}{dx} = \frac{5}{16} \rho \gamma^2 (gh)^{3/2} \sin \phi_0 \frac{\gamma}{c_0} m
\]

(A.1)

where
\begin{align*}
    c_0 & = \text{wave velocity in deep water} \quad \text{[m/s]} \\
    g & = \text{acceleration due to gravity} \quad \text{[m/s}^2]\] \\
    h & = \text{water depth} \quad \text{[m]} \\
    \gamma & = \text{breaker index} \quad \text{[-]} \\
    m & = \text{beach slope, dh/dx} \quad \text{[-]} \\
    \rho & = \text{mass density of water} \quad \text{[kg/m}^3]\] \\
    \phi_0 & = \text{angle of wave approach in deep water} \quad \text{[degr]} \\
    y & = \text{alongshore distance} \quad \text{[m]} \\
    x & = \text{cross-shore distance} \quad \text{[m]}
\end{align*}
The second driving force could be generated by a difference in wave set-up, and is directed north due to higher waves at the south end of the beach, see Figure A 9.

\[
\rho g \left( h + h' \right) \frac{dh'}{dy}
\]

(A.2)

Figure A 9 Alongshore cross-section, difference in set-up and driving forces

The current itself generates a friction force, prescribed in Equation (A.3).

\[
\tau_{cwy} = \frac{\rho g V^2}{C^2} \left[ 0.75 + 0.45 \left( \frac{\xi}{V} \right)^{1.13} \right] \approx \frac{\rho g}{\sqrt{2\pi C}} \gamma \sqrt{h} \sqrt{f_w} V
\]

(A.3)

where

- \( C \) = Chézy coefficient \([m^{1/2}/s]\)
- \( \tilde{U}_0 \) = orbital motion amplitude near the bottom \([m/s]\)
- \( V \) = alongshore current velocity, averaged over the depth \([m/s]\)
- \( \xi \) = coefficient, defined by Equation (A.4) \([-\]

\[
\xi = \frac{p\kappa C}{\sqrt{g}} = C \frac{f_w}{2g}
\]

(A.4)

- \( \kappa \) = Von Karman coefficient, 0.4 \([-\]
- \( f_w \) = friction factor \([-\]
- \( p \) = proportionality factor \([-\]

Equilibrium of forces gives:

\[
\frac{dS_{xy}}{dx} + \rho g \left( h + h' \right) \frac{dh'}{dy} = \tau_{cwy}
\]

(A.5)

Resulting in the following velocity:

\[
V = \frac{5\pi}{8\sqrt{2}} \sin \varphi_0 \frac{C}{c_0} \gamma \sqrt{g} hm + \frac{\sqrt{2\pi}}{\gamma \sqrt{h}} \frac{C}{\sqrt{f_w}} \left( h + h' \right) \frac{dh'}{dy}
\]

(A.6)
In case of IJmuiden’s Third Harbour perched beach a brief computation in EXCEL tells that the southward force overrules the force generated by the difference in wave set-up, and the resulting alongshore current would be directed south. In Figure A 10 the decrease of wave height and water depth is drawn for a sloping beach, together with currents due to the oblique wave angle, difference in wave height and the resulting alongshore current related to the cross-shore distance.

One has to bear in mind the fact that this derivation is based on refutable assumptions. The goal is to convince the reader of the fact that the northward force due to difference in set-up can be of the same order of magnitude as the southward force due to the oblique incoming waves. In reality one can expect that the beach will develop to a typical pocket beach shape, as described in §2.3.3 and displayed in Figure A 40. In this equilibrium situation, the beach will lay parallel the dominant incoming wave crests. The actual beach development due to the non-uniform hydraulic conditions is still hard to predict in an analytical way and therefore computational modelling is recommended.
Appendix C. Beach shape around groins

In this appendix an extensive derivation is given that eventually leads to the formula for the accretion length of groins, $Y(t)$, that is used in §2.3.3 to estimate the equilibrium shape of beach between groins. Groins are usually narrow structures, constructed perpendicular or slightly oblique to the shoreline and extended generally beyond the low water line in the surf zone. The main function is to reduce the littoral drift in the inner surf zone, thus retaining or stabilising the beach fill. Figure A 11 shows the regions of erosion and accretion. At the lee-side, a part of the coastline is sheltered from wave attack, and diffraction takes place. This results in a difference in wave set-up. This results in a complex system of sediment transports, where transport in the direction of the groin is also possible.

![Coastline development around a groin](image)

The primary consideration for the design of the groin is the coastline development in time. To solve this problem, Van der Velden (1989), used the equations of continuity (Equation (A.7)) and motion (Equation (A.8), assuming that the changes in $\varphi$ are relatively small).

$$\frac{\partial S_x}{\partial x} + d \frac{\partial y}{\partial t} = 0 \quad (A.7)$$

$$\frac{\partial S_x}{\partial \varphi} = S_x \quad (A.8)$$

where

- $d$ = thickness of the active layer over which the changes take place [m]
- $y$ = coastline [m]
- $S_x$ = sediment transport along the coast at location $x$ [m$^3$/yr]
- $s_x$ = constant rate of change of sediment transport along x-axis [m$^3$/yr/rad]
- $\varphi'$ = angle of wave approach at depth $d$ [rad]

Transforming the equation of motion, using the chain rule, gives:

$$\frac{\partial S_x}{\partial x} = \frac{\partial S_x}{\partial \varphi} \frac{\partial \varphi}{\partial x} \quad (A.9)$$

Since $\frac{\partial \varphi}{\partial x}$ is assumed small, it can be approximated by $-\frac{\partial y}{\partial x}$, therefore:
\[ \frac{\partial \varphi}{\partial x} = -\frac{\partial^2 y}{\partial x^2} \]  

(A.10)

The angle \( \varphi \) between the wave crest at depth \( h \) and the instantaneous coastline at some time can be defined as:

\[ \varphi = \varphi' - \frac{\partial y}{\partial x} \]  

(A.11)

substitution of equations gives

\[ \frac{\partial S}{\partial x} = -s \frac{\partial^2 y}{\partial x^2} \]  

(A.12)

\[ a \frac{\partial^2 y}{\partial x^2} - \frac{\partial y}{\partial t} = 0 \]  

(A.13)

where

\[ a = \frac{s}{d} = \frac{S}{\varphi'd} \]  

(A.14)

To solve Equation (A.13), one initial and two boundary conditions are required. The initial condition is that \( y=0 \) at \( t=0 \) for all \( x \). The three boundary conditions are:

- \( S_x = S \) at \( x = -\infty \) for all \( t \);
- \( S_x = 0 \) at \( x=0 \) for all \( t>0 \).

The second condition is valid until the accretion reaches the tip of the groin. This conditions implies accretion progresses seaward always making an angle \( \varphi' \) with respect to the x-axis at the groin, \( \partial y/\partial x=\varphi' \). This means that the wave crests are parallel to the accretion coastline. Furthermore, the depth contours tend also to become parallel to the approaching waves.

The solution for that changing coastline is given by the next equation:

\[ Y(t) = \varphi' \sqrt{\frac{4at}{\pi}} \left( e^{-u^2} - u\theta\sqrt{\pi} \right) \]  

(A.15)

where

\[ u = -\frac{x}{\sqrt{4at}} \]  

(A.16)

\[ \theta = 1 - \frac{2}{\sqrt{\pi}} \int_0^u e^{-v^2} dv \]  

(A.17)

The values of \( \theta \) and the expression \( \left\{ e^{-u^2} - u\theta\sqrt{\pi} \right\} \) can be evaluated using the normal distribution.

For \( u > 2.5 \), \( \theta \approx 0 \) and \( \left( e^{-u^2} - u\theta\sqrt{\pi} \right) \leq 10^{-4} \). Therefore, accretion is less then 0.01% of the accretion near the groin. It can also be concluded that the groin has little influence on the coastline at distances more than \( 5\sqrt{at} \) upstream.

The seaward accretion length \( Y(t) \) at the groin can be solved through Equation (A.14) and Equation (A.15) and is given in Equation (A.18). It can be seen that the seaward accretion length is proportional to the square root of time.

\[ Y(t) = \varphi' \sqrt{\frac{4at}{\pi}} = 2 \sqrt{\frac{\varphi'S}{\pi d}} \sqrt{t} \]  

(A.18)
Appendix D. Physical studies involving perched beaches

In this appendix, several physical studies involving perched beaches are discussed, in addition to §2.4.4. The studies that will be treated in chronological order are:
- Santa Monica causeway project, California (USA);
- Lido di Dante (Italy);
- Sorensen and Beil’s experiments;
- Sawaragi’s flume tests;
- Lido di Ostia (Italy);
- Riccione and Cesenatico;
- DUT physical tests;
- Flemish coast;
- Singapore;
- Gibraltar beach development project.

Santa Monica causeway project, California (USA)
First Dunham (1968) explains the possible use of a perched beach concept as a solution to the freeway-routing needs. To assure the workability, more studies were needed. Chatham (1972) and Chatham et al. (1973) describe movable bed studies to aid in ascertaining feasibility and optimum design factors of the perched beach concept. Several tests were conducted with different crest heights and a wide range of wave conditions, some with a stone apron, until equilibrium was reached for both the existing beach and the perched beach. The most important conclusions by Chatham were:
- little or no beach fill material will be lost seaward of the toe structure for normal wave conditions but larger storm waves may cause erosion;
- the installation of a stone apron shoreward of the toe structure will reduce the amount of erosion;
- if the beach fill is extended a sufficient distance seaward, the toe structure serves no useful purpose;
- a three dimensional movable bed is feasible and necessary to determine the final design features of a perched beach.

An important lesson that can be drawn from these experiments is that, if the location of the submerged sill is carefully selected, a protective stone apron will significantly reduce erosion of the perched beach. Therefore it is important to locate the expected position where the sand bar is likely to form under the most severe storm condition and to place the submerged sill a short distance seaward of this position. In this way, the bar will form on top of the protective stone apron during the storm and when the milder swell waves returns, the bar will be washed shoreward again to fill the trough caused by the storm waves.

Lido di Dante
WL | Delft Hydraulics (1983) describes a study to optimize the preliminary beach design for the Lido di Dante artificial sand nourishment by scale model tests. Lido di Dante is situated in the coastal region of Emilia Romagna, at the Adriatic in Italy. The coastline was submitted to erosion, due to subsidence, decrease of river sediment discharges and gradients in alongshore transport. The mean grain diameter of the existing seabed was 195 µm. The existing profile had a slope of 1:15 around the water line and 1:200 to 1:500 seaward of the MSL –2 meter line. The preliminary beach fill design was executed as the existing profile, but moved 25 meter offshore, see Figure A 12. 100 m³/m sand was used for the nourishment and the underwater dam was made of 1 m³ synthetic bags filled with sand, see Figure A 13.
Appendix D

12 tests were conducted to investigate the behaviour of the filled beach and the effectiveness of the underwater dam, for different wave conditions and dam configurations. The following conclusions were derived:

- Offshore sediment transports are reduced due to the presence of the underwater dam;
- The influence of the dam on the wave characteristics in the inshore zone is rather small. Consequently, the beach erosion in the area around the water line is similar for both situations with and without a bar. This is probably due to the minor reduction of the wave conditions by the submerged breakwater;
- Most of the eroded beach material has been transported in an offshore direction, just landward of the dam, where a bar has been formed, see Figure A 14;
- The results for a 3-layer and a 2-layer dam, concerning beach erosion, bar formation and offshore sand loss at the dam are rather similar;
- A 3-layer dam with only one sand bag in the top layer is not stable under the applied extreme wave conditions. However, no damage occurs, if the top layer consists of two bags;
• All tests show a scour hole development seaward of the dam, see Figure A 14. The maximum depth of
the scour hole is defined as a function of time: \( h_{\text{max}} = \gamma t^{0.4} \), where \( \gamma = 0.054 \text{ m/s}^{0.4} \) and \( 0.137 \text{ m/s}^{0.4} \) for respectively intermediate and high wave conditions. With limited storm duration, the maximum scour
depth was limited to 0.65 m. It was recommended to protect the seabed just seaward of the dam against
scouring by means of an additional row of bags;
• It is expected that sand losses will increase if finer beach material is used.

![Figure A 14](image)

**Sorensen and Beil**

Sorensen (1987) and Sorensen and Beil (1988) describe conducted wave tank experiments. Five test cases
were investigated. The first consisted of 1:20 nourishment without a toe structure. The remaining four cases
were perched beach conditions with a sill located at various depths, all with a 1:20 beach slope. Irregular
waves were run for a total of 42 hours. The profile was measured after 6, 12, 18, 24 and 42 hours, after
which the beach had approximately reached an equilibrium condition.

Sorensen and Beil seem to imply that increasing the crest height of the submerged sill will decrease the
overall horizontal distance from the submerged sill to the beach face. A crest height at still water level
requires the least volume of beach fill material if the beach slope is maintained constant.

These conclusions can easily be understood based on the geometry. Three other results of the four tests that
were not sufficiently highlighted were:
• The sand lost from the perched beach was deposited just seaward of the sill. The depths at both sides of
the sill were nearly the same, see Figure A 15;
• The retreat of the still water point must be coupled to the expected storm duration. Using a storm
duration longer than will actually occur, will lead to a too conservative design;
• Though a crest height at SWL will lead to less fill material, it also means that there is little or no
swimming water and dangerous situations for swimmers can occur;
• The shoreline retreat appeared to be the smallest for the nourishment without a dam!
Appendix D

Sawaragi’s flume tests
Sawaragi (1988) conducted flume tests for a perched beach scheme. The seabed slope was 1:30 on which sand suppletions were deposited under a slope of 1:10 and 1:30, with sand of size 300 µm. The sand was nourished up till the breakwaters crest. The most interesting conclusion drawn was that increasing the crest height will not always lead to a better result. Another remarkable conclusion was that, with constant crest height to the water level, the length of the stable part of the profile around the water line increases with an increasing distance of the breakwater to the shore, i.e. with a breakwater at greater depths. At the end a relation for the crest width is deducted, but for specific conditions and therefore not generally applicable.

Lido di Ostia
WL Delft Hydraulics (1989) conducted a morphological study on the feasibility of a coastal protection plan for Lido di Ostia, Italy. Four cases have been simulated in a wave flume, see Table A 1. The design consisted of a 3 kilometre long submerged structure at MSL -4.0 m in combination with a nourishment. Due to lack of sand, mainly coarse quarry sand is used, covered with a finer layer, for better recreational purposes.

<table>
<thead>
<tr>
<th>Test name</th>
<th>Crest height (m wrt MSL)</th>
<th>Crest width (m)</th>
<th>Sediment size (µm)</th>
<th>Initial slope (-)</th>
<th>Beach slope after simulation (-)</th>
<th>Retreat SWL –1.0m (m)</th>
<th>Offshore losses (m³/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>1.50</td>
<td>10</td>
<td>300-500</td>
<td>1:20</td>
<td>1:20</td>
<td>28</td>
<td>25.2</td>
</tr>
<tr>
<td>T2</td>
<td>2.00</td>
<td>30</td>
<td>300-500</td>
<td>1:20</td>
<td>1:16</td>
<td>23</td>
<td>19.6</td>
</tr>
<tr>
<td>T3</td>
<td>1.50</td>
<td>15</td>
<td>700-900</td>
<td>1:20</td>
<td>1:40</td>
<td>5</td>
<td>5.2</td>
</tr>
<tr>
<td>T4</td>
<td>2.20</td>
<td>15</td>
<td>300-500</td>
<td>1:40</td>
<td>1:16</td>
<td>19</td>
<td>32.0</td>
</tr>
</tbody>
</table>

Table A 1 Lido di Ostia test results
The result of the physical test of the preferred scheme is shown in Figure A 16.

Figure A 16  
Lido di Ostia physical test

Besides the physical tests, mathematical computations are conducted with the Crostran computational model, see Figure A 17 for the results of the preferred scheme.
The minor offshore losses of test T3 are not unexpected, due to the much larger grain size and an initial slope that was less steep than the equilibrium slope. The coarse nourishment material led to 10% of the offshore losses compared with the finer materials. It is worth reviewing if in another stable design the slope cannot be designed steeper and the crest height lower for swimming safety. The other three initial designs were far from their equilibrium profile. That explains the larger offshore losses. Figure A 18 shows the preferred design scheme.

Still, one expected erosion over the entire length of the nourishment due to offshore losses. Due to alongshore gradients at the updrift side of the nourishment, some additional erosion is expected there. In Appendix F the actual performance of this scheme will be evaluated.
Riccione and Cesenatico
In order to optimize the beach nourishment schemes for the coasts of Riccione and Cesenatico, WL | Delft Hydraulics (1990) performed flume tests. The nourishment scheme consisted of a beach fill, which was retained by a new submerged barrier, an existing barrier more offshore and a system of submerged barriers perpendicular to the coast. The mean used grain size was 180µm.

For two initial layouts, tests were performed with three different wave conditions. Based on the information gained from these tests a design profile for the beach fill was defined, see Figure A 19 and Figure A 20.

The following conclusions were drawn:
- Especially the winter conditions, but also intermediate wave conditions have a significant impact on the design of the nourishment scheme. During these conditions seaward migrating bars are developed, which, as they move over the barrier can cause considerable sediment loss;
- By applying a beach fill that is better adapted to the erosive wave conditions, the structural retreat due to offshore transport can be reduced;
- Scour effect can be expected on both sides of the barrier. The side on which the scouring occurs depended on the wave conditions. As a result of intermediate conditions, a scour hole was found on the landward side of the barrier, while scouring at the seaward side was found after a period of storm conditions. The scouring at the seaward side may go deeper than measured and therefore a bed protection is recommended;
- To reduce initial losses it is advised to adjust the nourished profile as much to the storm profile;
- For Cesenatico an option was not to build a new barrier, see Figure A 20.

![Figure A 19 Recommended scheme for Riccione](image-url)
Groenewoud et al. (1996) have investigated the effect of a submerged breakwater on profile development. Although the initial profile is a submerged breakwater without nourishment, due to a steeper developing slope the end bathymetry has the qualities of a perched beach and will therefore be treated. Six series of physical tests have been carried out at Delft University of Technology with different wave conditions, \( F/H_s = 0.75 \) to 1.50. The test layout is shown in Figure A 21, scale 1:15.

---

**Figure A 20**  
Recommended scheme for Cesenatico

**DUT physical tests**

Groenewoud et al. (1996) have investigated the effect of a submerged breakwater on profile development. Although the initial profile is a submerged breakwater without nourishment, due to a steeper developing slope the end bathymetry has the qualities of a perched beach and will therefore be treated. Six series of physical tests have been carried out at Delft University of Technology with different wave conditions, \( F/H_s = 0.75 \) to 1.50. The test layout is shown in Figure A 21, scale 1:15.

**Figure A 21**  
Wave flume initial layout
Figure A 22 shows the profile development with the submerged breakwater. Erosion around the waterline takes place and is mainly accumulated near the breakwater. In between a bar is formed. It might be possible that the equilibrium profile is not yet established.

![Profile development for the physical tests with the submerged breakwater](image)

Figure A 22 Profile development for the physical tests with the submerged breakwater

Figure A 23 shows the effect of a submerged breakwater on the development of a beach with an initial 1:15 slope with and without a submerged breakwater. It can be seen that the submerged breakwater has a noticeable impact on profile development.

![Comparison between initial profile and developed profile with and without the breakwater](image)

Figure A 23 Comparison between initial profile and developed profile with and without the breakwater

Measured wave heights and profile developments have been compared with Unibest-TC (2DV mathematical model) computations and showed promising agreement. In addition, 3D movable bed tests are conducted with offshore breakwaters with gaps between them, but will not be treated here.
**Flemish coast**

For the erosive Flemish coast, see Figure A 24, a first beach replenishment of 7,000,000 m³ sand over 8 km reach was executed in 1977. A second nourishment of 730,000 m³ sand over a reach of 2.9 km was already necessary in 1986. Again, intense local erosion at the beach of Knokke-Zoute transported the total sand volume out of the region in a period of only 5 years.

![Flemish North Sea coast near Knokke](image)

*Figure A 24 Flemish North Sea coast near Knokke (source: internet)*

Therefore, a research program was set up to explore a basic understanding of the local beach morphology. Due to the complex interaction of wave-induced cross-shore transport, alongshore tidal drift and the impact of the breakwater obstruction by the harbour extension of Zeebrugge, a traditional nourishment would not provide a durable solution for the Flemish coast.

![Investigated beach suppletion profiles](image)

*Figure A 25 Investigated beach suppletion profiles (source: internet)*
Initially, physical model tests (scale 1:25) in a wave flume were conducted, see Figure A 25 for the profiles and Figure A 26 for the cumulative volume variation in cross-shore direction. Comparisons with detailed field registrations show that the complex wave-alongshore tidal current interaction is not represented by this model.

![Figure A 26](image.png)

*Figure A 26 Resulting volume balance development (source: internet)*

Therefore, an extended 3D physical model (scale 1:60) was explored where local complex hydrodynamics could be simulated, which was confirmed by the good agreement between physical test results and the in situ data. The, for the time being, stable profile with horizontal terraces no longer stayed stable under the combined wave-flow impact. Therefore it was concluded that the traditional beach nourishment scheme will not resolve the structural erosion.

The alternative solution, a perched beach protected by a seaward gravel foot, was identified as a more suitable coastal protection system for the area. While the resulting sand volumes for both "traditional" sand suppletions (the 1986-reference and the new design with horizontal terraces) had a quite similar magnitude, the beach nourishment scheme with a gravel toe at the foreshore reduced the erosive sand volume with 40%, see Figure A 28.

![Figure A 27](image.png)

*Figure A 27 Cumulative sand balances (source: internet)*
Singapore
Chin et al. (2000, from Chia 2001) also conducted five wave flume tests for a proposed perched beach in Southern Island of Singapore. A model to prototype scale of 1:25 was chosen and the model was constructed according the Froude Number modelling law. The prototype perched beach consisted of a submerged sill with crest height at MLW –2.5 m (CD 0.0m), a beach width of 174 m and a gradient of 1:18. A 55-meter wide stone apron was placed just shoreward of the submerged sill. Three test were conducted with still water level fixed at high tide (CD +1.4m), mid tide (CD +1.4m) and low tide (CD 0.0m) using the 100-year wave height of 1.6 m. The tests were conducted until no significant profile changes were recorded. Two other tests were conducted with water level changing according to a 12-hour semi-diurnal tide and an initial storm duration of 3 hours (1.6 m wave height) followed by 9 hours of smaller wave conditions (1.0 m wave height, 5-year return period).

The conclusions reached were:

- The armour stones and stone apron were stable under storm wave conditions;
- The beach sand material remained within the perched beach and showed negligible tendency to be moved offshore of the submerged sill;
- The armour stones of the submerged sill and stone apron trapped sand within the void spaces, which constituted an initial erosion loss of the beach.

Further examination of the measured profiles for the tests with high and mid tides shows that the erosion of the beach slope resulted in bar formations which were located 81 m and 71 m (prototype distances) from the still water line respectively. However, in the test conducted with the still water level fixed at mean low tide (CD 0.0m), no bar formation was recorded. The main reason could be the high turbulence caused by the wave breaking on top of the stone apron when the water level was low. Therefore, any sand deposit on the stone apron will therefore be washed onshore. For the two conducted tests with varying water levels, it was found that milder waves would move the sand deposit on the stone apron due to the initial 3 hours storm wave onshore. The final profile resembled the equilibrium beach profile with a steeper upper slope gradient 1:11 and a gentler lower slope gradient 1:22. Chin et al. found that the measured upper slope gradient of 1:11 matched very closely with the empirical relationship of \[ \tan \beta = 0.45(d_{50}/H_0)^{0.5}(H_0/L_0)^{0.3} \] by Sunamura and Horikawa (1974).

Gibraltar Beach development
WL | Delft Hydraulics (2000-a,b,c,d,e) was commissioned by the Gibraltar Land Reclamation Company to evaluate the proposed perched beach schemes of Eastern Beach, Catalan Bay and the Rubble Tip area in Gibraltar in two stages. They will be treated separately.

For Eastern Beach and Catalan Bay the proposed scheme had a toe structure placed in 3.5-m deep water with a crest height of MSL -1.0 m, see Figure A 28. The plane beach had a slope of 1:40 and started at the breakwater at MSL –3.0 meter.

Figure A 28 Breakwater cross-section to be evaluated

In the first stage, the scheme was evaluated by the mathematical UNIBEST-DE program. It was concluded that the proposed plane 1:40 profile for Eastern Beach/Catalan Bay remained stable during storm wave events. However, due to the limited sand buffer in the upper part of the profile WL | Delft Hydraulics recommended to increase the beach slope above MSL to 1:20.
In the second stage, the scheme was verified by physical modelling. Three tests were conducted. The first was without a breakwater and beach fill and with the existing profile. The second was a breakwater concept with a crest height at MSL –1.0 and –0.5 m and beach slopes of 1:40 and 1:20. Here the breakwater did not dissipate enough energy and led to an unacceptable amount of offshore losses. The third test was a sill concept with a crest at MSL –4.5m and a beach slope of 1:90. This option was more stable, but still resulted in annual losses of 10% beach fill, see Figure A 29.

![Figure A 29](image)

**Figure A 29**  
Temporary best beach profile, Eastern beach

Two additional tests were conducted to investigate the influence of the crest width. At the first test, the crest was 15m wide and at a height of MSL –0.7m. The beach started at MSL –2.2m and had a Dean profile (A=0.09, see §2.2.5). The profile tended to become steeper to 1:70 and sediment losses were acceptable, see Figure A 30. Under the same conditions but with a crest width of 9 meter the maximum transport rates increased with a factor of two. The recommended beach profile is shown in Figure A 30 and the breakwater cross-section is shown in Figure A 31.
For the Rubble Tip area the proposed scheme had a toe structure placed in 3.5-meter deep water with a crest height of MSL -1.0 meter, see Figure A 32. The plane beach had a slope of 1:30 and started at the breakwater at MSL –3.0 meter.

Figure A 30 Final recommended beach profile, Eastern beach (mirror image)

Figure A 31 Final recommended cross-section of the Eastern beach breakwater

Figure A 32 Breakwater cross-section to be evaluated for the Rubble tip area
In the first stage, the scheme was evaluated by the mathematical UNIBEST-DE program. It was concluded that the proposed plane 1:30 profile for the Rubble Tip area was not stable. In the modified layout the crest height is kept at MSL –1.0m and the breakwater was shifted 50m seaward with an initial profile similar to the storm wave profile recommended for Eastern Beach/Catalan Bay (1:20 above MSL, 1:40 below MSL).

In stage 2, the beach scheme is verified and optimised by physical modelling. Here it appeared that the offshore losses were much higher. Three tests were conducted. For the breakwater option with a crest at MSL –1.0m unacceptable losses occurred. A sill option, crest at MSL –2.0 m, still gave unacceptable losses. With a deeper sill, MSL-4.5m and with coarser sand (800 µm instead of 220 µm) the annual losses were reduced to less than about 1% and a 1:70 slope was established, see Figure A 33.

![Figure A 33 Recommended beach profile, Rubble tip area](source)

Two additional tests were conducted to investigate the influence of the crest width. At the first test, the crest was 15m broad and at a height of MSL –0.7m. The beach started at MSL –2.7m and had a Dean profile (A=0.135). The grain size had changed to 350 µm. Now the beach tended to become more gentle (1:65). Again, under the same conditions, but with a crest width of 12m, the transport rates increased and here also the beach became gentler. The recommended beach profile is shown in Figure A 34 and the breakwater cross-section is shown in Figure A 35.
Figure A 34  Final recommended beach profile, Rubble tip area (mirror image)

Figure A 35  Final recommended cross-section of the Rubble tip area breakwater
Appendix E. Mathematical studies involving perched beaches

This appendix describes mathematical studies involving perched beaches, which are discussed in §2.4.4. The studies that are discussed here in chronological order, are, in sequence of treatment:

- Tien Honderd polder, Cadzand, the Netherlands;
- Winter’s thesis;
- Chia’s thesis;
- Third Harbour IJMuiden.

Tien Honderd polder, Cadzand, the Netherlands

WL | Delft Hydraulics (1991-a,b) describe a feasibility study for a study in practice to reduce the uncertainties concerning the perched beach principle. 'Tien honderd polder', a very erosive coastal compartment in Cadzand, the Netherlands showed to be the most promising location for a test case study. Previous studies recommended the use of the perched beach principle, but a practical study had to eliminate the uncertainties.

The area is characterised by the severe conditions; the tide varies globally between NAP -2.0 m and NAP +1.8 m (i.e. M.L.W. and M.H.W.), significant wave heights up to 2.5 m occur regularly. Tidal driven currents varied between 0.95 and 1.75 m/s in the gully and between 0.05 and 0.60 at the gully edge, depending on direction and being spring tide or neap tide.

From Durosta computations the following conclusions are drawn:

- For lower crest heights, the influence on dune erosion is negligible in the first place. It did reduce offshore erosion and will therefore reduce dune erosion for a longer period of time;
- Onshore shift of the breakwater with constant height, i.e. increasing crest height, reduced the offshore losses, but had a marginal effect on dune erosion, erosion of the gully edge increased and the useable beach width decreased. A dam located at the gully edge was therefore preferable, see Figure A 36;
- During storm conditions, a scour hole developed at the seaward side of the breakwater. A toe protection was therefore recommended.

Other conclusions are that:

- The presence of a gully close to the shore makes this test area not representative for general use in other coastal compartments;
- The presence of the gully did make it ultimately suitable for an underwater dam, because of barely any onshore transport without a submerged dam. The submerged dam will not block of any onshore transport and can only retain the offshore transports.

Considerations of different aspects indicated that the preferred perched beach scheme, see Figure A 36, was roughly as expensive as repeated beach nourishment solely. De Ruig and Roelse (1992-a,b) underscore the considerable risks of loss of functions and valuable interests within the adjacent coastal sections.
Winter’s thesis
Winter (1993) investigated the profile development of a nourishment behind a submerged breakwater using the cross-shore profile development simulation program UNIBEST-TC. It appeared that the most important processes were the change in wave height, mean wave period and the transition zone width. These processes resulted in a steeper slope, compared to the reference slope without a submerged breakwater. It is concluded that in this case the protected nourishment is economically better than the unprotected one. Of two breakwaters, the one with a lower crest is recommended, see Figure A 37.
Chia's thesis
Chia (2001) sets out a conceptual design of a pocket perched beach in deep water at Marina East, Singapore. Cross-shore modelling was conducted with Unibest-TC. With a maximum beach width of 150 m, the most suitable crest height of the sill was at MSL −4.6 m, with a starting beach slope at 1:17. A 30-meter wide stone apron, measured from the outer edge of the sill, was recommended to prevent sediment losses from the beach into the 20m deep water. Figure A 38 and Figure A 39 show the layout and cross-section of the design. The most suitable sediment size $D_{50}$ was 500 to 700 µm. In the long term the beach reached an equilibrium profile that could be approximated by Dean’s power curve $h = ax^{2/3}$, with the starting point at the mean high water spring.

![Appendix E](image)

**Figure A 38**  Layout and cross-section of Marina East perched pocket beach

**Figure A 39**  Cross-section view of Marina East perched pocket beach
Appendix E

Third Harbour, IJmuiden
Lievense Consulting Engineering has commissioned Alkyon to evaluate the perched beach scheme with the future losses as mean criterion. The beach was subject to fluctuating water levels. The Onderwater (2002-b) study describes the assessment of the final dynamic equilibrium profile. This is done by interpretation of 2D Unibest-TC and Durosta computations. It appeared that with the use of 220 µm sand, no beach would rise above the NAP level, see Figure A 40. An additional analysis led to the conclusion that the use of 350 µm and 500 µm sand would lead to a steeper beach profile and enables a beach profile above the NAP level.

Figure A 40 Equilibrium position of the perched beach with the use of 220 µm sand
Appendix F. Performance of prototype perched beaches

In addition to §2.4.4, performances of prototype perched beaches will be discussed in this appendix. In sequence of treatment these perched beaches are:
- Cape Cod, Massachusetts (USA);
- Slaughter beach, Delaware (USA);
- Emilia Romagna Coast (Italy);
- Keino-Matsubaru beach (Japan);
- Lido di Ostia (Italy).

Cape Cod, Massachusetts

Gutman (1979) describes four perched beach projects on Cape Cod. Sand-filled nylon bags are used and the development was monitored. Figure A 41 shows the scheme. In general, all sandbags were durable enough to withstand the local breaking waves and ice cover.

![Figure A 41 Cap Cod design scheme](image)

Although a significant perch developed at two of the sites, long duration or repeated storms with no intervening period of accretion completely eroded the perch, see Figure A 42. Opposed to similar project in the Chesapeake Bay the perched beach erosion technique on Cape Cod provided very little bank protection, yet some widening of the recreational beach took place.

![Figure A 42 Profile development Cape Cod](image)
Slaughter beach, Delaware
Douglas and Weggel (1987) describe the performance of an ocean scale perched beach in Slaughter, Delaware (USA), completed in 1979. A 300-meter long sill, 75 meter offshore, was constructed with a crest level at mean low water. The existing beach level was 0.5 meter lower. At both ends, groins were constructed. The enclosure was filled with 20,000 m³ of coarse sand. After construction, beach profile changes were documented by surveying.

An interesting feature of the beach fill response was that the rate of volume loss from inside the sill enclosure appeared fairly constant in time. Four years after the completion of the fill, the volume of sand within the sill was back to what it was before the fill. Overtopping waves scoured the fill in the area immediately behind the sill. The sill did not function as a perch, but rather became a low breakwater that reduced the wave energy reaching the beach (Walker and Tatami, 1987). Although the total volume of sand had reduced to its pre-fill condition, the sand was distributed much differently throughout the enclosure.

Emilio Romagna Coast
The Emilio Romagna Coast, Italy, extends along the Adriatic for about 135 km. Erosion problems of the sandy coast required measures. Liberatore (1992) describes a few examples of systems of submerged breakwaters. First and second generation works are distinguished.

For the first generation works, one m³ bags were used to build low submerged breakwaters, see Figure A 43. From a functional point of view, the surveys showed general improvement. For few beaches, the results were unsatisfactory due to a lack of submerged groins connected to the submerged barriers. From a structural point of view, considerably scouring at the beach side of the barriers was observed, also as damages to the structures, due to displacement or breaking of the bags, see Figure A 44.

Figure A 43 Typical Emilio Romagna coast cross-section

Figure A 44 Profile development Emilio Romagna
For the second-generation works, the quantity of the beach fill was larger and the barriers consisted of larger sections and larger sandbags of two m³ instead of one m³. The importance of transversal elements (groins) has been recognised for stability of the fill and is now always provided. As regards the location depth for barriers, 2.5 and 3.0 m are now considered preferable whereas previously barriers were built in water depths up to 3.5 meter.

Figure A 45  Typical beach protection system

Figure A 45 shows a typical sketch of a protective system for beach nourishment used in the Emilia Romagna region, with submerged barriers and submerged groins. Figure A 46 gives an air view of an executed nourishment with the use of submerged barriers and groins in Pellestrina.

Figure A 46  Pellestrina beach protection works
Keino-Matsubara beach, Japan
Chia (2001) describes another large scaled perched beach in Keino-Matsubara beach in Japan, which was completed in 1983. An 80 meter long submerged sill with a crest height of MLW –2.0 m was constructed 75 m from the existing coastline, see Figure A 47. The existing seabed level at the sill was 5 to 6 m below MLW. No end groins were constructed. 5,000 m$^3$ of sand was pumped behind the submerged sill, one year after the construction of the submerged sill.

Deguchi and Sawaragi (1986) reported that 30% of the beach fill were lost just after 2 months of the fill placement. Most of the sand was actually redistributed downdrift of the perched beach due to alongshore transport. It was also concluded that “the amount of borrow sand moved in the alongshore direction surpasses the amount of sand transported in the cross-shore direction”.

Lido di Ostia, Italy
The initial beach profile after construction (1989-1991) is shown in Figure A 48.
After construction, the beach has been monitored. An analysis is given by Ferrante et al. (1992). The formation of a steep ridge of coarse sediments is observed near the waterline, which is common for low tide coasts as found in the Mediterranean. The emerged beach has elevated, while the submerged beach has generally deepened, keeping more or less the same slope, see Figure A 49. Offshore losses were even less than predicted. The southward alongshore transport caused the predicted maximum 20-30 m shoreline advance (at southern end) and retreat (at northern end). Along the adjacent shores no adverse effects have been observed. Pilarczyk and Zeidler (1996) in fact only summarise the article of Ferrante et al. (1992) that ends by the statement that final conclusions will have to be drawn after a longer monitoring period.

Tomasicchio (1996) describes a non-negligible erosion phenomenon, started at the updrift side. The high periodic maintenance cost resulted in a 700-meter long 0.5-meter submerged breakwater to defend the neighbour part of the coast. In 1995 the width of the emerged beach at the north side is reduced to the same size before the construction works! The submerged beach slope has become steeper to 1:10, only due to the much coarser sediment. The erosion has led to a sinusoidal profile, see Figure A 50. Erosion at the landward side of the breakwater up to SWL –4.0 m, the fixed breakwater extending to SWL –1.5 m, toe erosion of the breakwater up to SWL –5.5 m and a large sand deposit further seaward up to SWL –2.0 m. It has to be stated that it is not clear if this observation in 1995 shows signs of stabilisation.
Appendix G. Delft3D-WAVE

This appendix gives a conceptual description of SWAN, in addition to §3.3.

Computational grid
The computational grid is a grid in four dimensions, $x$, $y$, $\theta$, $\sigma$- space. The computational grid in $x$-, $y$-space should be larger than the area where the user wants to know the wave conditions by disturbances from the boundaries. The spatial resolution should be sufficient to resolve relevant details of the wave field. For the computational spectral grid, the frequency space and directional space have to be defined. The frequency space is defined as the minimum and maximum frequency and the frequency resolution. The directional space can be less than $360^\circ$ when the user wants to reduce computational time.

Action balance equation
In SWAN, the waves are described with the two-dimensional wave action density spectrum. The action density is equal to the energy density divided by the relative frequency: $N(\sigma, \theta) = E(\sigma, \theta)/\sigma$. The evolution of the wave spectrum is described by the spectral action balance equation:

$$\frac{\partial}{\partial t} N + \frac{\partial}{\partial x} c_x N + \frac{\partial}{\partial y} c_y N + \frac{\partial}{\partial \sigma} c_\sigma N + \frac{\partial}{\partial \theta} c_\theta N = \frac{S}{\sigma}$$

(1) (2) (3) (4) (5) (6)

where

$N(\sigma, \theta) = \text{action density} [\text{N/m/s}]$

$$N(\sigma, \theta) = \frac{E(\sigma, \theta)}{\sigma} \quad (A.20)$$

$c_x, c_y = \text{propagation velocities in x- and y-space} [\text{m/s}]$

$c_\sigma, c_\theta = \text{propagation velocities } \sigma\text{- and } \theta\text{ space} [\text{m/s}]$

$\sigma = \text{relative frequency} [\text{s}^{-1}]$

$\theta = \text{wave direction}[^\circ]$

$S = \text{source term} [\text{N/m}]$

$E(\sigma, \theta) = \text{wave energy density} [\text{J/m}^2]$

The following components can be underestimated in the spectral action balance equation:

(1) = the local rate of change of action density in time
(2),(3) = propagation of action in geographical space
(4) = shifting of the relative frequency due to variations in depths and currents
(5) = depth-induced and current induced refraction
(6) = source term

The source term in terms of energy density represents the effects of generation, dissipation and non-linear wave-wave interactions. Generation by wind is not taken into account. The dissipation term can be divided into three different contributions. The first one, white capping, is not taken into account, because this fraction is negligible without wind and will reduce computing time. The bottom friction term can generally be represented as:

$$S_{d_b} (\sigma, \theta) = -C_{bottom} \frac{\sigma^2}{g^2 \sinh (kd)} E(\sigma, \theta) \quad (A.21)$$

where

$C_{bottom} = \text{bottom friction coefficient, either defined by JONSWAP, Collins or Madsen} [-]$

$k = \text{wave number} [\text{m}^{-1}]$

$d = \text{water depth} [\text{m}]$

The third dissipation term is implemented in the next formula by the factor $D_{tot}$, the rate of dissipation of the total energy due to wave breaking according to Battjes and Janssen (1978).
\[
S_{dc,br}(\sigma, \theta) = -\frac{D_{tot}}{E_{tot}} E(\sigma, \theta) \tag{A.22}
\]

where
\[
E_{tot} = \text{total wave energy} \quad [J/m^2]
\]
\[
D_{tot} = \text{rate of dissipation of the total energy due to wave breaking according Battjes and Janssen (1978)} \quad [-]
\]
\[
D_{tot} = -\frac{1}{4} \alpha_{BJ} Q_b \left( \frac{\sigma}{2\pi} \right) H_m^2 \tag{A.23}
\]

in which
\[
\alpha_{BJ} = \text{Battjes and Janssen dissipation coefficient due to wave breaking} \quad [-]
\]
\[
Q_b = \text{fraction of breaking waves} \quad [-]
\]
\[
\frac{1 - Q_b}{\ln Q_b} = -8 \frac{E_{tot}}{H_m^3} \tag{A.24}
\]

in which
\[
H_m = \text{maximum wave height that can exist at a given depth} \quad [m]
\]

Non-linear wave-wave interactions can be separated in two terms. In deep water, quadruplet wave-wave interactions dominate the evolution of the spectrum. Since here only shallow water is concerned and quadruplet wave-wave interactions are negligible, they are de-activated to reduce computing time.

In very shallow water triad wave-wave interactions transfer energy from lower frequencies to higher frequencies. To model this, the Lumped Triad Approximation (LTA), derived by Eldeberky and Battjes (1996), is used in SWAN.

**Numerical background**

SWAN uses implicit upwind schemes in both geographical and spectral space, supplemented with a central approximation in spectral space. This eventually leads to the next discretisation:

\[
\begin{align*}
\left[ \frac{c_x N_{i,j} - [c_x N]_{i,j-1}}{\Delta x} \right]^n_{i,j,i_x,i_y} + & \left[ \frac{c_y N_{j,i} - [c_y N]_{j,i-1}}{\Delta y} \right]^n_{i,j,i_x,i_y} \nonumber \\
\left[ \frac{(1-\nu)(c_x N)_{i,j+1} + 2\nu(c_x N)_{i,j} - (1+\nu)(c_x N)_{i,j-1}}{2\Delta \sigma} \right]^n_{i,j,i_x,i_y} + & \left[ \frac{(1-\eta)(c_\theta N)_{i,j+1} + 2\eta(c_\theta N)_{i,j} - (1+\eta)(c_\theta N)_{i,j-1}}{2\Delta \theta} \right]^n_{i,j,i_x,i_y} = \frac{S}{\sigma} \gamma^n_{i,i_x,i_y,i_z,i_\theta}
\end{align*}
\]

where
\[
i_x, i_y, i_z, i_\theta = \text{grid counters} \quad [-]
\]
\[
\Delta x, \Delta y, \Delta \sigma, \Delta \theta = \text{increments in geographical space and spectral space respectively} \quad [m]
\]
\[
\nu, \eta = \text{coefficients to determine the degree to which the scheme in spectral space is upwind or central} \quad [-]
\]

The boundaries in SWAN are fully absorbing for wave energy that is leaving the computational domain or crossing a coastline.

With SWAN, one can use a curvilinear computational grid. Due to the directional spreading there are disturbed regions at the boundaries, see Figure 3-7. Disturbances should not enter the area of interest.
Appendix H. Delft3D-FLOW

This appendix gives a conceptual description of the FLOW module, in addition to §3.4.

FLOW equations

The motion of fluid is governed by the laws for conservation of mass and conservation of momentum. In a quasi-three-dimensional modelling system like Delft3D, the continuity equation and momentum equation are simplified in a depth-averaged approach and solved in two steps. First, the depth averaged continuity and momentum equations are solved. Secondly, the water depth is divided in a number of depth layers and the same simplified continuity and momentum equations are applied for each depth layer. Herefore the total water depth is replaced by the layer thickness and the vertical velocities at the seabed and the surface by the vertical velocities at the underside of the layer and the upper side of the layer.

The depth averaged continuity equation for a Cartesian rectangular co-ordinate system can be formulated as follows:

\[
\frac{\partial \zeta}{\partial t} + \frac{\partial (d + \zeta) U}{\partial x} + \frac{\partial (d + \zeta) V}{\partial y} = 0
\]

(A.26)

The depth averaged momentum equation, formulated equation in a Cartesian rectangular co-ordinate system, in x-direction is given by:

\[
\frac{\partial u}{\partial t} + \frac{\partial u}{\partial x} U + \frac{\partial u}{\partial y} V + \frac{\partial \eta}{\partial x} - fV + \frac{\tau_{bx}}{\rho_w (d + \eta)} - \frac{F_x}{\rho_w (d + \eta)} - \nu \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) = 0
\]

(A.27)

In y-direction, the momentum equation yields:

\[
\frac{\partial v}{\partial t} + \frac{\partial v}{\partial x} U + \frac{\partial v}{\partial y} V + \frac{\partial \eta}{\partial y} - fU + \frac{\tau_{by}}{\rho_w (d + \eta)} - \frac{F_y}{\rho_w (d + \eta)} - \nu \left( \frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) = 0
\]

(A.28)

Comp.: (1) (2) (3) (4) (5) (6) (7) (8)

where

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>d</td>
<td>water depth below the reference plane [m]</td>
</tr>
<tr>
<td>( \eta )</td>
<td>water level variation above the reference plane [m]</td>
</tr>
<tr>
<td>f</td>
<td>Coriolis parameter, ( f = 2 \Omega \sin \phi ) [s(^{-1})]</td>
</tr>
<tr>
<td>( \Omega )</td>
<td>angular rotation of the earth = 7.27*10(^{-5}) [rad/s]</td>
</tr>
<tr>
<td>( \phi )</td>
<td>latitude [degr]</td>
</tr>
<tr>
<td>( F_{x,y} )</td>
<td>x- and y-component of external forces [N/m(^2)]</td>
</tr>
<tr>
<td>u, v</td>
<td>depth averaged velocity [m/s]</td>
</tr>
<tr>
<td>( \nu )</td>
<td>diffusion coefficient (eddy viscosity) [m(^2)/s]</td>
</tr>
<tr>
<td>g</td>
<td>gravity acceleration [m/s(^2)]</td>
</tr>
<tr>
<td>( \tau_{bx,by} )</td>
<td>x- and y-component of the bed load shear stress [N/m(^2)]</td>
</tr>
<tr>
<td>( \rho_w )</td>
<td>water density [kg/m(^3)]</td>
</tr>
</tbody>
</table>

The following components can be underestimated in the momentum equation:

(1) = velocity gradient
(2), (3) = advection terms
(4) = barotropic pressure gradient
(5) = Coriolis force
(6) = bottom stress term
(7) = external force term
(8) = viscosity term
Appendix H

Boundary conditions

Kinematic boundary condition: Vertical velocity is zero near the surface and the bottom.

\[ w_{\sigma=1} = 0 \]  \hspace{1cm} (A.30)

\[ w_{r=0} = 0 \]  \hspace{1cm} (A.31)

Bed boundary for the momentum equation:

\[ \vec{t}_b = \frac{g \rho_0 \vec{u}_b \left| \vec{u}_b \right|}{C_{3D}^2} \]  \hspace{1cm} (A.32)

where

\[ \left| \vec{u}_b \right| = \text{magnitude of the horizontal velocity in the first layer just above the bed} \] [m/s]

\[ C_{3D} = \text{3D-Chézy coefficient} \] [m^{1/2}/s]

\[ C_{2D} = \text{2D-Chézy coefficient, according Chézy, Manning or White-Colebrook} \] [m^{1/2}/s]

\[ k_s = \text{Nikuradse roughness length} \] [m]

\[ z_0 = \text{bed roughness height} \] [m]

Grid

For every computational cell on the horizontal plane a water level point, depth points and velocity points in both x- and y- direction are given, though not at the same place. Figure A 51 shows the location of the different points. To determine the values at intermediate points, interpolation is used. In Sediment Online, water level points instead of bottom points are used to solve the Navier Stokes equation. For three-dimensional computations, a \( \sigma \)-grid is used. A number of layers can be defined and each layer thickness is determined by the relative layer thickness, the bottom depth and the water level.

Wave-current interaction

Several processes that are accounted for in a wave-averaged manner are:

- Vertical mixing processes, enhanced due to turbulence generated near the surface by whitecapping and wave breaking and near the bottom due to energy dissipation in the bottom layer;
- Net mass flux, which has some effect on the current profile, especially in cross-shore direction;
- Alongshore currents and cross-shore set-up in the surf zone, generated due to variations in the wave-induced momentum flux (radiation stress);
- The bed shear stress is enhanced; this affects the stirring up of sediment and increases the bed friction.
The general procedure to derive the wave-induced forces is as follows:

1. Average the continuity- and momentum equation over the wave period

The momentum equation in x-direction, averaged over the wave motion and expressed in Cartesian co-ordinates is given by:

\[
\frac{\partial \overline{u_j}}{\partial t} + \overline{u_i} \frac{\partial \overline{u_j}}{\partial x_j} + \ldots + g \frac{\partial \zeta}{\partial x_j} - \frac{1}{\rho} \frac{\partial \overline{\tau_{ij}}}{\partial x_i} = F_j
\]  

(A.35)

where

- \(i, j = \{1, 2, 3\}\)
- \(\overline{u_j}\) = wave-mean velocity component \([\text{m/s}]\)
- \(\zeta\) = wave –mean free surface elevation \([\text{m}]\)
- \(\overline{\tau_{ij}}\) = components of the wave-averaged normal stress tensor \([\text{N/m}^2]\)
- \(F_j\) = wave-induced force that remains after averaging the momentum equation over the wave period \([\text{N/m}^2]\)

\[
F_j = \frac{\partial S_{ij}}{\partial x_j}
\]  

(A.36)

where

- \(S_{ij}\) = radiation stress tensor \([\text{N/m}]\)

2. Express the residual terms that remain compared to the case without waves in terms of the wave properties; \(S_{ij}\) can be expressed in terms of wave parameters, using the wave propagation model.

Since the above given procedure can only be applied when the mean motion is uniform with the depth, and this mostly does not apply, the Generalised Langrangian Mean (GLM) method of Andrews and McIntyre (1978) is used. The relation between the GLM velocity and the Eulerian velocity is given by:

\[
\overrightarrow{u}_{\text{GLM}} = \overrightarrow{u}_{\text{E}} + \overrightarrow{u}_{S}
\]  

(A.37)

where

- \(\overrightarrow{u}_{\text{GLM}}\) = GLM- velocity vector \([\text{m/s}]\)
- \(\overrightarrow{u}_{\text{E}}\) = Eulerian-velocity vector \([\text{m/s}]\)
- \(\overrightarrow{u}_{S}\) = Stokes drift vector \([\text{m/s}]\)

Wave current interaction terms:

- **Forcing by radiation stress gradients:**

The right-hand side of Equation (A.35) can be written as:

\[
F_j = \frac{Dk_j}{\omega}
\]  

(A.38)

where

- \(D\) = total energy dissipation due to waves \([\text{m}^1]\)
- \(k_i\) = wave number in i-direction \([\text{m}^{-1}]\)
- \(\omega\) = wave frequency \([\text{rad/s}]\)

For a depth-averaged model the momentum equation, leaving out most of the terms, can be written as:

\[
\frac{\partial U}{\partial t} + \ldots + g \frac{\sqrt{U^2 + V^2}}{C_{2D}^2 (d + \zeta)} + \ldots = \ldots + F_x
\]  

(A.39)
\[ \frac{\partial V}{\partial t} + \ldots + g \sqrt{U^2 + V^2} + \ldots = \ldots + F_y \]  
(A.40)

where

\[ F_x = -\frac{\partial S_{xx}}{\partial x} - \frac{\partial S_{yy}}{\partial y} = D \frac{k}{\omega} \]  
(A.41)

\[ F_y = -\frac{\partial S_{xy}}{\partial x} - \frac{\partial S_{yx}}{\partial y} = D \frac{k}{\omega} \]  
(A.42)

In 3D situations the wave-induced force is only applied in the top layer

- **Stokes drift and mass flux**

  Stokes drift is defined as a wave induced drift velocity due to a net horizontal displacement of fluid particles. The Stokes drift for 3D modelling is computed from the linear wave theory:

  \[ \mathbf{u}_{x,y} = -\frac{\omega k a^2}{2 \sinh^2 (kH)} \left( \cos \phi, \sin \phi \right) \]  
(A.43)

  where

  \[ \phi = \text{angle between the current and the waves} \] [degr]

  \[ \phi = \tan^{-1} \left( \frac{M_x^s, M_y^s}{} \right) \]  
(A.44)

  where

  \[ M_x^s, M_y^s = \text{wave-induced mass fluxes} \] [m³/sm]

  \[ M_x^s = \int_{-d}^{0} \rho_0 u^x \, dz = \frac{E}{\omega} k_x \]  
(A.45)

  \[ M_y^s = \int_{-d}^{0} \rho_0 u^y \, dz = \frac{E}{\omega} k_y \]  
(A.46)

  with

  \[ E = \text{wave energy} \] [J/m²]

  \[ E = \frac{1}{8} \rho_0 g H_{rms}^2 \]  
(A.47)

- **Streaming**

  Streaming is defined as a wave-induced current in the wave boundary layer, directed in the wave propagation direction. This is modelled as a time-averaged shear stress, which results from the fact that the horizontal and vertical velocities are not exactly 90° out of phase. It is based on the wave bottom dissipation \(D_f\) and decreases linearly to zero across the wave boundary layer.

- **Wave induced turbulence**

  The vertical mixing processes are enhanced by the wave actions. This is accounted for by adding the wave energy production and dissipation terms in the turbulence model. The contribution due to wave breaking is linearly distributed over a half wave height below the mean water surface. The contribution due to bottom friction is linearly distributed over the thickness of the wave boundary layer, see Figure A 52.
• **Enhancement of the bed shear-stress by waves**

For 3D simulations, the bed shear-stress corrected for Stokes drift is given by

$$\tau_b = \left| \frac{\tau_n}{\bar{U}_{2D}} \right| \left( \bar{U} - \bar{U}^S \right)$$

(A.48)

with:

$$\bar{U}_{2D} = \frac{1}{d + \zeta} \int_{-d}^{\zeta} \bar{u} \, dx$$

(A.49)
Appendix I. Sediment Online transport formulas

Suspended sediment transport
In Delft3D, three-dimensional transport of suspended sediment is calculated by solving the three-dimensional advection-diffusion (mass-balance) equation for the suspended sediment.

\[
\frac{\partial c}{\partial t} + \frac{\partial uc}{\partial x} + \frac{\partial vc}{\partial y} + \frac{\partial (w-w_s)}{\partial z} = -\frac{\partial}{\partial x} \left( \varepsilon_{s,x} \frac{\partial c}{\partial x} \right) - \frac{\partial}{\partial y} \left( \varepsilon_{s,y} \frac{\partial c}{\partial y} \right) - \frac{\partial}{\partial z} \left( \varepsilon_{s,z} \frac{\partial c}{\partial z} \right) = 0 \quad (A.50)
\]

where
- \( c \) = mass concentration \([\text{kg/m}^3]\)
- \( u, v, w \) = flow velocity components \([\text{m/s}]\)
- \( d_y \) = interval between water level point in or x or y direction \([\text{m}]\)
- \( \varepsilon_{s,x}, \varepsilon_{s,y}, \varepsilon_{s,z} \) = eddy diffusities \([\text{m}^2/\text{s}]\)
- \( w_s \) = (hindered) sediment settling velocity of sediment fraction \([\text{m/s}]\)

The settling velocity of the sediment is dependent on the representative diameter of the sediment as described by Van Rijn (1993). In this case, \( 100 \mu \text{m} < d_s < 1000 \mu \text{m} \), this velocity is as follows:

\[
w_{s,p} = \frac{10\nu}{d_s} \left( \frac{\left( 1 + 0.01(s-1)gd_s^3 \right)^{1/2}}{\nu^2} - 1 \right) \quad (A.51)
\]

where
- \( \nu \) = kinematic viscosity coefficient of water \([\text{m}^2/\text{s}]\)
- \( s \) = relative density of the sediment \([-\text{]}\)
- \( d_s \) = representative diameter of the sediment \([\text{m}]\)

With the k-\( \varepsilon \) turbulence closure module used in this study, the vertical sediment mixing coefficient \( (\varepsilon_s) \) can be calculated directly from the vertical fluid mixing coefficient \( (\varepsilon_f) \). The latter is calculated by the turbulence closure model.

\[
\varepsilon_s = \beta_{\text{eff}} \varepsilon_f \quad (A.52)
\]

where
- \( \varepsilon_s \) = vertical sediment mixing coefficient \([\text{m}^2/\text{s}]\)
- \( \beta_{\text{eff}} \) = effective Van Rijn’s ‘Beta’ factor: \([-\text{]}\)
- \( \beta \) = Van Rijn’s ‘Beta’ factor \([-\text{]}\)

\[
\beta_{\text{eff}} = 1 + (\beta - 1) \frac{\tau_c}{\tau_w + \tau_c} \quad (A.53)
\]

\[
\beta = 1 + 2 \left( \frac{w_s}{u_{*,c}} \right)^2 \quad (A.54)
\]

\( \tau_c \) = bed shear stress due to currents \([\text{N/m}^2]\)
\( \tau_w \) = bed shear stress due to waves \([\text{N/m}^2]\)
\( \varepsilon_f \) = vertical fluid mixing coefficient \([\text{m}^2/\text{s}]\)

The local flow velocities and eddy viscosities are based on the results of the hydraulic computation. To solve the advection-diffusion equation, initial and boundary conditions have to be prescribed for each sediment fraction.
Initial conditions:
One has to define the initial concentration. Either this can be one global value or a space varying value read from a restart file or from a user defined input file.

Boundary conditions
At the free surface the vertical diffusion is zero. At the water surface boundary, the vertical flux is set to zero. The exchange of sediment in suspension and the bed is modelled by calculating the sediment fluxes from the lowest computational layer to the bed and vice versa. The boundary condition at the bed is given by:

\[-w_z c - \varepsilon_z \frac{\partial c}{\partial z} = D - E\]  \hspace{1cm} (A.55)

where
\[D = \text{sediment deposition rate} \quad [\text{kg/m}^2/\text{s}]\]
\[E = \text{sediment erosion rate} \quad [\text{kg/m}^2/\text{s}]\]

At the open boundaries, the boundary concentration has to be defined. For coarser sediment an additional option allows to specify that, at all open boundaries, the flow enters carrying the sediment at the equilibrium concentration profile.

Bed load transport
For a horizontal bed, the magnitude of bed load transport is described by Van Rijn (1993):

\[\left|S_b^*\right| = BED0.5\eta \rho_s d_{50} u_* D_{*}^{0.3} T\]  \hspace{1cm} (A.56)

where
\[\left|S_b^*\right| = \text{mass bed load transport rate} \quad [\text{kg/m/s}]\]
\[BED = \text{multiplication factor for bed load transport, range 0.8-1.0, see Appendix P} \quad [-]\]
\[\eta = \text{relative availability of the sediment fraction in the mixing layer} \quad [-]\]
\[\rho_s = \text{sediment density} \quad [\text{kg/m}^3]\]
\[d_{50} = \text{sediment diameter} \quad [\text{m}]\]
\[u_* = \text{effective bed shear velocity}: u_* = u_*\sqrt{\mu_c}\]  \hspace{1cm} [\text{m/s}]
\[u_* = \text{bed shear velocity} \quad [\text{m/s}]\]
\[\mu_c = \text{current efficiency factor} \quad [-]\]
\[D_{*} = \text{non-dimensional particle diameter} \quad [-]\]
\[T = \text{non-dimensional bed-shear stress} \quad [-]\]

The bed-load transport vector components are calculated under the assumption that the bed-load transport occurs in the same direction as the velocity vector in the bottom layer. If a bed slope exists, the magnitude of the vector is adjusted, depending on the direction of the slope and the vector.
Bed level changes

For suspended sediment, the transfer between the bed and the flow is modelled using sink and source terms acting on the layer that is entirely above Van Rijn’s reference height $a$, see Figure A 53.

The erosive flux due to upward diffusion through the underside of the $kmx$ layer is approximated by the expression:

$$ E = \varepsilon_s \frac{\partial c}{\partial z} \approx \varepsilon_s \left( \frac{c_a - c_{kmx}}{\Delta z} \right) \approx \varepsilon_s \frac{c_a}{\Delta z} - \varepsilon_s \frac{c_{kmx}}{\Delta z} $$

(A.57)

where

$\varepsilon_s$ = sediment diffusion coefficient $[m^2/s]$

$c_a$ = reference concentration $[kg/m^3]$

$c_{kmx}$ = average concentration of $kmx$ cell $[kg/m^3]$

$\Delta z$ = difference in elevation between the centre of the $kmx$ cell and Van Rijn’s reference height, $\Delta z = z_{kmx} - a$ $[m]$

The deposition flux of sediment through the underside of the $kmx$ cell is approximated by:

$$ D = w_s c_{kmx,bot} \approx w_s c_{kmx} $$

(A.58)

Combining the latter two equations result in a total source and sink term:

$$ Source = c_a \left( \frac{\varepsilon_s}{\Delta z} \right) $$

(A.59)

$$ Sink = c_{kmx} \left( \frac{\varepsilon_s}{\Delta z} + w_s \right) $$

(A.60)
The reference height is calculated in accordance with Van Rijn (1993):

\[ c_a = SUS \cdot 0.015 \rho_s \frac{d_{50} (T_a)^{1.5}}{a (D_*)^{0.3}} \]  

(A.61)

where

- \( SUS \) = multiplication factor for calibration [-]
- \( \rho_s \) = density of the sediment \([kg/m^3]\]
- \( d_{50} \) = characteristic grain size [m]
- \( T_a \) = non-dimensional bed-shear stress [-]
- \( D_* \) = non-dimensional particle diameter [-]

The change in the quantity of sediment at the bed caused by the bed load transport is calculated from the gradients in the bed load transport in the \( x \)- and \( y \)-direction.

\[
\Delta_{sed}^{(m,n)} = \Delta t \cdot \frac{f_{MORFAC}}{A^{(m,n)}} \left( S^{(n,m-1)}_{b,uu} \Delta y^{(n,m-1)} - S^{(n,m)}_{b,uu} \Delta y^{(n,m)} + S^{(n-1,m)}_{b,vv} \Delta x^{(n-1,m)} - S^{(n,m)}_{b,vv} \Delta x^{(n,m)} \right) 
\]

(A.62)

where

- \( \Delta_{sed}^{(m,n)} \) = change in quantity of bottom sediment at location \((m,n)\) \([kg/m^2]\]
- \( \Delta t \) = computational time step \([s]\]
- \( f_{MORFAC} \) = morphological time factor [-]
- \( A^{(m,n)} \) = area of cell \(m,n\) \([m^2]\]
- \( S \) = computed bed load transport vector \([kg/ms]\]
- \( \Delta x^{(m,n)} \) = cell width in the \(x\)-direction, held at the V-point of cell \((m,n)\) [m]
- \( \Delta y^{(m,n)} \) = cell width in the \(y\)-direction, held at the U-point of cell \((m,n)\) [m]
Appendix J. Measured wave heights

The next table shows the measured wave heights, averaged per individual test, in addition to §4.2, and the averaged wave height per wave condition at four locations (see Figure A 55) and the transmission coefficient between location 2 and 3 is given. In total, there are 14 successive tests with three different wave conditions.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Wave Condition</th>
<th>Measured wave height (m)</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Loc. 1</td>
<td>Loc. 2</td>
<td>loc. 3</td>
<td>loc. 4</td>
</tr>
</tbody>
</table>
| T102      | 1              | 1.59   | 1.44   | 0.66   | 0.65   | 0.46  
| T103      | 3              | 3.31   | 2.92   | 1.08   | 1.10   | 0.37  
| T105      | 1              | 1.60   | 1.42   | 0.65   | 0.64   | 0.46  
| T106      | 2              | 2.54   | 2.31   | 0.86   | 0.87   | 0.37  
| T108      | 1              | 1.60   | 1.44   | 0.65   | 0.63   | 0.46  
| T109      | 3              | 3.36   | 2.92   | 1.08   | 1.08   | 0.37  
| T110      | 1              | 1.60   | 1.44   | 0.65   | 0.63   | 0.45  
| T111      | 1              | 1.61   | 1.42   | 0.65   | 0.63   | 0.46  
| T112      | 3              | 3.33   | 2.88   | 1.08   | 1.09   | 0.38  
| T113      | 1              | 1.60   | 1.42   | 0.65   | 0.63   | 0.46  
| T114      | 2              | 2.54   | 2.30   | 0.87   | 0.87   | 0.38  
| T115      | 2              | 2.53   | 2.30   | 0.87   | 0.87   | 0.38  
| T116      | 2              | 2.53   | 2.27   | 0.87   | 0.86   | 0.38  
| T117      | 2              | 2.55   | 2.27   | 0.87   | 0.83   | 0.38  
| Mean      | 1              | 1.600  | 1.427  | 0.652  | 0.637  | 0.457 
| Mean      | 2              | 2.538  | 2.290  | 0.868  | 0.860  | 0.379 
| Mean      | 3              | 3.333  | 2.907  | 1.080  | 1.090  | 0.372 

Table A 2 Measured wave height per test and averaged for mean wave condition at four locations

Figure A 55 Location of the water level gauges
Appendix K.  Comparison of transmission models

The following models are compared with regard to the applicability to simulate the crest width.
1. Van der Meer (1991), see Equation (2.4) in §2.2.1;
2. Goda (1967), see Equation (2.1) in §2.2.1;
3. D’Angremond et al. (1996) for permeable breakwaters, see Equation (2.6) in §2.2.1;
4. D’Angremond et al. (1996) for impermeable breakwaters, see Equation (2.7) in §2.2.1;
5. Seabrook and Hall (1998), see Equation (2.8) in §2.2.1.

For all three wave conditions, the transmission versus the crest width is plotted. With this, a sloping seaward side of the breakwater is assumed as in Figure A 56. because most of the transmission formulations take the front slope into account. This is important because especially the high wave conditions are mainly affected by this front slope.

Figure A 56  Schematised breakwater profile, sloping seaward side without a berm.

The first wave condition, \( H_s = 1.5 \) m, shows the following picture. Here the formulation of d’Angremond et al. (1996) and Seabrook and Hall (1998) for permeable breakwaters agrees best with measurements.

Figure A 57  Transmission versus crest width, measured versus approaching models (\( H_s = 1.5 \) m)
For the second wave condition, $H_s = 2.5$ m, Seabrook and Hall’s (1998) formulation appears to be a good one, see Figure A 58.

![Figure A 58](image1)

For the third wave condition, $H_s = 3.5$ m, Seabrook and Hall’s (1998) formulation again appears to be best, see Figure A 59.

![Figure A 59](image2)

The overall conclusion that can be drawn for this case is that the formulations of Goda and Van de Meer approach the measurements worse than the formulations of d’Angremond et al. (1996) and Seabrook and Hall (1998). This is mainly due to the influence of the crest width that is either or not accounted for.
Appendix L. Measured water levels

The next table shows measured and adjusted averaged water levels per test and per wave condition. The water levels are obtained from the main wave height signal and transformed to prototype scale. The four locations are therefore the same as for the wave heights, see Appendix J. The upper values represent the mean signal and the lower values are adjusted values, relative to location 1, see Figure 4-2 in §4.2.

<table>
<thead>
<tr>
<th>Test Name</th>
<th>Wave Condition</th>
<th>Measured water level (m)</th>
<th>Loc. 1</th>
<th>Loc. 2</th>
<th>Loc. 3</th>
<th>Loc. 4</th>
<th>Set-up 2,3</th>
</tr>
</thead>
<tbody>
<tr>
<td>T102</td>
<td>1</td>
<td>0.40</td>
<td>-0.04156</td>
<td>0</td>
<td>-0.04737</td>
<td>+0.03709</td>
<td>+0.04232</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.3582</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T103</td>
<td>3</td>
<td>0.60</td>
<td>-0.13947</td>
<td>0</td>
<td>-0.13683</td>
<td>0.24706</td>
<td>0.26309</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.46</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T105</td>
<td>1</td>
<td>0.40</td>
<td>-0.0272</td>
<td>0</td>
<td>-0.0318</td>
<td>0.0329</td>
<td>0.0369</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.373</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T106</td>
<td>2</td>
<td>0.50</td>
<td>-0.0751</td>
<td>0</td>
<td>-0.0821</td>
<td>0.1476</td>
<td>0.15330</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.425</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T108</td>
<td>1</td>
<td>0.40</td>
<td>-0.0265</td>
<td>0</td>
<td>-0.0292</td>
<td>0.0394</td>
<td>0.0391</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.374</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T109</td>
<td>3</td>
<td>0.60</td>
<td>-0.1447</td>
<td>0</td>
<td>-0.1417</td>
<td>0.2502</td>
<td>0.2675</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.455</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T110</td>
<td>1</td>
<td>0.40</td>
<td>-0.0290</td>
<td>0</td>
<td>-0.0363</td>
<td>0.0452</td>
<td>0.0454</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.371</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T111</td>
<td>1</td>
<td>0.40</td>
<td>-0.0250</td>
<td>0</td>
<td>-0.0464</td>
<td>0.0389</td>
<td>0.0417</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.375</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T112</td>
<td>3</td>
<td>0.60</td>
<td>-0.1504</td>
<td>0</td>
<td>-0.1544</td>
<td>0.2474</td>
<td>0.2716</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.45</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T113</td>
<td>1</td>
<td>0.4</td>
<td>-0.0300</td>
<td>0</td>
<td>-0.0401</td>
<td>0.0419</td>
<td>0.0442</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.37</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T114</td>
<td>2</td>
<td>0.5</td>
<td>-0.0818</td>
<td>0</td>
<td>-0.0892</td>
<td>0.1590</td>
<td>0.1676</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.42</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T115</td>
<td>2</td>
<td>0.50</td>
<td>-0.0821</td>
<td>0</td>
<td>-0.0910</td>
<td>0.1562</td>
<td>0.1655</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.42</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T116</td>
<td>2</td>
<td>0.50</td>
<td>-0.0851</td>
<td>0</td>
<td>-0.0969</td>
<td>0.1548</td>
<td>0.1680</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.415</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T117</td>
<td>2</td>
<td>0.50</td>
<td>-0.0846</td>
<td>0</td>
<td>-0.0985</td>
<td>0.1574</td>
<td>0.1672</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.415</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>1</td>
<td>0.40</td>
<td>-0.03</td>
<td>0</td>
<td>-0.04</td>
<td>0.04</td>
<td>0.04</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.37</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>2</td>
<td>0.50</td>
<td>-0.08</td>
<td>0</td>
<td>-0.09</td>
<td>0.15</td>
<td>0.16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.42</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>3</td>
<td>0.60</td>
<td>-0.14</td>
<td>0</td>
<td>-0.14</td>
<td>0.25</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.46</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table A 3  Measured water levels and adjusted water levels for the mathematical model
Appendix M. measured velocities

The velocities are measured 1 meter above the bottom and halfway the depth, just behind the breakwater, see Figure A 60. This is only done for a few tests, see Table A 4 and Table A 5. Table A 4 shows the velocities of the averaged 1/3 highest values. Table A 5 shows the mean velocity gauge signal, per test and averaged per wave condition.

<table>
<thead>
<tr>
<th>Test name</th>
<th>Hs at CD-10m (m)</th>
<th>Tp (m)</th>
<th>Duration (hr)</th>
<th>Set-up (m wrt msl)</th>
<th>Velocity (av. of highest 1/3, m/s)</th>
<th>half</th>
<th>trough</th>
<th>1 m above bed</th>
</tr>
</thead>
<tbody>
<tr>
<td>T102</td>
<td>1.5</td>
<td>5.5</td>
<td>0-12</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>T103</td>
<td>3.5</td>
<td>8.5</td>
<td>12-18</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>T106</td>
<td>2.5</td>
<td>7.0</td>
<td>30-36</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>T113</td>
<td>1.5</td>
<td>5.5</td>
<td>72-85.5</td>
<td>0.4</td>
<td>0.43</td>
<td>0.47</td>
<td>0.53</td>
<td>0.42</td>
</tr>
<tr>
<td>T112</td>
<td>3.5</td>
<td>8.5</td>
<td>66-72</td>
<td>0.6</td>
<td>0.84</td>
<td>0.90</td>
<td>0.96</td>
<td>0.80</td>
</tr>
<tr>
<td>T114</td>
<td>2.5</td>
<td>7.0</td>
<td>85.5-91.5</td>
<td>0.5</td>
<td>0.66</td>
<td>0.65</td>
<td>0.70</td>
<td>0.61</td>
</tr>
</tbody>
</table>

Table A 4  Measured velocities, averaged of highest 1/3

<table>
<thead>
<tr>
<th>Test name</th>
<th>Hs at CD-10m (m)</th>
<th>Tp (m)</th>
<th>Duration (hr)</th>
<th>Set-up (m wrt msl)</th>
<th>Velocity (mean signal, m/s)</th>
<th>VCM1</th>
<th>VCM2</th>
<th>VCM3</th>
<th>VCM4</th>
<th>half</th>
<th>trough</th>
<th>1 m above bed</th>
</tr>
</thead>
<tbody>
<tr>
<td>T102</td>
<td>1.5</td>
<td>5.5</td>
<td>0-12</td>
<td>0.4</td>
<td>-0.12</td>
<td>-0.02</td>
<td>-0.08</td>
<td>0.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T103</td>
<td>3.5</td>
<td>8.5</td>
<td>12-18</td>
<td>0.6</td>
<td>-0.17</td>
<td>-0.08</td>
<td>-0.15</td>
<td>-0.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T106</td>
<td>2.5</td>
<td>7.0</td>
<td>30-36</td>
<td>0.5</td>
<td>-0.26</td>
<td>-0.07</td>
<td>-0.17</td>
<td>-0.02</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T113</td>
<td>1.5</td>
<td>5.5</td>
<td>72-85.5</td>
<td>0.4</td>
<td>-0.19</td>
<td>-0.06</td>
<td>-0.15</td>
<td>0.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T114</td>
<td>2.5</td>
<td>7.0</td>
<td>85.5-91.5</td>
<td>0.5</td>
<td>-0.19</td>
<td>-0.05</td>
<td>-0.15</td>
<td>0.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T115</td>
<td>2.5</td>
<td>7.0</td>
<td>91.5-97.5</td>
<td>0.5</td>
<td>-0.19</td>
<td>-0.05</td>
<td>-0.15</td>
<td>0.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T116</td>
<td>2.5</td>
<td>7.0</td>
<td>97.5-103.5</td>
<td>0.5</td>
<td>-0.19</td>
<td>-0.05</td>
<td>-0.15</td>
<td>0.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T117</td>
<td>2.5</td>
<td>7.0</td>
<td>103.5-109.5</td>
<td>0.5</td>
<td>-0.19</td>
<td>-0.06</td>
<td>-0.14</td>
<td>-0.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>1.5</td>
<td>5.5</td>
<td></td>
<td>0.4</td>
<td>-0.12</td>
<td>-0.02</td>
<td>-0.08</td>
<td>0.03</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>2.5</td>
<td>7.0</td>
<td></td>
<td>0.5</td>
<td>-0.19</td>
<td>-0.06</td>
<td>-0.15</td>
<td>0.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>3.5</td>
<td>8.5</td>
<td></td>
<td>0.6</td>
<td>-0.26</td>
<td>-0.07</td>
<td>-0.17</td>
<td>-0.02</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table A 5  Velocity measurements per individual test, mean velocity gauge signal

![Figure A 60 Location of the velocity gauge]
Appendix N. Bathymetry

First, the FLOW grid is defined, with smaller grid sizes at the breakwater end the shoreline and larger ones in offshore direction, see Figure A 61. This is done in such a way that the 15 meter wide crest is covered by three grid cells, see Figure A 62.

Secondly, the horizontal site values have to be translated to prototype values by multiplying by the length scale factor 15 and adding 1193.3 to place the breakwater depth points on the finer grid. The measured bottom points are averaged for every bottom grid point, i.e. averaging all the measured values half a grid size before until half a grid size behind the bottom point, see Figure A 62.

Figure A 61 Grid and bathymetry

Secondly, the horizontal site values have to be translated to prototype values by multiplying by the length scale factor 15 and adding 1193.3 to place the breakwater depth points on the finer grid. The measured bottom points are averaged for every bottom grid point, i.e. averaging all the measured values half a grid size before until half a grid size behind the bottom point, see Figure A 62.
The depth values are subtracted from 0.599, multiplied by 15 and added to 0.52 meter, (MSL=CD+0.52m). The measured depth values in 9501 points are translated to the 98 grid points by averaging the surrounding depth points. At the submerged breakwater, depth values are defined manually, since steep corner points would be flattened by the averaging process.

At the first bottom measurements, the gauges were not calibrated and for that bathymetry a correction term of 0.008 has to be implemented, so the initial bottom formula becomes:

\[
d\text{prototype} = (0.599 - (d\text{scalemodel} - 0.008)) \times 15 + 0.52
\]

(A.63)

Figure A 62 Initial bathymetry around the breakwater for both the physical and the mathematical model.
Appendix O. Initial FLOW input parameters

This appendix explains the initial FLOW input file parameter settings, as referred to in §4.3.1.

Delft3D-FLOW mdf file:

<table>
<thead>
<tr>
<th>Domain</th>
<th>Grid parameters</th>
<th>Curvilinear grid</th>
</tr>
</thead>
<tbody>
<tr>
<td>K=10 layers, for distribution, see Table A 6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Latitude</td>
<td>0 degrees N</td>
<td></td>
</tr>
<tr>
<td>Orientation</td>
<td>0 degrees</td>
<td></td>
</tr>
<tr>
<td>Bathymetry</td>
<td>imported</td>
<td></td>
</tr>
<tr>
<td>Dry Points</td>
<td>Not applied</td>
<td></td>
</tr>
<tr>
<td>Thin dams</td>
<td>Not applied</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table A 6</th>
<th>Layer thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>layer</td>
<td>Thickness (%)</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
</tr>
<tr>
<td>4</td>
<td>15</td>
</tr>
<tr>
<td>5</td>
<td>15</td>
</tr>
<tr>
<td>6</td>
<td>15</td>
</tr>
<tr>
<td>7</td>
<td>15</td>
</tr>
<tr>
<td>8</td>
<td>10</td>
</tr>
<tr>
<td>9</td>
<td>5</td>
</tr>
<tr>
<td>10</td>
<td>5</td>
</tr>
</tbody>
</table>

Time Frame

- Duration of the simulation differs per test
- Time step: 0.1 min / 12 sec
- Number of time steps: 7200

Processes

- De-activated: Salinity, Temperature, Wind
- Activated: Constituents, Wave

Initial conditions

- Uniform water level: reference level +0.6 m

Boundaries

- Reflection coefficient Alfa:
  - Specifies the amount by which the boundary is reflective for short wave disturbances that propagates towards the boundary from inside the model (limit interval 0:10,000). For water level boundaries for
  - alpha is advised \( \alpha = \frac{T_d}{g} \sqrt{\frac{H}{g}} \)
  - \( T_d = \) time for a wave to travel from the left to the right
  - boundary; \( d_{w} = -5, c = (gh)^{0.5} = 7\) m/s \( L = 765, T_d = 115\) s
  - Input, Alfa=100

- Type of open boundary: Water level
- Forcing type of the boundary condition: Time series

Physical Parameters

- Constants
  - Gravity: 9.810 m/s²
  - Water density: 1000 kg/m³
  - Temperature: 15.00 degrees C
  - Salinity: 31.00 p.p.t.

- Roughness
  - According to Manning imported from file, same in U and V direction, different for sand and rock, according to Table A 7 and Figure A 63.

<table>
<thead>
<tr>
<th>Table A 7</th>
<th>Manning coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manning</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>0.028</td>
</tr>
<tr>
<td>Rock</td>
<td>0.035</td>
</tr>
</tbody>
</table>
The Chézy friction coefficient is calculated from: \( C = \frac{h^{1/6}}{n} \), where \( h \) is the water depth and \( n \) is the Manning coefficient.

Bottom stress formulation due to waves: Fredsøe (only for \( k=1 \))

### Viscosity
- Horizontal eddy viscosity: 10 [m²/s]
- Horizontal eddy diffusivity: 10 [m²/s]
- Vertical eddy viscosity: 1.0E-6 [m²/s]
- Vertical eddy diffusivity: 1.0E-6 [m²/s]
- Ozmidov length scale: 0.000 [m]
- Turbulence model: k-epsilon

### Numerical parameters
- Extra drying/flooding: max
- Threshold depth: 0.010m
- Marginal depth: -999.000m
- Smoothing time: 60 min

### Discharges
- Not applied

### Monitoring
- Observation points: every point on the breakwater crest in alongshore direction
- Drogue: not applied
- Cross-section: In alongshore direction over the breakwater

### Additional parameters
- Filsed “sedinp.gib”
- Filmor “morph.inp”

### Output
- Storage: Map and Com file, interval 5 minutes
- Print: History file, interval 5 minutes
- Details: all possible output
Appendix P. Morphology input file parameters

This appendix describes the morphological input file parameters as referred to in §4.3.1. First, the ‘Morph.inp’-file is given and below, every parameter is explained and the choice of the assigned initial value will be account for.

Morph.inp file

6.0    : MORFAC (morphological scale factor), one real
15.0      : MORSTT (minutes after ITDATE to begin bed updating) e.g. 2 min /0.1min/dt=20dt
.2          : THRESH (threshold sediment thickness for reducing sediment exchange) [m].
true.     : MORUPD (update bathymetry during FLOW run)
.true.     : EQMBC (equilibrium concentration at inflow boundaries)
.true.     : DENSIN (include effect of sediment on density gradient)
1.0        : AKSFAC (van Rijn's reference height = AKSFAC * KS)
2.0        : RWAVE (wave related roughness = RWAVE * estimated ripple height. Van Rijn Recommends range 1-3)
.false.    : ROUSE (set equilibrium sediment concentration values to standard Rouse profiles)
1.0        : ALFABS (longitudinal bed gradient factor for bed load transport)
1.5        : ALFABN (transverse bed gradient factor for bed load transport)
1.0        : SUS (Calculate suspended sediment transport/ multiplication factor for suspended sediment transport)
1.0        : BED (Calculate bed load transport/ multiplication factor for bed load transport)
0.0        : SUSW (wave-related suspended sed. transport factor)
0.8        : BEDW (wave-related bed-load sed. transport factor)
0.5        : SEDTHR (minimum threshold depth for sediment computations (m))
0.8        : THETSD (Fraction of erosion to assign to adjacent dry cells)
0.0        : HMAXTH (Max depth for variable THETSD. Set < SEDTHR to use global value only)
0.1 : FWFAC

Parameters

MORFAC morphological scale factor, the change in quantity of bottom sediment is multiplied by this factor

\[ \Delta_{sed}^{(m,n)} = \frac{\Delta t \cdot f_{MORFAC}}{A^{(m,n)}} \left( \Delta y^{(n,m-1)} - S_{b,uu} \Delta y^{(n,m)} + S_{b,vv} \Delta x^{(n-1,m)} - S_{b,vv} \Delta x^{(n,m)} \right) \]

MORSTT During the stabilising period from the initial conditions to the boundary conditions, the patterns of erosion and accretion do not reflect the true morphological development and should therefore be ignored. This factor makes it possible to delay the morphological bottom updating. It has to be defined in simulation-minutes from the first time step, i.e. not including the morphological scale factor.

THRESH This user-specified erosion threshold reduces the calculated bed-load transport vector as follows.

\[ S_b^* = f_{FIXFAC} \cdot S_b^* \]
\[ f_{FIXFAC} = \frac{DPSED}{TRESH} \]

where

DPSED = depth of the sediment at the bed [m]

Below this threshold depth, the bed load transports are gradually reduced to zero when the sand layer thickness approaches zero.
MORUPD  The user can specify whether or not to update the bathymetry. It can be useful if only the initial transport patterns are required. In this study, this ‘switch’ is set on ‘false’ during the calibration of the hydraulic parameters.

EQMBC  When activating this parameter will use equilibrium concentration at inflow boundaries. All the sand load entering though the boundaries will hereby near-perfectly adapted to the local flow conditions. In this study, this option is used.

DENSIN  This parameter includes the effect of sediment on density gradient. With large concentration it is expected that the effect of sediment on the density will be enough significant to use this option.

AKSFAC  Van Rijn's (1993) reference height $a$ (see Figure A 53 in Appendix I) is given by:

$$ a = \min \left[ \max \left\{ \text{ASKFAC} \cdot k_s, \frac{\Delta_r}{2}, 0.01h \right\}, 0.20h \right] $$

RWAVE  Wave roughness adjustment factor; wave related roughness $\bar{k}_{r,w} = R W A V E \cdot \Delta_r$, with the limits $0.01 \leq \bar{k}_{r,w} \leq 0.1m$.

Van Rijn Recommends range 1-3, in this study the default value 2 is used.

ROUSE  Set equilibrium sediment concentration values to standard Rouse profiles. In this study, this option is set on ‘false’, by whom the equilibrium concentration is calculated using the actual mixing values calculated by FLOW, and an upwind method of numerical integration to solve the stationary advection-diffusion equation.

ALFABS  Longitudinal bed gradient factor for bed load transport, default value 1 is used.

$$ \tilde{S}_b = \alpha_z \tilde{S}^e $$

$$ \alpha_z = \alpha_{bs} \frac{\tan(\varphi)}{\cos\left[\tan^{-1}\left(\frac{\tilde{c}_z}{\tilde{c}_s}\right)\right]} - 1 $$

ALFABN  Transverse bed gradient factor for bed load transport, default value 1 is used.

$$ S_{b,n} = \left[ S_b^e \right] \alpha_{bn} \frac{u_{b,cr}}{\tilde{u}_b} \frac{\tilde{c}_b}{\tilde{c}_n} $$

where

$$ \frac{\tilde{c}_b}{\tilde{c}_n} = - \frac{\tilde{c}_b^{(w)}}{\tilde{c}_x} \frac{S_{b,y}^e}{\tilde{S}_b^e} + \frac{\tilde{c}_b^{(v)}}{\tilde{c}_y} \frac{S_{b,x}^e}{\tilde{S}_y^e} $$

is the bed slope in the direction normal to the unadjusted bed-load transport vector.

SUS  Multiplication factor for suspended sediment transport. The reference concentration is multiplied by SUS as follows:

$$ c_a = \text{SUS} \cdot 0.015 \rho_s \frac{d_{50}(T_9)}{\alpha(D_s)^{0.3}} $$

The default value 1 is used.
<table>
<thead>
<tr>
<th>Variable</th>
<th>Description</th>
</tr>
</thead>
</table>
| **BED**  | Multiplication factor for bed load transport, see Appendix I: sediment transport formulas.  
\[
S_{B}^{*} = BED0.5\eta_{r}d_{s0}u_{c}D_{s}^{-0.3}T
\]

The default value 1 is used.

<table>
<thead>
<tr>
<th>SUSW</th>
<th>Wave-related suspended sediment transport factor, 0.0 is used.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>BEDW</strong></td>
<td>Wave-related bed-load sediment transport factor</td>
</tr>
<tr>
<td>SEDTHR</td>
<td>Minimum threshold depth for sediment computations (m), in this study 0.1 is used.</td>
</tr>
<tr>
<td>THETSD</td>
<td>Fraction of erosion to assign to adjacent dry cells. This factor lies between 0 and 1 and represents the portion of erosion of the last wet computational cell that is assigned to the adjacent dry cell instead of the wet cell itself. In this study, the value 0.8 is used initially, see §4.4.5.</td>
</tr>
<tr>
<td>HMAXTH</td>
<td>Max depth for variable THETSD. In this study the value 0.0 is used</td>
</tr>
<tr>
<td>FWFAC</td>
<td>Tuning parameter for wave streaming. The default value 0.1 is used.</td>
</tr>
</tbody>
</table>
Appendix Q. Sediment input file parameters

This appendix describes the morphological input file parameters as referred to in §4.3.1. First the 'sedinp.102'-file is given and below, every parameter is explained and the choice of the assigned initial value will be account for.

'SEDINP.102' input file

<table>
<thead>
<tr>
<th>No</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.0e+5</td>
<td>LSED: number of sediment fractions, CSOIL</td>
</tr>
<tr>
<td></td>
<td>sand</td>
<td>SEDTYP(L): type of sediment</td>
</tr>
<tr>
<td>2</td>
<td>2650.0</td>
<td>RHOSEL: density sediment [kg/m^3]</td>
</tr>
<tr>
<td>3</td>
<td>0.000220</td>
<td>SEDDIA</td>
</tr>
<tr>
<td>4</td>
<td>0.0000</td>
<td>SALMAX: salinity for saline settling velocity</td>
</tr>
<tr>
<td>5</td>
<td>0.0000</td>
<td>WS0: settling velocity fresh water (for mud)</td>
</tr>
<tr>
<td>6</td>
<td>0.0000</td>
<td>WSM: settling velocity saline water (for mud)</td>
</tr>
<tr>
<td>7</td>
<td>0.0000</td>
<td>TCRSED: critical stress for sedimentation</td>
</tr>
<tr>
<td>8</td>
<td>0.0000</td>
<td>TCRERO: critical stress for erosion</td>
</tr>
<tr>
<td>9</td>
<td>0.0000</td>
<td>EROPAR: erosion parameter (kg/m^2/s)</td>
</tr>
<tr>
<td>10</td>
<td>1600.0</td>
<td>CDRYB: dry bed density</td>
</tr>
<tr>
<td></td>
<td>102sed.dep</td>
<td></td>
</tr>
</tbody>
</table>

Parameters

LSED  Number of sediment fractions. In this study, one sediment fraction is used.

CSOIL Reference density for hindered settling calculations (not active when ≥ 10^6 [kg/m^3])

\[ w_s = \left( 1 - \frac{e_s^{tot}}{CSOIL} \right)^5 w_s,0 \]

In this study the value 10.0e+5 is used.

RHOSEL Sediment density: 2650 kg/m^3 is used

SEDDIA Sediment diameter: 0.000220 meter

SALMAX Salinity for saline settling velocity. Only applicable for cohesive sediment and therefore set on 0.0

WS0 Settling velocity fresh water (for mud). Only applicable for cohesive sediment and therefore set on 0.0

WSM Settling velocity saline water (for mud). Only applicable for cohesive sediment and therefore set on 0.0

TCRSED Critical stress for sedimentation. 0.0 is used

TCRERO Critical stress for erosion. 0.0 is used

EROPAR Erosion parameter (kg/m^2/s). 0.0 is used

CDRYB Dry bed density: 1600 kg/m^3 is used

102sed.dep Sediment input file with initial available sediment per water level point in kg/m^2. Everywhere the value 10,000 is used, except the value zero between the land boundary file at the breakwater, see Figure 3-10.
Appendix R. Initial WAVE input parameters

This appendix describes the initial wave input parameters for the simulation in addition to § 4.3.1. The model as used version is SWAN.

Flow
- Use bottom activated
- Use water level activated
- Use current de-activated

Grids
- Computational geographical space: regular rectangular
- Spectral resolution for selected grid:
  - Directional space: sector, 255-285°
  - Frequency space: lowest frequency: 0.05 [Hz]
  - Highest frequency: 1 [Hz]
  - Number of frequency bins: 24
- Bottom geographical space: regular rectangular
- Bathymetry: adjusted at the submerged breakwater

Time frame
- Not adjustable

Tidal information
- Water level according Appendix L

Boundaries
- Three equal boundaries at the North, West and South side
- Specification of boundary: parametric
- Spectral space:
  - Shape: Jonswap
  - Peak enhancement factor $\gamma$: 3.3
  - Period: peak, according Table 4-2
  - Width energy: 1 degree
- Condition type: constant
- Conditions: according Table A 8

<table>
<thead>
<tr>
<th>Wave condition</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{\text{significant}}$</td>
<td>1.5</td>
<td>2.5</td>
<td>3.5</td>
</tr>
<tr>
<td>Peak period $T_p$</td>
<td>5.5</td>
<td>7.0</td>
<td>8.5</td>
</tr>
<tr>
<td>Direction (nautical)</td>
<td>270</td>
<td>270</td>
<td>270</td>
</tr>
<tr>
<td>Width energy distribution</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>

Table A 8  Initial wave height and wave period conditions at the boundary

Obstacles
- 9 sheets, transmission coefficient as in §4.4
Physical parameters

Constants

- Gravity: 9.81 [m/s²]
- Water density: 1025 [kg/m³]
- North: 90 [degr]
- Minimum depth: 0.05 [m]
- Convention: nautical
- Set-up: de-activated, only in stand-alone version
- Forces: radiation stress

Wind
- constant wind
  - Speed: 0 [m/s]
  - Direction (naut.): 0 [degr]

Processes

- Type of formulations: 3rd generation
- Bottom friction: Jonswap coefficient: 0.067
- Depth induced breaking
  - Alfa: 1
  - Gamma different per wave condition, see §4.4.3.
- Non-linear triad interactions, activated
  - Alfa: 0.1
  - Beta: 2.2
- Various
  - Wind growth: de-activated
  - Whitecapping: de-activated
  - Quadruplets: de-activated
  - Wave propagation in spectral space: activated
  - Frequency shift: activated

Numerical parameters

Spectral space

- Directional space: 0.5 [-]
- Frequency space: 0.5 [-]

Accuracy criteria

- Perc. of wet grid points: 95 [%]
- Maximum no. of iterations: 5 [-]

Output curves

Not applied

Output parameters
Appendix S: Input scripts

This appendix shows the Matlab and batch files, referred to in 4.3.2. The following Matlab script run003.m is used to initiate the batch file runcond.bat, followed by the Matlab script dsdd.m and inifile.m. For each wave condition, the batch file modifies the parameters, initiates the Delft3D computation and stores the output in the right subdirectory. The ‘dsdd.m’ Matlab file reads the output of one run and stores them in files. The ‘inifile.m’ file reads data from an output file as writes an initial conditions file for the next simulation.

Run003.m

clear all;close all; rest=0; vergelijking=0;
cd d:\delft3d\batch\morfac=62\003
!md run
!md output
!copy flow\6cel2x.dep run\6cel2.dep
!copy flow\gibsedx2.dep run\gibsed2.dep
!copy d:\matlab\t02\dsdd1.m batch\dsdd1.m

cd batch
%REM          aaa gammabr xxx yyy  hshs  tptp ddir  kt1  kt2  kt3  kt4  kt5  kt6  kt7   kt8  kt9      x1   x2   x3   x4   x5   x6    x7
x8   x9   rww   hhh Tscale CJON ddd dt1 dt2 duur run test
!call runcond 0 0.73   0 10 1.64 5.5 270 0.95 0.94 0.92 0.90 0.87 0.87 0.93 0.97 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0363.bct 0.363 6.0 62 MEAN 0.05 3.0 300 003 102
i=-1;run dsdd1;
!call runcond 0 0.85 0 26 3.29 8.5 270 1.00 1.00 0.98 0.78 0.64 0.87 0.93 0.97 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 044.bct 0.44 1.0 0.0125 0.75 1200 003 103
run dsdd1;
!call runcond 0 0.73 0 10 1.64 5.5 270 0.95 0.94 0.92 0.90 0.87 0.87 0.90 0.97 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0363.bct 0.363 6.0 62 MEAN 0.05 3.0 300 003 105
run dsdd1;
!call runcond 0 0.74 0 6 2.41 7.0 270 1.00 0.95 0.96 0.98 0.99 0.65 0.78 0.93 0.94 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0412.bct 0.41 2.0 0.0125 0.75 1200 003 106
run dsdd1;
!call runcond 0 0.73 0 10 1.64 5.5 270 0.95 0.94 0.92 0.90 0.87 0.87 0.90 0.97 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0363.bct 0.363 6.0 62 MEAN 0.05 3.0 300 003 108
run dsdd1;
!call runcond 0 0.85 0 26 3.29 8.5 270 1.00 1.00 0.98 0.78 0.64 0.87 0.93 0.97 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 044.bct 0.44 1.0 0.0125 0.75 1200 003 109
run dsdd1;
!call runcond 0 0.73 0 6 1.64 5.5 270 0.95 0.94 0.92 0.90 0.87 0.87 0.90 0.97 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0363.bct 0.363 6.0 62 MEAN 0.05 3.0 300 003 110
run dsdd1;
!call runcond 0 0.73 0 6 1.64 5.5 270 0.95 0.94 0.92 0.90 0.87 0.87 0.90 0.97 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0363.bct 0.363 6.0 62 MEAN 0.05 3.0 300 003 111
run dsdd1;
!call runcond 0 0.73 0 6 2.41 7.0 270 1.00 0.95 0.96 0.98 0.99 0.65 0.78 0.93 0.94 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0412.bct 0.41 2.0 0.0125 0.75 1200 003 112
run dsdd1;
!call runcond 0 0.73 0 11 1.64 5.5 270 0.95 0.94 0.92 0.90 0.87 0.87 0.90 0.97 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0363.bct 0.363 6.0 62 MEAN 0.05 3.0 300 003 113
run dsdd1;
!call runcond 0 0.74 0 6 2.41 7.0 270 1.00 0.95 0.96 0.98 0.99 0.65 0.78 0.90 0.94 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0412.bct 0.41 6.0 49 MEAN 0.05 3.0 300 003 114
run dsdd1;
!call runcond 0 0.74 0 6 2.41 7.0 270 1.00 0.95 0.96 0.98 0.99 0.65 0.78 0.90 0.94 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0412.bct 0.41 6.0 49 MEAN 0.05 3.0 300 003 115
run dsdd1;
!call runcond 0 0.74 0 6 2.41 7.0 270 1.00 0.95 0.96 0.98 0.99 0.65 0.78 0.90 0.94 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0412.bct 0.41 6.0 49 MEAN 0.05 3.0 300 003 116
run dsdd1;
!call runcond 0 0.74 0 6 2.41 7.0 270 1.00 0.95 0.96 0.98 0.99 0.65 0.78 0.90 0.94 155 1560 1565 1570 1575 1580 1585 1590 1595
1580 1590 1595 0412.bct 0.41 6.0 49 MEAN 0.05 3.0 300 003 117
run dsdd1;
Runcond.bat

cd..
REM======== copy input files=========== runcond.bat
copy d:\delft3d\ini\mod.exe run\mod.exe
copy mdf\morf.tpl run\morf.gib
copy mdf\md-wave.tmpl run\md-wave.gib
copy mdf\gib.mdf run\gib.mdf
copy flow\*.* run\*.*
copy flow\6cel2x.grd run\6cel.grd
copy flow\6celx2.enc run\6cel.enc
copy flow\6celx2.thd run\6cel.thd
cd run
mod -s "hshs" %5 md-wave.gib
mod -s "tptp" %6 md-wave.gib
mod -s "ddir" %7 md-wave.gib
mod -s "kt1" %8 md-wave.gib
mod -s "gammabr" %2 md-wave.gib
mod -s "aaa" %1 morf.gib
mod -s "xxx" %3 morf.gib
mod -s "yyy" %4 morf.gib
mod -s "kt2" %9 md-wave.gib
shift
mod -s "kt3" %9 md-wave.gib
shift
mod -s "kt4" %9 md-wave.gib
shift
mod -s "kt5" %9 md-wave.gib
shift
mod -s "kt6" %9 md-wave.gib
shift
mod -s "kt7" %9 md-wave.gib
shift
mod -s "kt8" %9 md-wave.gib
shift
mod -s "kt9" %9 md-wave.gib
shift
mod -s "x1" %9 md-wave.gib
shift
mod -s "x2" %9 md-wave.gib
shift
mod -s "x3" %9 md-wave.gib
shift
mod -s "x4" %9 md-wave.gib
shift
mod -s "x5" %9 md-wave.gib
shift
mod -s "x6" %9 md-wave.gib
shift
mod -s "x7" %9 md-wave.gib
shift
mod -s "x8" %9 md-wave.gib
shift
mod -s "x9" %9 md-wave.gib
shift
mod -s "rvw" %9 gib.mdf
copy d:\delft3d\bct\%9 gib.mdf
shift
mod -s "hhh" %9 gib.mdf
mod -s "hhh" %9 md-wave.gib
shift
mod -s "Tscale" %9 morf.inp
shift
mod -s "CJON" %9 md-wave.gib
shift
mod -s "ddd" %9 gib.mdf
shift
mod -s "dt1" %9 gib.mdf
shift
mod -s "dt2" %9 md-wave.gib
mod -s "dt2" %9 morf.gib
shift
mod -s "duur" %9 morf.gib
shift
cd..
copy d:\matlab\t02\dsdd1.m batch\dsdd1.m
copy run\mod.exe batch\mod.exe
cd batch
mod -s "fff" %9 dsdd1.m
del mod.exe
del *.log
cd..
shift
copy d:\delft3d\ini1\%9.ini run\test.ini
cd run
del mod.exe
@echo off
rem ===== Mor batch: morf.gib =====
@echo gib > runid
%D3D_HOME%\%ARCH%\menu\bin\d3dtmpl -mor -simulation -rm -use runid
set WRITE_WIDGET=yes
copy %D3D_HOME%\%ARCH%\menu\default\morjob.def morsys.job
echo morf.gib >> morsys.job
@echo on
@echo D3D-MOR run gib running now ....
@echo off
%D3D_HOME%\%ARCH%\mor\bin\morsyssm.exe > mor-gib.scr
@echo on
@echo D3D-MOR run gib finished
@echo off
REM==========copy output=======
cd..
cd output
md %9
cd..
copy run\*.* output\%9\*.*
del output\%9\hot*. *
del output\%9\*.grd
del output\%9\*.dep
del output\%9\*.rgh
del output\%9\*.bct
del output\%9\*.thd
del output\%9\*.bnd
del output\%9\*.bcc
del output\%9\*.enc
del output\%9\*.ldb
del output\%9\*.crs
del output\%9\*.obs
del output\%9\*.txt

copy run\gibsed2.dep output\%9\gibsed2.dep
copy run\6cell2.dep output\%9\6cell2.dep
cd batch
dsdd.m

% omzetten diepten
rrr=0;
infile3='d:\delft3d\batch\morfac=62\fff\run\trim-gib.dat'; file3=vs_use(infile3);
tmax1=vs_let(infile3,'map-info-series','ITMAPC'); tmax1=size(tmax1); tmax1=tmax1(1,1);
mmmax=vs_let(infile3,'map-const','MMAX'); nmax=vs_let(infile3,'map-const','NMAX');

ds0=vs_let(infile3,'map-sed-series',{1:1},'DPS',{1:nmax, 1:mmmax}); ds0=squeeze(ds0);
dep1(1,mmmax)=(sum(ds0(2:nmax-1,:)))/(nmax-2);
ds=vs_let(infile3,'map-sed-series',{tmax:tmax1},'DPS',{1:nmax, 1:mmmax}); ds=squeeze(ds);
dep1(tmax:mmmax)=(sum(ds(2:nmax-1,:)))/(nmax-2);
bodsed=vs_let(infile3,'map-sed-series',{tmax:tmax1},'BODSED'); bodsed=squeeze(bodsed);

save run\6cel2.dep ds -ascii
save run\gibsed2.dep bodsed -ascii
if(rrr==1)
!del run\*.gib
!del run\*.def
!del run\*.dat
!del run\*.ini
!del run\*.sed
endif

inifile.m

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% initial conditions file aanmaken
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
file3=vs_use('d:\delft3d\batch\morfac=62\norm\dir\output\fff\trim-gib.dat');
tmax2=vs_let(file3,'map-info-series','ITMAPC'); tmax2=size(tmax2);

mmax=vs_let(file3,'map-const','MMAX'); nmax=vs_let(file3,'map-const','NMAX'); nmid=nmax/2+1;
S1=vs_let(file3,'map-series',{tmax:tmax2},'S1'); S1=squeeze(S1);
U2=vs_let(file3,'map-series',{tmax-1:tmax1},'U1'); U2=squeeze(U2);
V2=vs_let(file3,'map-series',{tmax-1:tmax1},'V1'); V2=squeeze(V2);
conc=vs_let(file3,'map-series',{tmax:tmax2},'R1'); conc=squeeze(conc);

nul=S1*0; matrix=S1;
for i=1:10    matrix=[matrix;U2(:,:,i)];end
for i=1:10    matrix=[matrix;V2(:,:,i)];end
for i=1:10    matrix=[matrix;conc(:,:,i)];end

cd..
save run\test.ini matrix -ascii
cd batch
Appendix T. Determination of the transmission coefficients (1)

The water level and wave height decay are given in Figure A 64, Figure A 65 and Figure A 66 for respectively wave conditions 1, 2 and 3, for the situation without sheets, no ‘wavemask’ land-boundary. Transmission solely by breakwater, as in Figure 4-10 in §4.4.1.

Figure A 64 Wave condition 1: $H_{sig} = 1.5 \text{ m}$

Figure A 65 Wave condition 2: $H_{sig} = 2.5 \text{ m}$

Figure A 66 Wave condition 3: $H_{sig} = 3.5 \text{ m}$
Appendix U. Determination of the transmission coefficients (2)

The water level and wave height decay are given in Figure A 67, Figure A 68 and Figure A 69 for respectively wave conditions 1, 2 and 3, for the situation with one sheet, one imposed transmission coefficient. The used transmission values are placed in each lower figure (Wave height decay) and in Figure 4-11, §4.4.1.

Figure A 67  Wave condition 1: $H_{sig} = 1.5$ m

Figure A 68  Wave condition 2: $H_{sig} = 2.5$ m

Figure A 69  Wave condition 3: $H_{sig} = 3.5$ m
Appendix V. Determination of the transmission coefficients (3)

The water level and wave height decay are given in Figure A 70, Figure A 71 and Figure A 72 for respectively wave conditions 1, 2 and 3, for the situation with nine sheets. Adjusted transmission coefficients are used to obtain the right overall measured transmission. The used transmission values and their location can be seen in Figure 4-12. The product of the series of imposed transmission coefficient is placed in the wave height decay figure.
Appendix W. Determination of the transmission coefficients (4)

The water level and wave height decay are given in Figure A 73, Figure A 74 and Figure A 75 for respectively wave conditions 1, 2 and 3, as in Appendix V. Now the sheets’ location and values are slightly changed to obtain the measured wave set-up and the bottom friction and breaker parameter are adjusted to obtain the right wave height at location 4. The values and location of the imposed transmission coefficients can be seen in Figure 4-14, §4.4.2. The used bottom friction factors and breaker parameters are listed in Table 4-7.

Figure A 73  Wave condition 1: $H_{sp} = 1.5$ m

Figure A 74  Wave condition 2: $H_{sp} = 2.5$ m

Figure A 75  Wave condition 3: $H_{sp} = 3.5$ m
Appendix X. Determination of the transmission coefficients (5)

The water level and wave height decay are given in Figure A 76, Figure A 77 and Figure A 78 for respectively wave conditions 1, 2 and 3, as in Appendix W. That means, the right water levels and wave heights for locations 2, 3 and 4, but now the location of the sheets moved one grid cell onshore to simulate a transition zone, see §2.2.2. Doing this will result in a slightly higher water level at location 3, but still gives a good representation of the wave height decay.

Figure A 76 Wave condition 1: $H_{sig} = 1.5\ m$

Figure A 77 Wave condition 2: $H_{sig} = 2.5\ m$

Figure A 78 Wave condition 3: $H_{sig} = 3.5\ m$
Appendix Y. Determination of the transmission coefficients (6)

The water level and wave height decay are given in Figure A 76 for wave conditions 1, as in Appendix W, that means, the right water levels and wave heights for locations 2, 3 and 4, but now the location of the sheets moved two grid cells onshore to simulate a transition zone, see §2.2.2. Doing this will result in a slightly higher water level at location 3, but still gives a good representation of the wave height decay. Unfortunately, for wave condition 2 and 3 the computation became unstable, thus leaving us without results.

Figure A 79 Wave condition 1: $H_{sig} = 1.5$ m
Appendix Z. Calculation of transport rates

There are several ways to obtain sediment transport rates, in cross-shore direction over the submerged breakwater. From all four output files, three of them give information about sediment transports (Exception is the wave output file: 'hwgxygib.dat').

Map File
In the map output file 'trim-gib.dat' results are stored as a function of the entire area. There are several ways to obtain the cross-shore transport using the map file. First, the suspended and bed load transports in u-direction (resp. ssuu and sbuu) can be averaged for every cell, alongshore on the breakwater, see Figure A 80.

Another way to obtain the cross-shore sediment transport is to look at difference in available sediment between the breakwater and the shore, see Figure A 81. The difference in available sediment is multiplied by the cell area, added for all cells onshore off the breakwater, divided by its density and divided by the breakwater length and simulation time to obtain a 'm^3/m/hr' value.

Another way to determine the transport rates is to look at the mass balance between breakwater and shore using the bathymetry, see Figure A 82. Differences in depths are multiplied by the grid cell area, summed up and divided by the breakwater length and simulation time to obtain the cross-shore transport in m^3/m/hr.

Figure A 80 Suspended and bed-load transport in u-direction above the breakwater

Figure A 81 Available sediment in cross-shore direction at the beginning and end of a simulation

Figure A 82 Initial and end bathymetry
Communication File
In the communication file, data are stored for other modules. Here cross-shore transport can be obtained by integration of the difference in initial and end depth, as described above. The Generalised Langrangian Mean velocities are stored to the communication file and can therefore be used to calculate the suspended sediment advection transport manually by the sum of the product of layer height, velocity and sediment concentration, stored in the map file, see Figure A 83.

History File
In the history output file ‘trih-gib.dat’ results are stored as a function of time for specified monitoring points or cross-sections. In this study, observation points are located on the breakwater, in alongshore direction on every grid cell, see Figure A 84. These observation points give suspended and bed load transport in u and v direction for there own cell. Averaging the sum of suspended and bed load transport for each cell in u direction gives the cross-shore transport.

Another way is to obtain the transports directly from the cross-section, located above the breakwater in alongshore direction, see Figure A 85.
Table A 9 gives an overview of the several ways to obtain the cross-shore transport.

<table>
<thead>
<tr>
<th></th>
<th>Suspended and bed load transport in ( u ) direction (( \text{ssuu}, \text{sbuu} ))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Difference in available sediment, ( (\partial BODSED/\partial t)/\rho )</td>
</tr>
<tr>
<td></td>
<td>Difference in depth, ( \partial\text{dps}/\partial t )</td>
</tr>
<tr>
<td>Map-file</td>
<td></td>
</tr>
<tr>
<td>Communication file</td>
<td>Difference in depth, ( \partial\text{DP}/\partial t )</td>
</tr>
<tr>
<td></td>
<td>Velocities and concentrations, ( \sum(\text{dh}(i) \times \text{U}(i) \times \text{Conc}(i)) )</td>
</tr>
<tr>
<td>History File</td>
<td>Mean transport in ( u ) direction in observation points ( \text{ZSSU, ZSBU} )</td>
</tr>
<tr>
<td></td>
<td>Transport through cross-section ( \text{SBTR, SSTR} )</td>
</tr>
</tbody>
</table>

Table A 9 - Different ways to obtain the cross-shore transport rate

When comparing all these values, it appeared that they differ in both magnitude and in some cases of direction. As an example, the offshore losses are calculated for test T103, i.e. wave condition 3, Table A 10 shows the calculated offshore losses. It is clear to see that the negative offshore transport (onshore transport) is larger for the transport computations based on the bathymetry or based on the available sediment. This implies the sand production between the breakwater and the shore.

<table>
<thead>
<tr>
<th>Offshore transport (( m^3/m/hr ))</th>
<th>Condition 1</th>
<th>Condition 2</th>
<th>Condition 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Map file</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ssuu / sbuu</td>
<td>-0.002</td>
<td>-0.020</td>
<td>-0.076</td>
</tr>
<tr>
<td>bodsed</td>
<td>-0.012</td>
<td>-0.038</td>
<td>-0.210</td>
</tr>
<tr>
<td>dps</td>
<td>-0.011</td>
<td>-0.038</td>
<td>-0.211</td>
</tr>
<tr>
<td>History file</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>zssu / zsbu</td>
<td>+0.001</td>
<td>-0.006</td>
<td>-0.069</td>
</tr>
<tr>
<td>ssstr / sbtr</td>
<td>-0.001</td>
<td>-0.017</td>
<td>-0.082</td>
</tr>
</tbody>
</table>

Table A 10 - Offshore transport rates per wave condition (\( m^3/m/hr \))

In this case, the production is not calculated, but the example in §4.4.5 shows that the transport could change sign. E.g., when an onshore sedimentation volume is computed of 0.19 \( m^3/m/hr \) and due to lack of conservation of mass there is a net production of 0.22 \( m^3/m/hr \), it can be that this all added between the breakwater and the shore. Then one should have to subtract this value from the mean sedimentation, so that mean erosion would take place of 0.03 \( m^3/m/hr \).

Being conscious of this fact, it is decided to use the cross-shore transport calculation by means of the bathymetry for comparing calculated and measured transports, because these transports agree with the actual bathymetry.
When the transport rates would have been extracted directly from the trim file (ssuu and sbuu) it could be that these transports are offshore directed while in the mean time the mean bottom between the breakwater and the shore rises due to a certain production of sand, see Figure A 86. This phenomenon could be described as the transport paradox.

Figure A 86  Transport paradox: offshore transport and bed level rise
Appendix AA. Check on conservation of sand mass

The sand mass balance can be checked in two different ways; by means of the water depth and by means of the available sediment.

By means of the water depth
To check the conservation of mass, first all values are averaged in alongshore direction. Now, for every time step one can determine the area of sand above a specific reference depth, by integration of the water depth. Hereby the products of the depths and cell widths are summed up, see Figure A 87.

Figure A 87 Calculation of cross-shore area for mass conservation check
By means of the available sediment

Another way to determine the mass balance is to determine the available sediment at different times. Again all values are averaged in alongshore direction. Figure A 88 shows averaged available sediment values at the beginning and the end of a simulation. Dividing the available sediment (given in kg/m$^2$) by the density of the soil (1595 kg/m$^3$), gives a volume of sand including pores per square meter. Multiplying by the grid width and integrating results in a certain volume.

This volume should equal the calculated volume by the depth integration method above.

For both methods described above one have to bear in mind that a closed system is assumed or input at the open boundary is negligible small. The transports through the open boundary are checked every time, but have seemed to be negligible every time (less than 1% contribution).
Appendix BB. Mass balance for different calculations

For some runs it appeared that the total cross-shore area changed in time. Theoretically, it could be that sand comes in or goes out at the boundary, but in all cases this contribution appeared negligible. In Figure A 89, the areas are plotted in time for each test condition, with a time step of six seconds. In each column, from top to bottom, test number, significant wave height, contribution to the area (m$^2$) and the import of sediment at the boundary (m$^2$/hr) are listed.

For some runs it appeared that the total cross-shore area changed in time. Theoretically, it could be that sand comes in or goes out at the boundary, but in all cases this contribution appeared negligible. In Figure A 89, the areas are plotted in time for each test condition, with a time step of six seconds. In each column, from top to bottom, test number, significant wave height, contribution to the area (m$^2$) and the import of sediment at the boundary (m$^2$/hr) are listed.

![Figure A 89](image)

Figure A 89 Increase of the mass balance, dt=6 seconds

It seems that the ‘numerical production’ of sand is larger for heavier wave conditions, in this case 66% for wave condition 3 ($H_{sig} = 3.5$ m) although this condition takes place 16% in time! Between two wave conditions, no leap in mass appeared. This indicates an artificial production solely during the simulation and not between two wave conditions, as it was on an earlier stage, see the encountered problems in Appendix FF.

When the time step is reduced from six to three seconds, the development of the cross-section area goes as shown in Figure A 90.

![Figure A 90](image)

Figure A 90 Increase of the mass balance, dt=3 seconds

The ‘numerical production’ of sand hardly reduced.
Appendix BB

Again halving the time step to 1.5 second’s gives a relatively smaller reduction of the error in mass conservation, see Figure A 91. There still is a production of 15 m² and again the production by the large wave condition is determinative for this error.

Figure A 91  
Increase of the mass balance, dt=1.5 seconds

Instead of reducing the time step, also the morphological time scale can be reduced. The next figure shows the increase of the mass area with a time step of 3 seconds and a morphological time scale of one instead of six. Again, most of the increase takes place during the strongest wave conditions, but one can also see a reduction during the weakest condition.

Figure A 92  
Increase of the mass balance, dt=3 seconds, morfac=1
As a result, the reduction of time step and morphological time factor does result in a reduction of the production. Below Table A 11 is repeated for these reductions. Appendix CC continues investigating other parameters on their influence on the mass production.

<table>
<thead>
<tr>
<th>run name</th>
<th>Time step (sec)</th>
<th>Mean grid size (m)</th>
<th>Morphological time step (-)</th>
<th>Net production (m³/m/hr)</th>
<th>Calculated offshore losses (m³/m/hr)</th>
<th>Measured offshore losses (m³/m/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Norm001</td>
<td>6.0</td>
<td>5.0</td>
<td>6</td>
<td>0.17</td>
<td>-0.13</td>
<td>0.05</td>
</tr>
<tr>
<td>Norm002</td>
<td>3.0</td>
<td>5.0</td>
<td>6</td>
<td>0.17</td>
<td>-0.14</td>
<td>0.05</td>
</tr>
<tr>
<td>Norm003</td>
<td>1.5</td>
<td>5.0</td>
<td>6</td>
<td>0.12</td>
<td>-0.08</td>
<td>0.05</td>
</tr>
<tr>
<td>Norm0025</td>
<td>1.5</td>
<td>5.0</td>
<td>1</td>
<td>0.07</td>
<td>-0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>Norm0026</td>
<td>1.5</td>
<td>5.0</td>
<td>1</td>
<td>-0.01</td>
<td>-0.01</td>
<td>0.05</td>
</tr>
<tr>
<td>Half001</td>
<td>3.0</td>
<td>2.5</td>
<td>6</td>
<td>0.18</td>
<td>-0.07</td>
<td>0.05</td>
</tr>
<tr>
<td>Half002</td>
<td>1.5</td>
<td>2.5</td>
<td>6</td>
<td>0.15</td>
<td>-0.20</td>
<td>0.05</td>
</tr>
</tbody>
</table>

*Table A 11 Production and offshore transports for various time steps and morphological time factors*
Appendix CC. Influence of parameters on mass production

This appendix, the influence of parameters on the mass production is discussed, as referred to in §4.4.5. For each parameter, the same conditions are simulated for the same duration, but different values. The mass volume is illustrated in a graph in time. The extra area, i.e. the increase of mass in m$^3$/m, is given per parameter value and compared at the end of this appendix.

**THETSD**

To examine possible explanations for the net production a check is made for possible production at the shoreline. The factor THETSD, i.e. the fraction of erosion that is assigned to the adjacent dry cells is set on zero instead of 0.8, for a short simulation of two conditions.

![Figure A 93 THETSD = 0.0](image)

Increasing the THETSD value again from zero to 0.4 results in a production as shown in Figure A 94.

![Figure A 94 THETSD = 0.4](image)
Table A 12 gives an overview of the area increase for the different THETSD values. The mass production seems to decrease a bit with increasing THETSD values, but still unsatisfactory.

<table>
<thead>
<tr>
<th>THETSD</th>
<th>0.0</th>
<th>0.4</th>
<th>0.8</th>
<th>1.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area increase</td>
<td>2.00</td>
<td>1.80</td>
<td>1.77</td>
<td>1.76</td>
</tr>
</tbody>
</table>

Table A 12  
Area increase for different THETSD values

Transport parameters (see Appendix P)
The reference run results are depicted in Figure A 97. For the other runs, the specific settings are described in the caption.
Appendix CC

Figure A 98  \( \text{BED}=0.0 \)

Figure A 99  \( \text{SUS}=0.0 \)

Figure A 100  \( \text{BEDW}=0.0 \)
Table A 13 gives an overview of the production rates for different test runs; first for a reference run with customary settings as described in Appendix P and Appendix Q, followed by production rates for test runs with deviating parameter values.
Another check was conducted to investigate the influence of the fixed layer on possible sediment production. Hereby the available sediment was set equal, so that the breakwater becomes erodible. Although this does not reflect reality, it could give good insight in the problems. This simulation crashed soon after the warm-up time when the bottom updating was activated. Still, until that time, there was sand production as well.

![Figure A 104](image)

**Figure A 104** Simulation with excluded fixed layer or erodible breakwater
Appendix DD.  Erosion and transports

In this appendix, transport and erosion patterns are given for every simulation in Figure A 105 to Figure A 118. Concerning the transport, measured and calculated transports are given for the three wave conditions and averaged for all wave conditions in m³/m/hr.

Concerning the erosion-sedimentation pattern in the right figure, the initial bathymetry and the measured and calculated end bathymetry is given for the most transforming part of the cross-section.

For the most recent runs, also the production is given and the number of negative values for the available sediment (bodsed) and the number of negative concentration values. These numbers are counted for all the tests, output time steps and places. The captions declare the specific settings of the figures.

Figure A 105   Run001, dt=6 sec, morfac=6, THETSD=0.8

Figure A 106   Run0014, dt=6 sec, morfac=6, THETSD=0.0
Figure A 107  Run0015, dt=6 sec, morfac=6, THETSD=0.4

Figure A 108  Run0018, THETSD=0.4 dt 1.5-3.0-6.0 sec, hed=50

Figure A 109  Run0018, THETSD=1.0 dt 1.5-3.0-6.0 sec, hed=30
Figure A 110  Run0018. THETSD=0.8

Figure A 111  Run0018. THETSD=1.0, hed=60 instead of 10

Figure A 112  Run002, calculated and measured offshore losses and end bathymetries versus the initial bathymetry, d=3 sec., THETSD=0.8
Figure A 113  Run0021, calculated and measured offshore losses and end bathymetries versus the initial bathymetry, d=3 sec, sheets moved 5 meter onshore, THETSD=1.0

Figure A 114  Run0025, morfac=1 instead of 6.

Figure A 115  Run0026, morfac=1 instead of 6.
Halving the grid size leads to the next results:

<table>
<thead>
<tr>
<th>Hsig (m)</th>
<th>measured</th>
<th>calculated</th>
</tr>
</thead>
<tbody>
<tr>
<td>mean</td>
<td>0.05</td>
<td>-0.20</td>
</tr>
<tr>
<td>cond. 3</td>
<td>3.50</td>
<td>0.18</td>
</tr>
<tr>
<td>cond. 2</td>
<td>2.50</td>
<td>0.03</td>
</tr>
<tr>
<td>cond. 1</td>
<td>1.50</td>
<td>0.00</td>
</tr>
</tbody>
</table>

production: 0.16

# negative bosed values: 6694
# negative concentrations: 3399

Figure A 117  Run half001, THETSD=0.8, dt=3 sec.
Figure A.118  Run half002, THETSD=0.8, dt=1.5 sec
Appendix EE. Influence of bottom roughness

In this study, the bed roughness is computed according Manning. In this formulation, the Manning coefficient $n$ must be specified. The Chézy coefficient is calculated from:

$$C = \frac{h^{\frac{1}{3}}}{n}$$  \hspace{2cm} (A.64)

where

$h$ = water depth \hspace{1cm} [m]
$n$ = Manning coefficient \hspace{1cm} [-]

A typical Manning value is 0.028 for sand and 0.035 for rocks as used in this study. When the Chézy coefficient is obtained, one can calculate the Nikuradse equivalent geometrical roughness $k_s$ with:

$$k_s = \frac{12h}{10^{\left(\frac{C}{18}\right)}}$$  \hspace{2cm} (A.65)

or

$$k_s = \frac{12h}{10^{\left(\frac{C}{18}\right)}}$$  \hspace{2cm} (A.66)

where

$k_s$ = Nikuradse equivalent geometrical roughness \hspace{1cm} [m]

Subsequently, the reference height ‘a’ can be calculated by:

$$a = \min \left[ \max \left( ASFAC \cdot \frac{k_s}{2}, \frac{\Delta_r}{2}, 0.01h, 0.20h \right) \right]$$  \hspace{2cm} (A.67)

where

$ASFAC$ = user specified proportionality factor, see Appendix P \hspace{1cm} [-]
$k_s$ = Nikuradse equivalent geometrical roughness \hspace{1cm} [m]
$\Delta_r$ = wave-induced ripple height \hspace{1cm} [m]
$h$ = water depth \hspace{1cm} [m]

The next table shows the derivation of Van Rijn’s reference height at the breakwater for different Manning coefficients, which are plotted in Figure A.119.

<table>
<thead>
<tr>
<th>name</th>
<th>values</th>
<th>units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manning coefficient, n</td>
<td>0.035 0.028 0.020 0.015</td>
<td>-</td>
</tr>
<tr>
<td>water depth, h</td>
<td>1.19 1.19 1.19 1.19</td>
<td>m</td>
</tr>
<tr>
<td>Chezy coefficient, C</td>
<td>29 37 51 69</td>
<td>m$^{1/2}$ / s</td>
</tr>
<tr>
<td>geom. roughness, ks</td>
<td>0.33 0.13 0.02 0.00</td>
<td>m</td>
</tr>
<tr>
<td>ASKFACE</td>
<td>1 1 1 1</td>
<td>-</td>
</tr>
<tr>
<td>ASKFACE*ks</td>
<td>0.33 0.13 0.02 0.00</td>
<td>m</td>
</tr>
<tr>
<td>Delta_r</td>
<td>0 0 0 0</td>
<td>m</td>
</tr>
<tr>
<td>Delta_r/2</td>
<td>0 0 0 0</td>
<td>m</td>
</tr>
<tr>
<td>0.01*h</td>
<td>0.01 0.01 0.01 0.01</td>
<td>m</td>
</tr>
<tr>
<td>0.20*h</td>
<td>0.24 0.24 0.24 0.24</td>
<td>m</td>
</tr>
<tr>
<td>a=min(max(...),0.20h)</td>
<td>0.24 0.13 0.02 0.01</td>
<td>m</td>
</tr>
</tbody>
</table>

Table A.14 Reference height as a function of the Manning coefficient

Figure A.119 Reference height versus Manning coefficient
In appeared that halving the Manning coefficient more or less from 0.035 to 0.015 reduces the reference height a factor 24 from 0.24 to 0.01 meter. More important is that the number of covered layers is reduced from three to zero. The actual layer thickness above the breakwater is shown in both Table A 15 and Figure A 120 in percentages and absolute values (meters).

<table>
<thead>
<tr>
<th>name</th>
<th>values</th>
<th>units</th>
</tr>
</thead>
<tbody>
<tr>
<td>water depth, h (m)</td>
<td>1.18</td>
<td></td>
</tr>
<tr>
<td>layer number, k</td>
<td>1 2 3 4 5 6 7 8 9 10</td>
<td></td>
</tr>
<tr>
<td>relative layer thickness</td>
<td>5 5 10 15 15 15 15 10 5</td>
<td>5 %</td>
</tr>
<tr>
<td>absolute layer thickness</td>
<td>0.06 0.06 0.12 0.18 0.18 0.18 0.18 0.12 0.06 0.06</td>
<td>m</td>
</tr>
<tr>
<td>cumulative layers thickness</td>
<td>0.06 0.12 0.24 0.41 0.59 0.77 0.94 1.06 1.12 1.18</td>
<td>m</td>
</tr>
</tbody>
</table>

Table A 15 Relative, absolute and cumulative layer thickness

The actual influence of the bottom roughness can be seen in Figure A 121. Although the velocity, concentration and advection transport pattern hardly changes, the total transport reduces significantly, but the influence becomes less for smaller Manning coefficients. This is the same as expected from Table A 15. The reference height hardly changes for small Manning values.
Figure A 121  Influence of bottom roughness on transport rates above the breakwater crest
Appendix FF. Encountered problems

In this appendix some of the major setbacks are discussed, since the solution of these setbacks fulfil a significant part of the study, but are not put forward elsewhere in this report.

- Initial Bathymetry (1)
  First, the schematised bathymetry from the report (WL | Delft Hydraulics, 2000-e) was used, but it appeared that this bathymetry was slightly different from the more extensive bathymetry data set, measured in the wave flume. As initial bottom, the measured values from the measurement data set are interpolated to the grid points of both the curvilinear FLOW grid as the regular rectangular SWAN grid, as discussed in Appendix N and all the simulations had to be done over again.

- Initial Bathymetry (2)
  After some simulation time it appeared that there was a step in the bottom development in time. Further investigation revealed that the first bottom measurements in the flume were done without calibrating the gauges. An offset of 0.15 meter (prototype value) had to be added to the initial bottom. This exerted too much influence on the hydraulics that the calibration had to be redone with the new initial bathymetry.

- negative available sediment values in the trim file
  Net positive transport at the breakwater made the available sediment less than zero and the fixed bed layer not fixed at all; it appeared that the breakwater eroded. This problem is solved with a new morsyssm.exe version. Consequently, again all the transport computations were done over again.
  A smoothing effect took place by double interpolation from erosion/sedimentation values in water level points to bottom points and back to water level points. Checking the sediment mass balance showed a great difference between the end of the first run and the beginning of the succeeding run. This was done by comparing the sum of the products of the depth and width for each cell. Although output files showed a transport over the dam offshore, the bathymetry between the breakwater and the shore appeared to rise. This ‘numerical sedimentation’ is a factor 2 greater than erosion due to sediment transport!

Figure A 122 shows a part of the bathymetry at two time steps; the last one of a test run and the first one of the next test run. For the following explanation, one has to bear in mind the staggered grid as described in Figure A 51 of Appendix H. The calculated water level points (e.g. ‘dps-t=27’) are interpolated after the first run to obtain a value at the bottom points (e.g. ‘dp-run102’) that is stored in a communication file. The next run uses these values in the bottom points to calculate the values at the water level points (e.g. dps-t=28) again by interpolation. In this way, between every one of the 15 succeeding test, interpolation took place two times. These 30 interpolations resulted in a huge smoothing.

To solve this problem a new morsyssm executable (version 9b) made it possible to read depth values in water level points. Running this executable from Matlab made it possible to read the depth values at the water level points at the last time step from the output file from one run and save these values in a depth file to be read for the next run.
Appendix FF

Figure A 122  Depth points for succeeding time steps after interpolation

- Normally the bottom grid in SWAN is as large as the computational grid in SWAN. This causes differences in water level and bottom outside and inside the FLOW area, where these values are used from the FLOW computation. The SWAN bottom grid is changed to a regular rectangular grid smaller than the FLOW grid. By doing this, SWAN also computes with the computed FLOW bottom outside the FLOW grid, see Figure 3-7 in §3.6.3. With the new settings, calibration with bottom updating was done over again.

- Sediment hardening
  Initially, for every run only the calculated bathymetry of the previous run was used. To reduce the warm-up time, an initial condition file is used with non-zero water level and velocity values, see Appendix S. For every new wave condition, the new run used the same file for the available sediment, “bodsed.dep”, with zero available sediment at the breakwater. When sediment settled on the breakwater became hard because in the next run the available sediment was set on zero again, but the bottom depth had reduced, see Figure A 123 for an illustrating example. Figure A 124 shows the difference between the initial bottom in the water level points of the first run and the calculated bottom of the last run. The right vertical axis and the blue line show the available sediment at the beginning of the last run. It appeared that the breakwater’s cross-area increased with 1 cubic meter, especially at the toe.
  Later on, running Delft3D out of Matlab made it possible to read the available sediment at the last time step from one run, save the values in a new file to be read in the next run. This solved the problem of sediment hardening, but forced to use Matlab to avoid this problem.
Figure A 123 Depth and available sediment of two succeeding tests, illustrating example.

Figure A 124 Breakwater initial depth and available sediment at the beginning of the first and last run.

difference at breakwater=-1 [m²]  
sum dx(i)*dep(i) run 102=-5027 [m²](left:-5004 right:-23)  
sum dx(i)*dep(i) run 117=-5006 [m²](left:-4997.0 right:-9)  
difference run 102-117 = -22 [m²](left:-7 right:-15)  
losses at the boundary: 0.24 [m³/m/109.5hr]
• Numerical instability. The last two solutions created a new problem of instability. To solve this problem, for all three wave conditions runs were conducted. Here both the time step and morphological factor were decreased to prevent simulation crashes.

• Exhaustion of the available sediment. Initially 4000 kg/m², i.e. 2.5m was used, but near the shoreline, this value reached to zero. This is solved by increasing the available sediment value to 10,000 kg/m², i.e. 6.25 m. The great drawback was that the runs with bottom updating had to be redone.