Morphological pilot-experiment Vinjé-basin

Evaluation of a 3D morphological model

M.Sc. thesis

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Hydraulic and Geotechnical Engineering Group
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This paper evaluates the behaviour and the usefulness of a 3D morphological pilot-experiment. It reports the set up and execution of a 3D wave-current physical model in more detail. Furthermore, a selection of the collected data will be compared with computational output from a 3D morphological model. The bathymetries are mainly determined by a new waterline detection method. The application of this model will be evaluated.

The morphological pilot-experiment showed that despite the limited equipment it is feasible to carry out such test. However, the calibration of the upstream sand supply requires much attention. Comparing the measurements of the pilot-experiment with the computational results gives a qualitative agreement. Simultaneous observation of the process parameters such as longshore current and concentration shows significantly lower values than the computer predicted values. Analysing the images of the video with the new waterline detection technique appears to be working, but is not very accurate because of the sensitivity to the spurious reflections.

The morphological impact around structures would be more accurately determined with advanced measurement instruments, operated from a moveable carriage. For longer experimental programmes, an efficient sediment supply system and a more sophisticated sandtrapping and pumping system are necessary in order to keep the model running smoothly.
Preface

This study has been performed by Van Chuong Ha as a part of the requirement for the attainment of a M.Sc. degree from the Faculty of Civil Engineering and Geosciences, Hydraulics and Offshore Engineering Division of Delft University of Technology.

I would like to thank my supervisors, prof. ir. K. d’Angremond (Delft University of Technology) who had given me this great opportunity to finish the curriculum at the TUDelft, dr. ir. J. van de Graaff, (Delft University of Technology) for his guidance, and dr. ir. H.L. Fontijn (Delft University of Technology) for the interest in this pilot experiment. Last but not least, I would like to acknowledge dr. ir. J.A. Roelvink (Delft University of Technology and WL|Delft Hydraulics) for his patient guidance and support, and without whom I wouldn’t have been able to carry out this study.

Furthermore, I would like to thank my temporary colleagues and fellow graduate students at WL|Delft Hydraulics for showing their interest and compassion, which made my stay feel very comfortable to stay. Especially Giles Lesser, with whom I have spent two and a half weeks in De Voorst. By helping each other, we were able to carry out the pilot experiment successfully.

Finally, I want to thank my parents, brothers and sisters for their support and concern during my whole study in Delft.
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Summary

Morphological tests in a 3D basin have hardly been carried out at WL | delft hydraulics since the early ‘80s. This was mainly due to a couple of problems that occurred in doing such tests such as spurious waves due to first-order wave generation. These led to bottom disturbances which were often of the same order as the features under study. Also, due to scale effects (e.g. relatively large ripples) the results were hard to translate to prototype. In order to assess the present situation with regard to morphological experimenting in a 3D basin, a limited pilot experiment was carried out in the multidirectional wave basin at WL | delft hydraulics. The purpose of the pilot-experiment is to evaluate whether, at the present state-of-art, 3D data sets can be generated that can be used to validate numerical models.

This paper reports the execution of a morphological pilot-experiment that has been carried out in a wave-current basin in De Voorst. It contains the evaluation of the behaviour and the results of the physical model. Introducing and testing a new technique for measuring the bathymetry also form a part of the present study. In addition, a computer model was set up in order to make an initial comparison between the two models.

For convenience the pilot-experiment was divided into two parts. In the first part the test was carried out with a groyne and the second part with an offshore breakwater. A groyne consisting of concrete blocks and stones was constructed perpendicular to a straight shoreline and an offshore breakwater situated the same distance from the shore as the tip of the groyne. Both tests took place in the same basin which had an approximately 28 m long and 14 m wide beach. The beach made an angle of 20 degrees with the mean direction of the incoming waves. 100 Micron sand was used. Waves were multidirectional with a directional distribution with a standard deviation of 15 degrees. The peak period of the waves was 2 sec and significant wave heights of 10 and 15 cm were applied in the groyne and breakwater test respectively. A pump with a capacity of 250 l/s was used in order to supply the water for the longshore current. A circulation flow of approx. 125 l/s was applied for the 10 cm waves and approx. 187 l/s for the 15 cm waves.

During the tests measurements of relevant process parameters were collected. 6 Electro-Magnetic velocity Meters (E.M.S) and 6 gauges (W.H.M) were used to measure the velocity and the wave heights respectively. Sediment concentrations were collected by suction tubes. Circulation patterns were observed using small pieces of paper and floating plastic balls. The bathymetry was mainly measured by video observation during the lowering of the water level. Additionally, traditional levelling was applied in areas that were difficult to observe by video, i.e. in the shadow area of the breakwater.

This pilot experiment shows that extremely large ripples do not occur during both tests. Measured values of the ripples are approximately 1-1.5 cm height and 6-10 cm length.
Modification of the boundaries and frequent supply of sediment (600 l/h) at the upstream boundary seem to be the ingredients required to keep the beach stable.

Initial comparison between the investigated results of the physical model and computer model gives a good qualitative agreement. Although the determination of the bathymetry with the new waterline detection technique suffers from some inaccuracy, it gives a reasonable overall picture of morphological changes in the pilot experiment. Furthermore, the measured longshore currents as well as the sediment concentration are, in general, somewhat lower than the computational values. Computation of longshore discharge and sediment transport shows significant differences between the physical and computer models. In general, it can be concluded that the computer model produces a much higher concentration and a stronger longshore current, which results in higher sediment transport rate. Investigation of the bottom development in the area behind the breakwater (control area) also shows that accretion in the physical model is substantially lower than in the computer model. Apparently, the sedimentation produced by computer model is overestimated.

The report concludes that this morphological pilot experiment, despite limited equipment and time, could be successfully executed. The achieved result proves that it is feasible to carry out such experiments in the future. However, for generating 3D data sets that can be used to validate numerical models, the experiments need to be equipped with more advanced measuring-instruments. It also concludes that the use of the new waterline detection technique is limited, since it is sensitive to spurious effects on the images. To execute a morphological test in more quantitative as well as qualitative way the report recommends to use a more sophisticated sand supply and trapping system. To measure the bathymetry more accurately, we recommend to use a Profile Follower (profo), operated from a moveable beam.
I Introduction

Morphological changes in the coastal zone are the result of a complex interaction between waves, currents and sediment transport. To get an idea what is going on in this region, many experiments and measurements have been carried out to understand the underlying processes. However, much remains unknown about the physical processes that lead to these morphological changes. One of the methods to extend this knowledge is to study the interactions in a physical model. Once these processes are completely understood, numerical models can be set up to predict the developments of the coast. Physical models are easier to carry out and generally cheaper than carrying out field experiments, but may suffer from scale effects. This problem may be reduced by comparing laboratory and numerical models, without trying to scale the results up to prototype scale.

1.1 General

In the framework of the Delft Cluster project, a co-operation between five Delft-based technological institutes, a project is being carried out to make an inventory of the future demand for experimental facilities. One of the most uncertain aspects in this respect is the need for a 3D wave-current facility in which, apart from hydrodynamic tests, also morphological tests can be carried out.

Morphological tests in a 3D basin have hardly been carried out at WL| delft hydraulics since the early ‘80s. This was mainly due to a couple of problems that occurred in doing such tests:

- spurious waves due to first-order wave generation led to bottom disturbances which were often of the same order as the features under study;
- due to scale effects (e.g. relatively large ripples) the results were hard to translate to prototype.

Regarding the first point the wave generation techniques have been improved to such an extent that disturbances will be much smaller. The second point is addressed by changing the role of the experiments: no longer as a direct scale representation of reality, but rather as one of the tools in a composite modelling methodology: a controlled piece of reality is used to validate computer models. The computer models are then used to make predictions on a prototype scale.

The strong need for datasets to verify the recent 2DH or 3D morphological models is illustrated by the fact that some very old datasets (Keta Lagoon, 1994, groyne tests by Hulsbergen et al., 1977) are still used internationally, for lack of an alternative.

In order to assess the present situation with regard to morphological experimenting in a 3D basin, a limited pilot experiment was carried out in the multidirectional wave basin at WL | delft hydraulics. The purpose of the pilot-experiment is to evaluate whether, at the present state-of-art, datasets can be generated that can be used to validate numerical morphological models. To be suitable to this, experiments must meet a number of requirements:

- A clear distinction between areas affected by boundary disturbances, relatively undisturbed areas and the area of interest, where significant changes occur, i.e. near some structure.
• No spurious circulations, seiching or secondary waves.
• Simultaneous observation of the relevant process parameters such as wave heights, orbital velocities, vertical velocity, concentration; regular observation of bed forms and bed changes.

These requirements were tested in the existing multidirectional wave basin, which was equipped with a limited flow circulation capacity, strong enough to re-circulate the surfzone current.

In order to see whether such tests are feasible in the near future, with respect to the above mentioned requirements, the present study will consider the following subjects:

• to evaluate the behaviour of the physical model.
• to test a new technique for measuring the bathymetry using video images.
• to evaluate the usefulness of the physical model results by comparing them with numerical simulations.

1.2 Set up of the report

The outline of this report is as follow:

Chapter 2

In Chapter 2 the set up of the physical 3D wave-current model is described, including the measuring procedure and a plan of approach. Special attention is paid to the procedure of the bathymetry measuring technique; a video camera is used to record the waterline during the water lowering process.

Chapter 3

The construction of the structures and the execution of the tests are described in this chapter. Two kinds of structures have been applied in the morphological test; a groyne and an offshore breakwater. At the end of this chapter, two tables are given in which the activities, test codes and data are included.

Chapter 4

Data analysis of the measurements are presented in this chapter. The samples of concentration from different positions, wave heights and velocities are displayed. In order to check the discharge of the longshore current and the volume of the sand as supplied in physical model, an analysis is conducted by using the measurements performed in the basin. As a check on the measurements a simple analysis is carried out using the linear wave theory.

Chapter 5

The tools which are applied in this study will be described in more detail. First, a reference to the photogrammetric technique derived by Lipmann and Holman (1989) is made. Further, the basic principle of the new waterline detection technique applied in this study will be discussed. Evaluation and conclusions with respect to this technique are given in the last section.
Chapter 6

In order to compare the collected data with numerical results, a numerical model is set up. Therefore, an advanced computer program Delft3D is used. Measurements of the physical model will be compared to the computational outputs. Morphological changes behind the breakwater will be discussed in more detail.

Chapter 7

In the final chapter conclusions and recommendation with respect to this study are given.
2 Set up of experiments

2.1 The Vinjé-basin

The groyne test as well as the offshore breakwater test took place in the Vinjé Basin. Figure 2.1 shows the plan view of the pilot-experiment. On the first sight the set up of this experiment looks similarly to the one which was conducted by Reniers (1999). However, this pilot-experiment contains a moveable bed and the structures.

![Figure 2.1 Plan view wave-current basin](image)

The basin of approximately 40 meter long and 26 meter wide was used for the physical model. The beach profile is about 28 meter long and 14 meter wide. A cross-section of the beach profile is given in Figure 2.2.
2.1.1 Wave generation

Movements of individual wave paddles generate the irregular waves. Wave generation software steered the paddles so the desired wave spectrum can be created. The spectrum is single topped, with a peak frequency of 0.5 Hz and a directional spreading of 15 degrees. By manipulating the amplifier a certain significant wave height can be obtained.

The wave paddles are located at the offshore end of the basin, with a total length of 26.4 meter. The beach makes an angle of 20° with respect to the wave generator. This results in a total beach length of nearly 30 m. Because of the incident angle of the waves, the distance from the wave paddles to the beach varies along the beach. Both sides of the wave paddles are bordered by wave-guides.

2.1.2 Current generation

A pump with a maximum discharge capacity of 250 l/s is used to re-circulate the longshore current. Both sides of the wave-guides are terminated at some distance from the waterline to allow the in- and outflow of the longshore current. By adjusting the discharge the desired longshore current can be achieved (see Visser 1982).

The inflow opening has been constructed in such way that it is possible to distribute the pumped discharge along the cross-shore profile in order to match the wave-driven current distribution. Doing this one can increase the length over which the longshore current is uniform. Downstream the water is trapped in an outflow-basin, which is connected with the pump, in order to create a circulation flow.

2.1.3 Test section

To provide enough length to reach the equilibrium concentration over the depth, the measuring section was situated in such a way that the influences of the boundaries are small. Since the water enters the wave basin on a fixed rock-bed part it does not contain an initial sediment load. So the concentration profiles have to build up completely in the section with the sand bed. The top layer of the sand bed has a thickness of 0.20 m.

A walk bridge has been constructed across the shore, which is situated approximately 4.2 m in the upstream direction of the groyne. On the bridge six positions are marked from where the concentration measurements have been carried out.
2.2 Sediment

The bed material that has been used in the morphological model comes from the pit, because natural beach sand of 100 micron is very rare to get. By sieving the samples, the characteristics of the bed material are determined. $D_1$ is the sieve diameter passed by % by weight. Table 2.1 shows the result of the sieving-analysis. By plotting the sieve diameter against the cumulative percentage, $D_2$ can be determined (see Figure 2.3). Characteristic $D_2$ values are:

$D_{10} = 0.070 \text{ mm}$
$D_{50} = 0.114 \text{ mm}$
$D_{90} = 0.246 \text{ mm}$

<table>
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<tr>
<th>Sieve (mm)</th>
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<td>172.28</td>
</tr>
</tbody>
</table>

Table 2.1 Sieve analysis

Figure 2.3 Sieve curve
2.3 Bed-level measurements

2.3.1 Mean bed level

Since there was no profile follower (profo) available, the mean bed level was manually measured. The beam with instruments was used as a reference level (see Figure 2.4). First, the distance between the mean bed level and the chosen reference level was measured ($h_0$). Subsequently, when the basin was filled up with water, the distance between the water level and the reference level was also measured ($\delta$). These two distances determine eventually the mean bed.

![Diagram showing reference level, water level, and mean bed level](image)

Figure 2.4 Mean bed level

So the water depth relative to the mean bed level, $h$ is:

$$h = h_0 - \delta$$

2.4 Measurements

To collect all relevant parameters of the tests, different instruments are used. These are:

- WHM: wave height meters
- VCM: velocity meters
- Intake tubes: concentration measure instrument
- Video camera

The groyne causes totally different hydrodynamic processes than an offshore breakwater. In order to monitor these processes, the arrangement of instruments is dependent on the test. Initially, all VCM's were mounted on the beam across the shore. When the groyne or the offshore breakwater had been built, two WHM's and two VCM's were moved to another location. Their actual positions are given in Table 4.1 in Chapter 4. A video camera was used to observe the water level during the draining process. The main goal is to determine the bathymetry. This technique will be described in more detail in Chapter 5.
2.4.1 Wave height measurements

For each experiment, the wave spectrum was determined at different locations in the basin. Initially, all the wave height meters (WHM) were placed 1m from the toe of the beach, this is seawards from the long-hone current zone. In the later phase they were mounted on the beam where the velocity meters were also situated. This is 1m from the walk-bridge in the upstream direction across the beach. All information of the wave height meters was recorded in a computer. With standard wave analysis software information can be translated in wave spectrum parameters. These characteristic wave parameters are:

- peak frequency \( f_p \)
- peak period \( T_p \)
- zero-crossing period \( T = \sqrt{\frac{m_2}{m_3}} \)
- significant wave height \( H_{sw} = 4\sqrt{m_0} \)
- moments \( m_0, m_2, m_4 \)

2.4.2 Sediment concentration measurements

The sediment concentrations were measured using an array of 7 brass intake tubes with internal diameter of 3 mm. (see photo 2.1) This instrument was mounted on 6 different positions on the beam of the walk-bridge. The opening of the intake tubes was placed in the wave direction. Each intake tube has a number (1 to 7) corresponded with the bucket-number and was connected to a pump (see photo 2.3). The intake velocity of the mixture sediment and water is about 1.3 m/s.

The sediment concentrations were measured by the following procedure: First the mean bed level was measured. The concentration sampler is then adjusted just above the top of the ripples. The distance between the top and the trough of the ripples was in most cases approximately 1.5 cm. This means that the position of the lowest intake tube had to be adjusted 1.75 cm above the mean bed level.

Before the real test began, the physical model had to run for 30 minutes to generate the characteristic ripple pattern and to get an equilibrium situation in the test section between the inflow and outflow (relaxation time). After 30 minutes the test can start. It took about 14 to 15 minutes before all buckets were filled. The water was poured off and the remaining sediment was washed out in a volume meter (see photo 2.4). The volume meter consists of ten calibrated glass cylinders with different diameter. By reading the height of the filled cylinder the wet sediment volume is determined. A calibration table was used to determine the dry mass for each bucket. This table includes also the correction factor, called the trapping ratio. This trapping ratio is necessary, because of the fact that the sediment particles cannot completely follow the curved water particle trajectories to the intake tubes. After two tests, a reading of the bed level was again executed. By repeating of this procedure for every position, the mean bed level can be determined and also the concentrations.
Photo 2.1 Intake tubes (right) velocity meter (left)

Photo 2.2 Concentration measure instrument mounted on tripod

Photo 2.3 Pump installation with buckets

Photo 2.4 Calibrated volume meter
2.4.3 Current velocity measurements

A set of 6 two-dimensional Electro-Magnetic Velocity meters (E.M.S.) was used to measure the velocities in the longshore and cross-shore direction. This instrument generates an electromagnetic field at the measuring point, the degree of disturbance of this field is a measure of the water velocity. The array of six E.M.S. was attached to the beam to monitor the cross-shore distribution of the longshore current velocity. The distances between these meters were 1.5m. The current velocity meters were constantly set at 1/3 of the water depth from the bottom. Same procedure was carried out for all tests.

2.4.4 Bathymetry measurements

This pilot-experiment also offers a great possibility to test the new developed waterline detection method. Therefore, an ordinary video camera was used instead of an advanced digital camera (ARGUS). With the help of ARGUS-tools, images taking from the basin during the draining process will eventually be used to determine the bathymetry (see Chapter 5).

2.5 Program of experiments

2.5.1 Plan of approach

Preparation activities and measuring procedure during the experiments are:

* Preparation activities
  1. positioning the camera
  2. positioning the lights
  3. fill up the basin
  4. checking the water depth
  5. calibrating the velocity meters (E.M.S) in still water
  6. calibrating the wave height meters (W.H.M)
  7. generating the longshore current by activating the pump system
  8. calibrating the re-circulation system
  9. switching on the wave generator
  10. wait period (about half an hour) for generation of the characteristic ripple pattern

* Measuring procedures
  11. installation of the concentration sampler about 1.5 cm above the mean bed level
  12. starting computer program for data processing of wave height meters and for the velocity measuring devices
  13. start pumping out water-sediment samples at 7 heights above the mean bed level
  14. determination of sediment concentration with the volume meters and collecting the samples
  15. reading of wave spectra and wave parameters by running spectrum analyser program, stop processing
* Video observation
  16. switch on the lights and camera
  17. synchronise the time
  18. start recording video tape
  19. initial reading
  20. drain the basin by opening the valve
  21. reading water level each 5 minutes
* Last measurements
  22. measuring the bottom level

According to the pre-computation, a big quantity of sand has to be supplied at the updrift side of the model. There was no sophisticated sand supply-system, so manually feeding was necessary.

### 2.5.2 Video monitoring

A video camera was used to record the waterline during the draining process. From the financial point of view this was cost saving and accurate enough for the purpose of the pilot experiment. The purpose of taking video observations is, to detect the position of the waterline at different points in time. When also the water level is known, the bathymetry can be constructed using specially developed software.

The activities of making video are:
- positioning
- lights
- time synchronisation

To get a good view of the basin and the shoreline the camera must be placed on a high position. The tripod is not long enough for the camera to cover the whole area, therefore we had to use the wooden walk-bridge situated in the roof the shed. The tripod was mounted to beam of this bridge and the camera was attached to it. This way the camera can be moved in different directions.

Daylight through the windows gave a better quality of the picture than with the artificial light, however, a serious problem was the reflection. Additionally, it had an annoying impact of the view, especially when the reflection hit the waterline. A large black plastic curtain was hung up to avoid this problem.

An important aspect for recording and reading is time synchronisation. The time on the television screen must be correspond with the real time of readings during draining the basin. This is necessary in connection with image processing later on.
3 Execution of tests

3.1 Introduction

After the physical model has been constructed, calibration tests are carried out in order to obtain the proper longshore current. The calibration of the discharge which is applied in the model will be discussed in section 3.2. Furthermore, the construction of the structures, (groyne and offshore breakwater) including the lay-out is described in more detail in Sections 3.3.1 and 3.3.2.

The execution of the experiment was divided in two parts with a total time span of three weeks. In the first week, only test-running without structures was executed. The second week and the last week, tests were carried out with a groyne and an offshore breakwater respectively.

3.2 Calibration of discharge

In order to re-produce the longshore current generated by waves as if it involves a straight infinitely long and uniform beach, the wave basin was equipped with the openings in both wave guide walls, which were connected to a pump. However, the main problem still arises how to generate the proper longshore current in a wave basin. The criterion for generating the proper longshore current in a wave basin is that the profile is uniform along the beach and also the slope of the mean water level in the longshore direction is zero. Due to the limited length of the basin, the second criterion can hardly checked, because of the variation perpendicular to the shoreline caused by wave set-up and set-down. Besides, in this pilot-experiment no measuring instrument of the water level was available. Therefore, an alternative has been chosen to determine the longshore current. Figure 3.1 below shows the plan view of the wave-current basin which will illustrate the method of determining of the discharge.

When the inflow as well as the outflow are totally closed, a circulation flow is generated by wave actions. In this case \( Q_L \) is approximately equal to \( Q_{circ} \). In principle, when both boundaries are completely open and the pump is activated, \( Q_L \) will slightly increase and \( Q_{circ} \) will decrease. However, the quantities of these flow rate are still unknown. With the equation of conservative mass, \( Q_{pump} = Q_L - Q_{circ} \), an optimum of pump flow rate (\( Q_{circ}=0 \)) could be obtained by trial and error. The procedure is as follows:

1. determine \( Q_{pump} \) by reading the flow rate of the pump.
2. \( Q_L \) is determined by measuring the longshore velocity across the beach and integrated over the water depth and beach width.
3. \( Q_{circ} \) is the difference between \( Q_L \) and \( Q_{pump} \).
By repeating this procedure, it turns out that the optimum of $Q_{\text{pump}} = 125 \text{ l/s}$. In other words, with this pump discharge and one wave condition ($H_{\text{sig}} = 10 \text{ cm}$), the circulation flow $Q_{\text{circ}}$ is minimum. This result was also obtained with the pre-computation of the numerical model.

During the calibration test, several observation techniques have been applied to determine the longshore current. Pieces of paper and plastic balls are thrown across the beach in order to observe the flow pattern along the shoreline (see cross sections B, C and D in Figure 3.1). The same procedure was also used at the area adjacent the downstream boundary (A), in order to check the circulation flow in the basin.

Note that placing the velocity meters at cross sections B, C and D, the longshore flow rate could be simultaneously measured. A difference in distribution of the flow between B and D indicates that the flow rate along the shoreline is not uniform. However, due to a low budget of this pilot-experiment, a limited number of velocity meters was available so that no simultaneous measurements were performed.

Without structure on the beach, calibration tests show that the beach was still stable after a couple of hours of wave action. Also the longshore current seemed very stable during the calibration tests.
3.3 Structures

3.3.1 The groyne

Approximately after 6 hours of wave action, a groyne consisting of concrete blocks and stones was constructed perpendicularly to the straight shoreline in the region of interest. The initial beach profile had been adapted to the wave and current conditions. (see Figure 4.12) First, an analysis of physical processes will be made in order to see whether these processes also manifest in this model.

Morphological expectation

When a groyne is placed perpendicular to a sandy straight shoreline, the hydrodynamic processes will be locally influenced, which result in morphological changes around the structure. This is schematically given in Figure 3.3.

![Diagram](image)

--- initial shoreline

--- new shoreline

Figure 3.2 Hydrodynamic processes in vicinity of groyne

In principle the situation on the upstream side is mirrored on the downstream side. The upstream side is exposed to the wave attack, whereas the downstream side is sheltered. Due to diffraction in this area a complex current system occurs and also a complex sediment transport pattern. The major goal of this pilot-experiment is not to analyse the complex flow patterns and sediment transport around the structure, but rather observing the development of the morphological changes, like accretion in the upstream side and erosion at the downstream side. The main question is: does this phenomenon also occur in a physical model. In order to check this out, a groyne perpendicular to the beach is constructed. The lay-out and groyne parameters are presented below.
lay-out and data groyne

Lay-out:
beach slope  = 1:30
$L_z$  = 5.0 m : (initially length from original waterline)
h$_t$  = 0.195 m : water depth at tip of the groyne
$B_g$  = +/- 1 m : width of the groyne
stone slope  = 1:2 : slope at the tip of the groyne

wave data:
$H_s$  = 0.10 m : significant wave height
$T_p$  = 2.0 s : peak period
$\phi$  = 20° : angle of incident wave

other data:
$d_{50}$  = 100 $\mu$m : median diameter

Note that the water depth at the tip of the groyne is 0.195 m instead of 0.167 m (500/30), because when the structure was built the bed level had already adapted to wave and current conditions.

Results

A remarkable observation was that much less sand is trapped directly at the upstream side of the groyne than it was expected. Some accretion was found near the tip of the groyne in the downstream area. A lot of sand was trapped in the outflow. It appeared that the groyne strongly affected the upstream boundary, and consequently the velocity profile. The curved waterline along the beach indicates that the beach has changed. This is in line with the expectation near the groyne, but not at the boundary. Obviously, the upstream boundary is very sensitive in this case, because of the limited length of the beach. The beach was stable after adjusting the upstream boundary and regularly supplying sand in this zone.
3.3.2 The offshore breakwater

With the same concept as the groyne an offshore breakwater is constructed on the beach slope. The dimensions of the offshore breakwater are given below.

\[
\begin{align*}
L_b &= 3.34 \text{ m} & \text{: length of the breakwater} \\
h_b &= 0.17 \text{ m} & \text{: water depth at the wave side of the breakwater} \\
B_b &= +/- 1 \text{ m} & \text{: width of the breakwater} \\
X_b &= 5.0 \text{ m} & \text{: distance between initial waterline and the breakwater} \\
H_s &= 0.15 \text{ m} & \text{: significant wave heigh}
\end{align*}
\]

**Morphological expectation**

Wave-driven longshore current contains suspended sediment. In the shadow area of the breakwater the wave-driven current and the stirring effects of waves are less compared with the exposed area. The reduction of the sediment transport capacity in the sheltered area, results in accretion and formation of a salient. In the exposed area at the downstream side of the breakwater, wave-driven longshore current is again being generated by oblique incident waves. A positive sediment transport gradient here causes severe erosion.

Photo 3.3 Top view breakwater  
Photo 3.4 A submerged tombolo
Results

After four days of running the model (with interruption: net 20h) the evolution of an underwater tombolo was clearly visible. Observation showed that erosion occurred at both sides of the breakwater, at the upstream as well as at the downstream side. This phenomenon seemed almost in line with the effects of normal incident waves. It is not surprising, since the angle of incident waves in this test is also small (20° with the normal). In Chapter 6 this area will be considered in more detail.

Overview of activities and executed tests

Measurements that have been taken and activities during the tests are given in the next table.

<table>
<thead>
<tr>
<th>Activities</th>
<th>10 oct</th>
<th>11 oct</th>
<th>12 oct</th>
<th>13 oct</th>
<th>16 oct</th>
<th>17 oct</th>
<th>18 oct</th>
<th>19 oct</th>
<th>20 oct</th>
<th>7 nov</th>
<th>8 nov</th>
<th>9 nov</th>
<th>10 nov</th>
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<tbody>
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<td>construct walk bridge</td>
<td></td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>x</td>
<td></td>
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</tr>
<tr>
<td>experiment light pos.</td>
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<td></td>
<td></td>
<td></td>
<td>x</td>
<td>x</td>
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<td></td>
<td></td>
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</tr>
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<td>x</td>
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<td></td>
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<td></td>
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<td></td>
<td></td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
</tr>
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<td></td>
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<td></td>
<td></td>
<td>x</td>
<td></td>
<td></td>
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</tr>
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<td>construct breakwater</td>
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<td></td>
<td></td>
<td></td>
<td>x</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
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<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
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<td>x</td>
<td></td>
<td>x</td>
<td>x</td>
<td></td>
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</tr>
<tr>
<td>wave height meas.</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>concentration meas.</td>
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<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>draining basin</td>
<td></td>
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<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3.1 Activities during the tests

In order to get an overview of the whole experiment, a table is set up to distinguish the tests. An overview of the executed tests including different parameters is given in a Table 3.2.

<table>
<thead>
<tr>
<th>Testcode</th>
<th>Structure</th>
<th>Hs</th>
<th>Tp</th>
<th>Discharge (250 l/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C...</td>
<td>No</td>
<td>10/15cm</td>
<td>2s</td>
<td>50%/75%/95%</td>
</tr>
<tr>
<td>C...</td>
<td>groyne</td>
<td>10cm</td>
<td>2s</td>
<td>50%</td>
</tr>
<tr>
<td>G...</td>
<td>groyne</td>
<td>10/15cm</td>
<td>2s</td>
<td>50%</td>
</tr>
<tr>
<td>C...</td>
<td>offshore breakwater</td>
<td>15cm</td>
<td>2s</td>
<td>75%</td>
</tr>
<tr>
<td>B...</td>
<td>offshore breakwater</td>
<td>15cm</td>
<td>2s</td>
<td>75%</td>
</tr>
</tbody>
</table>

Table 3.2 Executed tests including parameters

The capital letters C, G and B represent respectively Calibration, Groyne and Breakwater test. The tests are also conducted in the same order as they are arranged in the table. All data are stored on a cd-rom.
The values given in Table 3.3 represent the net duration of several tests in case with structures and wave action.

<table>
<thead>
<tr>
<th>Date</th>
<th>Groynes test</th>
<th>Breakwater test</th>
</tr>
</thead>
<tbody>
<tr>
<td>16 October</td>
<td>+/-2 h</td>
<td>--</td>
</tr>
<tr>
<td>17 October</td>
<td>+/-2 h</td>
<td>--</td>
</tr>
<tr>
<td>18 October</td>
<td>+/-4 h</td>
<td>--</td>
</tr>
<tr>
<td>19 October</td>
<td>+/-6 h</td>
<td>--</td>
</tr>
<tr>
<td>20 October</td>
<td>+/-6 h</td>
<td>--</td>
</tr>
<tr>
<td>7 November</td>
<td>--</td>
<td>+/-2 h</td>
</tr>
<tr>
<td>8 November</td>
<td>--</td>
<td>+/-6 h</td>
</tr>
<tr>
<td>9 November</td>
<td>--</td>
<td>+/-6 h</td>
</tr>
<tr>
<td>10 November</td>
<td>--</td>
<td>+/-6 h</td>
</tr>
<tr>
<td>Total duration (hours)</td>
<td>+/-20 h</td>
<td>+/-20 h</td>
</tr>
</tbody>
</table>

Table 3.3 Duration of tests
4 Results of experiments

4.1 Introduction

In this chapter the results of the measurements of the tests are analysed and displayed. The position of all measuring instruments is presented in Table 4.1. For the analysis a selection of the collected data of the velocity and wave height is chosen. The following parameters will be analysed:

- The concentration distribution over the water depth.
- The longshore velocity at 1/3 of the water depth.
- Computation of orbital motions at 1/3 of the water depth using the measured wave heights.
- Computation of the longshore flow rate.
- Computation of the longshore suspended sediment transport.
- Bed level change with time in the groyne test.

Since the velocities are measured at a fixed height above the bed, assumptions need to be made in order to check the applied discharge and the supplied sediment. In Section 5 the results of the computer-analysed pictures will be presented. The morphological changes around the groyne and the offshore breakwater will be considered in more detail in Chapter 6.

4.2 The sediment concentrations

The results of measurements are presented in Figure 4.2. These concentration profiles are time- and bed averaged. Time- and bed averaging is carried out as follow:

- The height of each intake tube above the mean bed level is the averaged height of the two measurements determined before and after each test.
- The measured concentrations for each intake tube are averaged over two tests.

In case when the concentration of the two measurements differs too much (>20%), the tests are repeated. The intake points of concentration samples are presented in Figure 4.1.

![Diagram](https://via.placeholder.com/150)

Figure 4.1 Position of measuring points
The distance between measuring points and their positions are given in Table 4.1.

<table>
<thead>
<tr>
<th>measuring points</th>
<th>x co-ordinate [m]</th>
<th>y co-ordinate [m]</th>
<th>water depth [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>x1 vcm1 whm1</td>
<td>12.0</td>
<td>12.0</td>
<td>11.8</td>
</tr>
<tr>
<td>x2 vcm2 whm2</td>
<td>10.5</td>
<td>10.5</td>
<td>11.8</td>
</tr>
<tr>
<td>x3 vcm3 whm3</td>
<td>9.0</td>
<td>9.0</td>
<td>11.8</td>
</tr>
<tr>
<td>x4 vcm4 whm4</td>
<td>7.5</td>
<td>7.5</td>
<td>11.8</td>
</tr>
<tr>
<td>x5 vcm5 whm5</td>
<td>6.0</td>
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<td>11.8</td>
</tr>
<tr>
<td>x6 vcm6 whm6</td>
<td>4.5</td>
<td>4.5</td>
<td>11.8</td>
</tr>
<tr>
<td>GRSM</td>
<td>15.0</td>
<td>10.8</td>
<td>0.50</td>
</tr>
<tr>
<td>tip groyne</td>
<td>9.2</td>
<td>15</td>
<td>0.195</td>
</tr>
<tr>
<td>tripod</td>
<td>6.0</td>
<td>11.8</td>
<td>0.10</td>
</tr>
<tr>
<td>behind breakwater</td>
<td>6.0</td>
<td>14.0</td>
<td>0.09</td>
</tr>
<tr>
<td>initial water line</td>
<td>3.0</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Table 4.1 Co-ordinates of measuring points

Note that at the position of the directional meter (GRSM) concentration has not been taken. The position of the tripod and behind the breakwater are given in Figure 4.6.

![Concentration profiles at the walk-bridge with Hs=10 cm and Qp=125 l/s](image)

Figure 4.2 Concentration profiles at the walk-bridge with Hs=10 cm and Qp=125 l/s

Each profile in Figure 4.2 represents the concentration distribution over the water depth corresponding to the point across the beach. The measurements clearly show that the concentration increases with decreasing water depth. This can be rationally explained; breaking waves generate turbulent motion, which causes the stirring up of sand into suspension. And plunging breakers induce strong jets penetrating to the sand-bed resulting in the generation of large sediment concentration. (van Rijn 1993)

In the groyne test 15 cm wave height was also applied, but not for long. Concentration measurements with 15 cm wave height are presented in Figure 4.3.
Figure 4.3 Concentration profiles at the walk-bridge with $H_s=15$ cm and $Q_p=187.5$ l/s

In Figure 4.2 and 4.3 an overview is given of concentration measurements for $H_{sig}=10$ cm, $Q_p=125$ l/s and $H_{sig}=15$ cm, $Q_p=187.5$ l/s respectively. It can be said that increasing the significant wave height, $H_{sig}$, and the pump capacity, only leads to an increase of the concentrations in the bottom layers, and hardly in the layers near the water surface.

Figure 4.4 Concentration profiles measured at the same offshore distance with $H_s=10$ cm and $Q_p=125$ l/s

The profiles in Figure 4.4 present sediment concentration at the tip of the groyne and on the beam (measure point x3 in Figure 4.1). We can clearly see that the sediment concentration at the tip of the groyne in the upper layer is much higher than at the measure point x3. So the stirred up sediment in the breaker zone does not deposit in the upstream side of the groyne, but is bypassing the groyne.
Figure 4.5 Concentration profiles near the breakwater with $H_s = 15$ cm and $Q_p = 212.5$ l/s

Figure 4.5 shows that the shape of concentration distributions are almost identical, although the concentration in the sheltered area is substantial lower than in the exposed area. The wave action behind the breakwater is much lower than in exposed area, which results in lower concentration because of less stirring. This is according to the expectation. The co-ordinates of intake points are given in Table 4.1 and the positions in Figure 4.6.

4.3 Velocities

Velocities are measured at 1/3 of the water depth. The following standard parameters are computed:
- mean velocity in x- and y-directions
- orbital motions

The data set used in the following analysis is from the test codes G100 and G500. G100 stands for groyne test in which 10cm wave height is applied and G500 for 15cm wave height.
4.3.1 Longshore and cross-shore current

The mean values in $x$- and $y$- directions give the cross-shore and longshore currents respectively.

![Figure 4.7 Longshore and cross-shore current with $H_s=10$ cm and $H_s=15$ cm.](image)

Measurements displayed in these figures show that both the longshore current and cross-shore currents increase in the breaker zone. Note that the magnitude of the mean longshore and cross-shore currents are almost of the same order in the test G100. Unfortunately, there are four measuring points across the beach instead of six. Two of the velocity meters were moved to tip of the groyne.

4.3.2 Orbital motions

As a check on the measurements a simple analysis is carried out using linear theory. The amplitude of horizontal velocity of a particle in the direction of wave propagation, derived for the linear theory is given by:

$$u = \omega a \frac{\cosh(k(h+z))}{\sinh(kh)}$$  \hspace{1cm} (4.1)  

with:

- $u$: orbital horizontal velocity amplitude at z-level \hspace{1cm} [m/s]
- $\omega$: frequency \hspace{1cm} [rad/s]
- $a$: wave amplitude ($a=1/2h_{wp}$) \hspace{1cm} [m]
- $k$: wave number ($k=2\pi/L$) \hspace{1cm} [rad/m]
- $h$: water depth \hspace{1cm} [m]
- $z$: considered height \hspace{1cm} [m]
Using the $H_{rms}$ wave heights ($H_{rms} = H_{rms} / \sqrt{2}$), wavelength and the wave period, the orbital motions can be computed as follows:

\[ a = \frac{1}{2} H_{rms} \]
\[ k = 2\pi / L \]
\[ \omega = 2\pi / T \]
\[ z = -2 / 3 \ h \]

Substitution of the above parameters in eq. (4.1) yields:

\[ \frac{\omega}{T} = \frac{\pi}{H_{rms}} \frac{\cosh\left(\frac{2}{3}\pi \frac{h}{L}\right)}{\sinh\left(2\pi \frac{h}{L}\right)} \]  

(4.2)

With the dispersion relation and length:

\[ \omega^2 = gk \tanh(kh) \]  

(4.3)

\[ L_0 = \frac{gT^2}{2\pi} \]  

(4.4)

the relation between $L$ and $L_0$ can be derived:

\[ L = L_0 \tanh\left(\frac{2\pi h}{L}\right) \]  

(4.5)

The wavelength, $L$, at a certain depth can be determined iteratively or with the help of a table.

From the velocity amplitude computed by eq. 4.2 we can estimate the standard deviation of the orbital velocity as:

\[ \sigma_{u_{orb}} = \frac{u_{orb}}{\sqrt{2}} \]  

(4.6)

This can be compared to the measured standard deviation of the orbital velocity defined by:

\[ \sigma_{u_{orb}} = \sqrt{\sigma^2_u + \sigma^2_v} \]  

(4.7)

Horizontal velocity amplitudes computed with eq. (4.2) and analysis of the measurements are shown in Figure 4.8.
Figure 4.8 Orbital motions across the beach

The calculation of orbital velocity amplitudes only satisfies the measured value in the breaker zone (x=7-10m). According to the calculation, the order of magnitude of the amplitudes remains rather constant in the surf zone, whereas the measured values rapidly decrease outside the surf zone. Accordingly as the waterline approaches, the difference between the calculated and the measured values becomes considerable.

Figure 4.9 Measured wave heights across the beach.

Measurement of the wave heights (G100 and G500) looks satisfactory. Figure 4.9 shows that waves start breaking at 11m from the reference point. Broken waves loose their energy, which results in decreasing of the height according as they approach the shoreline.
4.4 Flow rate

Some assumptions need to be made to allow the calculation of the flow rate. Firstly, since all velocity measurements are carried out at one fixed level (1/3 of water depth) across the beach, the vertical distribution is not known. It is assumed to be logarithmic, using the following formula:

\[ U(z) = \sqrt{\frac{\tau_e}{\rho \kappa}} \frac{1}{\ln\left(\frac{z}{z_0}\right)} \]  

(4.8)

The velocity \( U(z) \) can be calculated as follows:

\[ U(z) = \sqrt{\frac{\tau_e}{\rho \kappa}} \frac{1}{\ln\left(\frac{z_0}{z_1}\right)} \]

and

\[ U(z) = \sqrt{\frac{\tau_e}{\rho \kappa}} \frac{1}{\ln\left(\frac{z}{z_0}\right)} \]

yielding

\[ U(z) = U(z_0) \frac{\ln\left(\frac{z}{z_0}\right)}{\ln\left(\frac{z_0}{z_1}\right)} = U(z_0) \frac{\ln(z) - \ln(z_0)}{\ln(z_0) - \ln(z_1)} \]  

(4.10)

with:

- \( z \) = water depth \([\text{m}]\)
- \( z_b \) = the height of the measuring device \([\text{m}]\)
- \( z_0 \) = \( \frac{\text{ks}}{30} = 0.0005 \) (\( \text{ks} = 0.015 \text{m height of ripples} \)) \([\text{m}]\)
- \( U(z_b) \) = the measured velocity at 1/3 of the water depth \([\text{m/s}]\)
- \( \rho \) = mass density of water \([\text{kg/m}^3]\)
- \( \tau_e \) = shear stress for current \([\text{N/m}^2]\)
- \( \kappa \) = Von Karman coefficient (0.4) \([-]\)

Secondly, as it can be seen in the previous section, there are just four points across the beach where the velocities were measured. Assuming that the longshore velocity decreases in offshore direction and is zero at approximately \( x = 14 \text{m} \), the longshore velocity profile can be made. (see Figure 4.11).
Integration of $U(z)$ over the water depth and over the width gives the flow rate of the discharge. Calculation of the discharge with the 4 measured velocity in case with $H_s=10$ cm gives qualitatively a good agreement. The difference between the pump and the measured discharge is approximately 13.6 l/s.

Increasing the pump rate discharge and wave heights ($Q_p=187.5$ l/s, $H_s=15$ cm) it turns out that the calculated value is 5 l/s higher than the pump discharge. This might lay on the fact that the estimation of the missing longshore velocities in seawards direction is somewhat too high. The results are present in Figures H-3 ang H-4.

### 4.5 Longshore sediment transport

In this section, a computation of sediment transport is carried in order to check the amount of sand, which was supplied during the test. Sediment transport can be described as the product of velocity and the concentration distribution integrated over the water depth. If the concentration distribution and the velocity as functions of the water depth are known, then the sediment transport can be obtained by integrating over the depth after multiplying them. Oscillating water movements and current movements induce the stirring up process. Assuming that the suspended sediment velocity is equal to the fluid velocity, sediment transport can be computed with the formula:

$$
S_{st}(x,y,t) = \int_0^{h(x,y)+\eta(x,y,t)} c(x,y,z,t) \cdot U_{st}(x,y,z,t) dz
$$

(4.11)

with:

- $S_{st}(x,y,t)$: Local instantaneous sediment transport rate per unit width in long-shore direction. [kg/s/m]
- $S_{st}(x,y,t)$: Local instantaneous sediment transport rate per unit width in cross-shore direction. [kg/s/m]
- $c(x,y,z,t)$: Local instantaneous sediment concentration [kg/m$^3$]
- $U_{st}(x,y,z,t)$: Local instantaneous x-component of the fluid velocity [m/s]
- $x$: Horizontal coordinate in cross-shore direction [m]
- $y$: Horizontal coordinate in long-shore direction [m]
- $z$: Height above the mean bed level [m]
- $t$: Time [s]
- $\eta$: Elevation of water surface [m]
- $h(x,y)$: Water depth [m]
A former study (Bosman, 1985) has shown that local and instantaneous concentration measurements display a great variation, even when the measurements are carried out under identical conditions. He suggested that it is not practical to relate the instantaneous concentrations to the instantaneous fluid velocities. In this study time- and bed-averaging method is applied. The equation (4.4) is then reduced.

\[
S_{*,*}(y) = \int_0^h c(y,z) \cdot \bar{U}_{*,*}(y,z) dz
\]  

(4.12)

As mentioned before, the velocity was measured at a fixed level, so the distribution of vertical velocity in the longshore direction can not be determined exactly. However, to obtain an indication of the longshore sediment transport rate, a simplified computation is carried out. Integration of equation (4.11) over the beach width yields for the longshore suspended transport 42.5 gr/s. This is approximately equivalent to 100 litre of sand per hour.

According to CERC-formula (Coastal Engineering Volume II, 1995) sediment transport \( \sim H_s^{2.5} \). Estimation for \( H_s=15 \) cm:

\[
100l/h \cdot \left(\frac{15}{10}\right)^{2.5} \approx 280l/h
\]

During the execution of the breakwater test, it turned out that approximately 600 litre of sand per hour had to be supplied to keep the model stable. This is about two times higher than the estimation. Concentration measurements across the shore were unfortunately not carried out during the breakwater test, because the walk bridge was then removed. So comparison of the longshore sediment transport in this case is not possible. In Chapter 6, this result will be considered in more detail by verifying with the numerical model.
4.6 Bottom development at the beam

Several measurements of the bottom profile during the groyne test were conducted at the beam. The results are presented in Figure 4.12. This Figure shows clearly that the initial bottom profile quickly adapts to the current and wave conditions. After 6 net hours of wave action erosion in the area between the waterline (x=3m) and and x = 10 m was visible. Breaking waves cause stirring up of sand at the bottom layer; in combination with the return flow (undertow) a net offshore transport of sand occurs.

![Bed level change; groyne test](image)

Figure 4.12 Bottom level change in time

4.7 Preliminary conclusions of measurements

- Increasing current strength and wave height cause an increase of the concentration near the bottom.

- The concentration distribution over the depth at the tip of the groyne has a different profile; a higher concentration in the upper layer near the water surface.

- Comparing the orbital motions calculated by linear theory with the measured values shows significant differences.

- Manually measured depth profile shows that the beach slope becomes less steep than the original profile (1:30).

- In order to keep the beach stable, manually supplying sand at the upstream boundary is necessary. Sand is easily and effectively supplied by tipping wheelbarrow loads of sand on the rock bed near the inflow side of the model.
5  Image processing

5.1  Introduction

The Argus video technique, developed at the Coastal Imaging Lab, Oregon State University (Holman et al., 1993) is an invention that contributes to the research in the coastal zone. A new development is the automatic detection of the waterline in order to monitor the intertidal beach (Aarninkhof, 1999). This technique becomes increasingly important for the study of near shore coastal processes. Application on the beach of Egmond shows that this technique yields promising results.

In this pilot experiment, no moving carriage was available to carry out levelling. Therefore a cheap alternative was tried. At the end of each day the water level was lowered slowly, at the same time a video observation was made of the area of interest, and the water level was recorded every five minutes. By combining the known water level and the waterline derived from the video images, a full 3D bathymetry could be obtained. In the following we will discuss the details of this technique.

In this chapter, first a reference to photogrammetric technique derived by Lippman and Holman (1998) has to be made, in order to understand the principle of rectification. The basic principle of the identification-technique developed by Roelvink will be discussed in paragraph 5.3. Transformation from the pixels on the image to the real world co-ordinates and its derivation after Lippmann and Holman is set out in 5.3.1. The flow chart in Figure 5.1 shows the steps of determination of the bathymetry.

![Diagram of steps in determining bathymetry](image)

Figure 5.1 Steps of defining bathymetry

Results and evaluation of processed images are eventually presented in the final paragraph.
5.2 Image dilution

The draining process takes about 45 to 50 minutes. This implies that within this time span many images have been recorded. A range of images contains a lot of pictures which are not relevant for the study will be removed. Dilution of the images reduces calculation steps and time. With a computer program images of each 40 seconds from the tape have been taken. Since the waterline does not change much within a time interval of 40 seconds, (this is about 0.6 cm in the z-level), it is acceptable. Examples of the dilution of images are given in Appendix B.

5.3 Waterline detection model

An image is built up in color-pixels. The color of each pixel is defined as a mixture of three base colors: red, green and blue (RGB). Using the information stored in the color pixels, the waterline can be identified. This has been done in the former study (e.g. Caljouw, 2000). However, this technique could not be used in this study since the images only showed shades of murky brown. In the present study the waterline detection-technique is based on the grayscale intensity image converted from an RGB image. Images processing toolbox stores an intensity image as a single matrix, with each element of the matrix corresponding to one image pixel. The relationship between the value in the image matrix and the colour map depends on which class is being used. The matrix can be of class double, in which case it contains values in the range [0,1]. The elements in the intensity matrix represent various intensities, or grey levels, where the intensity 0 represents black and the intensity 1 represents full intensity, or white. Using these properties, the waterline can be identified.

The gradient of color-intensity is the key to the present technique. Relative differences in intensity of the pixels on a transect are not big, except at the transition zone. Here, the gradient of the gray-intensity at the waterline is maximum. Determining the pixels with the greatest gradient defines the waterline.

![Figure 5.2 Definition sketch.](image)

The complete source codes of this waterline detection model are presented in Appendix A.
The results of waterline detection-technique from an image.

![Image of waterline detection](image.png)

Figure 5.3 Snap shot image

![Graph of waterline detection](graph.png)

Figure 5.4 Detected points of waterline

The left panel is a colour picture, which will be transformed into a grey-scale image. The right panel presents the detected pixels of the waterline.

### 5.3.1 Assigning z-level

Once the U- and V-coordinates are identified, a third dimension (z-level) is assigned to this point. During the draining process, z-levels are manually measured within a time span of 5 minutes. As mentioned before, the time plays a very important role in this feature. It must be equivalent with the time on the video-camera, in order to determine the z-level of an image at a certain time. Figure 5.5 on the left side shows the connection of the identified points that represents the waterline. The red points in Figure 5.6 on the right side are the reading points and the blue line is the fourth order interpolated polynomial. Interpolation makes it possible to determine the z-levels between the reading points.

![Graph of waterline](graph_waterline.png)

Figure 5.5 connected points of waterline.

![Graph of drain result](graph_drain_result.png)

Figure 5.6 Reading points of levelling.
Now, the multiple waterlines sampled at different z-level allows for the mapping of the beach and thus the bathymetry.

5.3.2 The rectification process

Co-ordinates of the pixels determined in the above described technique are transformed (=rectification). Images are two-dimensional whereas the real world is three dimensional. Rectification from image co-ordinates to real world co-ordinates requires some modification-techniques. First, the photogrammetric equations for transforming images and its derivation is described. The geometry and labelling conventions as used in Argus image rectification process are shown in Figure 5.7. The distinction between the focal plane and ground plane is expressed in the different letters. The focal plane represents the image and the co-ordinates on this plane are denoted with letters \((U,V)\), whereas the co-ordinates on the ground plane will be denoted with letters \((X,Y)\).

![Diagram of rectification process](image)

Figure 5.7 Geometry and labelling conventions used for photogrammetry

with:

- \(O\) : optic centre of the camera
- \(N = \text{Nadir}\) : projection point of \(O\) on the ground plane (the origin co-ordinate)
- \(Z_c\) : distance above the ground plane
- \(f_\gamma = \text{focal length}\) : distance between \(O\) and focal plane
- principal line : bisection line of the focal and ground plane respectively
- optical axis : axis through \(O\), \(p\) and \(P\)
- \(p = \text{principal point}\) : centre of the focal plane
- \(P\) : intersection of optical axis with the ground plane
\[ \tau = \text{camera tilt} \quad : \text{angle between the optical axis and the nadir line} \]

\[ q \text{ and } Q \quad : \text{location of a point in the focal plane and ground plane respectively} \]

Using the relations given in 5.1 and 5.2, ground co-ordinates \((X,Y)\) can be derived from the images co-ordinates \((U,V)\).

\[ X = \frac{Z_c}{\cos(\tau - \alpha)} \tan \gamma \]

(5.1)

\[ Y = Z_c \tan(\tau - \alpha) \]

(5.2)

where the angles \(\alpha\) and \(\gamma\) are defined as:

\[ \alpha = \arctan \left( \frac{V}{U} \right) \]

(5.3)

\[ \gamma = \arctan \left( \frac{U}{\sqrt{U^2 + V^2}} \right) \]

(5.4)

with:

\[ U_q \quad : \text{x-co-ordinate in the focal plane} \]

\[ V_q \quad : \text{y-co-ordinate in the focal plane} \]

In theory, these equations seem to give a good solution for the problem, however, several complications arise in applying these relationships in field. The problem is, it is hard to determine or estimate the camera tilt \(\tau\) and the focal length \(f_c\). One of the solution to solve the focal length problem is counting the pixels on the television screen. In doing so, the following expression is used:

\[ f'_c = \frac{x_e}{\tan \left( \frac{\delta}{2} \right)} \]

(5.5)

where \(f'_c\) the 'magnified' focal length and \(x_e\) is the measured distance from the principal point \(p\) to the right-hand edge of the enlarged image and \(\delta\) represents the field of view of the camera. Equation (5.5) introduces new parameter \(\delta\) that also hardly to measure in the field.

Another complication considers the definition of the co-ordinate system. Since the camera is positioned in such a way, giving the best point of view of the interested section, results in a co-ordinate system that does not corresponded with the more practical used co-ordinate system, where the x-axis is pointed in the direction cross the shore and the y-axis parallel with...
the shore. The angle of rotation between the two co-ordinate systems $\varphi$ and camera roll $\theta$, which is the angle between the camera relative to the horizon, are also again difficult to estimate accurately in the field.

The unknowns: $\tau$, $f_c$, $\varphi$ and $\theta$ must be solved, before the rectification process can start. Determination of these parameters can be made quite accurately by making use of targets at known locations in the image, the so-called Ground Control Points or the GCP's. By knowing both the ground and image co-ordinates of these control points, equation (5.1) and (5.2) can be solved iteratively. To solve all four unknown parameters, at least two GCP's are needed. The solution will be more accurately determined if there are more GCP's available.

### 5.4 Results

Figures C-1 to C-3 show the bathymetries of the groyne test obtained by image processing. On the first sight, the bathymetries are reasonably presented by the new waterline detection model. Considering these figures in more detail, the development determined by the model in the area adjacent to the waterline is questionable. The inaccuracy of bathymetry at the waterline can be explained as follows: the video camera and the illumination are fixed, whereas the waterline moves down the beach slope during the draining process. While this process progresses the waterline approaches closer to the video and the lights, which results in a good image. Murky brown effects far away from the illumination are the major causes of inaccuracy. The deviation can result in improper detection of the waterline. Figures C-6 to C-8 display the bathymetry for several days in 3-D form.

In addition, observation from photographs confirms that a little hole occurs at the tip of the groyne, although it is not large. The new waterline detection model unfortunately can not clearly visualise such holes. This is caused by the fact that 100 micron sand held water in any depressions for a long time, it prevented the video from capturing the details of these depressions. It is evident that the model is not capable of displaying the morphological changes around the structure.

The Figures C-4 and C-5 present two bathymetries of the offshore breakwater test. Apart from the inaccuracy of the waterline detection model due to depressions, it can be concluded that morphological changes are more visible than in the groyne test. This is intuitively correct, since parameters such as the pump discharge and wave height are increased. The detection model seems not to recognise the proper waterline behind the breakwater because of the shadow, subsequently the bed development is inaccurately determined. An alternative solution is to measure this area by traditional levelling method. The quantity of sand deposited here will be considered in more detail in Chapter 6. 3-D presentation of the bathymetry of 8 and 10 November can be found in Figures C-9 and C-10.

Subtracting the bathymetry of the several days with the initial profile, erosion and accretion areas are clearly shown. (see Figures C-15 to C-19)
5.5 Evaluation

The principle of this waterline detection model is based on the gray-intensity of the pixels. This implies that the model is very sensitive to color intensity. If a transect has two or more places where the gradient has the same magnitude, it results in more than one waterline. To avoid this undesired phenomenon the master-image has to be modified. In this image the highlighted objects (measuring instruments, detached breakwater or the groyne) in the region of interest are erased. Besides, other annoying effects are given below:

- the artificial light
A good illumination of the basin provides a good quality of an image. Sharp distinction of gray-intensity results in a more accurate determination of the waterline.

- foam on the water surface
The 100 Micron sand contained lots of minerals. Combination of wave action and dirty water produces white foams. Floating foam causes error in the identification of the waterline.

- movement of the plastic curtain
The black plastic curtain ensures that sunlight can not penetrate into the basin, but it also causes side effects. The movement of the plastic curtain during the draining process reflects the light from the water surface to the camera, which leads to a white spot on the image.

- erased spots of the image
Because of the fact that the erased spots do not contain the information of the bathymetry, interpolation is needed. This implies that uncertainties still exist at these locations. However, most of the problems could be improved. To avoid the daylight one can cover all the windows or leveling at night instead of during the day.

- depressions in the bathymetry
Using video to measure the bathymetry of a model that contains significant depressions is probably not effective. In our situation the 100 micron sand holds water in any depressions for a long time. This prevents the video from capturing the details of these depressions.
5.6 Conclusions

- The process of measuring the bathymetry by video taping while lowering the water level was workable, but did require a significant amount of time, during which the model could not be operated.

- By placing the camera on a position that covers the whole basin, more details of the bathymetry could be obtained.

- A proper illumination of the basin is essential with respect to the using of the new waterline detection technique.

- Removing all the foam from the water surface and reflecting objects before leveling the basin will increase the quality of the final results.

- Significant depressions in the bathymetry prevent the video from capturing the details, which causes an inaccuracy of the identified bathymetry.

- Applying this model to determine the bathymetry is laborious: it takes for instance a lot of time to remove all the reflections.

- The presence of spurious reflections considerably complicates the determination of the proper waterline. Due to the removing of spurious these effects of the master image interpolation is needed. When the considered area becomes larger, the interpolation of the bathymetry in this area becomes less reliable.
6 Comparison with numerical model

6.1 Introduction

In the former chapters a description of the physical model and analysis of the data were discussed. In this chapter, the analysed data of the physical model will be compared with computational results. Therefore, a numerical model is set up, this will be briefly described below.

Since this is a pilot-experiment, it is not intended to make a quantitative comparison with the numerical model, but rather as an indication of the usefulness of the physical model test. In order to make a comparison of collected data with computational results a comparison table is set up, in which only qualitative expressions are given. From this table one can quickly observe which aspects of the physical model show similarity with or differences the numerical model. In cases where it is possible, the components will be quantitatively expressed.

6.2 Numerical model: DELFT3D

6.2.1 Introduction

DELFT3D is a computer modelling framework for two-and three-dimensional flow and transport in coastal and river environments. The modelling framework consists of fully integrated modules. Different combinations of these modules allow the computation of water flow, water quality, sediment transport, ecological and chemical parameters, short wave generation and propagation, current-wave interaction and morphological simulation. In this study the morphological component is used. This MOR-module consists of the following processes: waves, flow, sediment transport and bottom changes. In the latest version, on-line sediment transport (version 03.05.007, Roelvink et al., 2000), the sediment transport and the bottom change are integrated within DELFT3D-FLOW module. This will be discussed in the next section.

6.2.2 DELFT3D-MOR

The morphological evolution of a coastal area concerns the fully coupled activities of waves, flows, sediment transport and bed level variation. These complex activities will be decomposed into separate processes by DELFT3D-MOR. The separated processes operate sequentially but are using each others results. The steering module, MORSYS, controls the order of in which these processes are activated and how data communication is organised. The simulated process is organised by MORSYS according to a user-defined process tree. By modifying in the MOR-input file, the executions of certain sub-processes can be manipulated.
6.2.3 Data communication

The coupling of the various modules requires a file in which relevant data used by the various modules can be stored. The communication file in DELFT3D is built up with a nefis structure (Natural File System). All the data relevant to the various modules will be written to this communication file.

6.3 The modules

It is not the intention to give an extensive description of various modules. Only the relevant modules will be briefly discussed here. This is done to give the reader some insight into the programs used for the various physical sub-processes. The input of different modules is prescribed by ASCII-files called md-files (master definition files). For more detailed information of numerical and physical backgrounds the reader is referred to the manuals.

6.3.1 The wave module

The wave-module consists of the HISWA and SWAN programs. In the computation only HISWA is used, which stands for Hindcast Shallow water WAVes. It is a numerical model for the prediction of short crested waves in shallow water under stationary conditions (Holthuijsen et al., 1989).

6.3.2 Updated DELFT3D-FLOW module

In the sediment version of DELFT3D-FLOW (v.03.05.007) three-dimensional sediment transport is implemented. This sediment version of DELFT3D-FLOW dynamically updates the elevation of the bed at each computational time-step. At each time-step, the FLOW module calculates the change in the mass of bottom sediment that has occurred as a result of the sediment sink and source terms. This change in mass is then translated into a change in thickness of the bottom sediment layer. This change in thickness is equivalent to a change in bed elevation. This is one of the distinct advantages of the sediment version of DELFT3D-FLOW as it means that the hydrodynamic flow calculations are continuously carried out using the correct bathymetry. The implementation of three dimensional suspended sediment transport in the hydrodynamic (FLOW ) module is described in more detail by Lesser (2000).

In the modified DELFT3D-FLOW module the calculation of bed-load sediment transport of ‘sand’ fractions is implemented. The implementation uses a three-step approach to calculate the spatial distribution of the bed-load transport, see Roelvink, et al. (2000). Activation of the bed-load transport option causes the bed-load transport vector to be calculated at each computational point at each time step.

When wave information is present on the communication file, this can be read and accounted for in FLOW. However, the computed flow field and bottom changes have an effect on the waves, which must be accounted for. MORSYS (v.04.02.003) allows the interaction between WAVE and FLOW and feedback to WAVE.
6.4 Input data

6.4.1 Wave input data

Input grid

The input grid covers the whole area of interest. The orientation of the various grids and their dimensions are shown in Figure 6.1. Note that the x- and y-axis are different from the physical model. There are 115 meshes in the longshore direction and 73 in the offshore direction corresponded with 240 and 38 meters in the x res. y-direction. The TRISULA-grid is chosen much larger than the actually interest area to ensure the accuracy. The size of the meshes reduces in the area where the groyne and the offshore breakwater are located. This in order to obtain more accurate computations.

Computational grid

The incident waves make an angle of 20° with the normal of the coastline. To avoid the disturbances on each lateral side of the boundary the computational grid must be larger than the input grid. With the next relation the enlargement of the computational grid can be determined.

\[ \tan(\phi) \leq \frac{\Delta x}{\Delta y} \]

where:
\[ \Delta y = 38 \text{m} \]
\[ \phi = 20^\circ \]
This implies that for \( \Delta x \) at least 14m must be chosen. The seaward boundary is taken approximately equal to the input grid. The mesh size in the x- and y-direction is chosen 1.0m respectively 0.2m. A nested grid, which has the same dimension as the physical model is placed in the interested area, however, the mesh size is reduced in both directions.

![Figure 6.1 Grids used in numerical model.](image)

WL | Delft Hydraulics
6.5 Input parameters

6.5.1 Wave height and wave period

For the computations in HISWA significant wave height $H_{sig}$ and the peak wave period $T_p$ should be given. These are:

$$H_s = \sqrt{2} \times H_{rms}$$

$$T_p = 2.0s$$

Two different wave heights will be applied in the computation. $H_{sig}=10cm$ is used in the model with the groyne, whereas the model with the offshore breakwater $H_{sig}=15cm$ is applied. The peak period for both model remains $T_p=2.0s$.

6.5.2 Flow input parameters

In the previous section the dimension of the grid is already described. All computations are executed in 3-dimensions. The depth is divided in 7 layers which varies in thickness. The log-distribution of the layers is given below:

<table>
<thead>
<tr>
<th>Layer</th>
<th>relative thickness (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (top)</td>
<td>26</td>
</tr>
<tr>
<td>2</td>
<td>24</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
</tr>
<tr>
<td>4</td>
<td>14</td>
</tr>
<tr>
<td>5</td>
<td>9</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>7 (bottom)</td>
<td>2</td>
</tr>
</tbody>
</table>

This results in the total number of cells that must calculated: $MNK_{max} = 115 \times 73 \times 7 = 58765$

Boundaries

Two different types of open boundaries are used: the up- and the downstream boundary are Riemann boundaries, which imply that the gradients of the water level and the velocity along the shoreline are zero. The sea boundary defined by the water elevation.

Bottom friction

The bottom friction formulation used here is the formulation according to Chézy.

$$C_{2D} = 18 \log_{10} \left( \frac{12h}{k_s} \right)$$

where:
\[ h = \text{total water depth} \]
\[ k_s = \text{Nikuradse roughness length} \]
With a water depth of 0.5m and $ks = 0.015$ m (height of ripple) the Chézy coefficient $C_{2D}$ is approximate $47 m^{1/3}/s$.

### 6.6 Results

#### 6.6.1 The groyne computation

The simulation runs for a period of 10 minutes to allow the hydrodynamic computations to stabilise before the updating of the bottom. After that it continues for another 12 minutes, using a morphological time-scale (MORFAC) of 100. This is equivalent to simulating the morphological developments taking place over 20 hours.

The Figure C-11 shows the morphological development after 20 morphological hours of simulation. A little scour hole of 3 cm has been developed at the head of the groyne. Due to the restriction caused by the groyne the flow is forced to accelerate and this is causing the erosion at the head of the groyne. Accretion and erosion can be easily displayed by subtracting the initial profile and the bathymetry after 20 hours of simulation. This is presented in Figure C-20. A sand bar of approximately 6 cm height is formed near the toe of the beach. Furthermore, it is interesting to note that a large erosion area of approximately 3 cm deep occurs adjacent to the groyne in the downstream area. The erosion seems in line with the expectation. Since the longshore transport is interrupted by the groyne, so there is no longshore sediment supply on the lee side of the groyne, consequently erosion occurs. In principle one can expect that the morphological changes in the adjacent areas of the groyne are mirrored however, this does not happen.

#### 6.6.2 The offshore breakwater computation

Computation with the offshore breakwater shows a more interesting results (see Figure C-13 and C-14). This is not surprising since a higher wave condition is applied in the simulation. Morphological changes at the front side as well as behind the breakwater are clearly visible. A scour hole at the tip of the breakwater has been developed. It is not major, probably because of a short simulation time. Accretion due to sedimentation of sand behind the breakwater seems in line with the expectation. An underwater tombolo has been formed. Furthermore, morphological changes in this simulation are significantly greater than in the case with the groyne. This satisfies the expectation since the physical parameters are increased. Figure C-21 shows the eroded and accreted area of the breakwater computation.
6.7 Comparison of numerical and physical model

6.7.1 Bathymetry: Groyne

Figure C-3 and C-11 show the final results of physical model and numerical respectively. In the physical model accretion at the upstream side of the groyne is visible, whereas numerical model hardly shows it. A good agreement has been found between numerical and the physical model concerning the erosion in the breaker zone (see Figures C-17 and C-20). A large erosion spot in the physical model near the upstream boundary can be explained by the fact that the incoming flow does not contain sediment initially. The concentration has to build up in this section, so the sand bed will erode in time. Furthermore, a major scour hole at the tip of the groyne is not found in both models.

Cross-sections

The cross-sections of the beach profile located in the upstream side of the structure at the position of the beam are shown in Figure D-1 to D-3. Erosion in the breaker zone of numerical computation is small compared with physical model. Manually measured points across the beach correctly confirm that the erosion near the inflow of the physical model has occurred. It can be noticed that these measurements are reasonable in line with the results of image processing. Figure D-4 shows the bed development in time of the numerical model. Initial waterline is situated at \( x = 3 \) meter.

6.7.2 Bathymetry: Offshore breakwater

Comparing the computational result with the merged images some similarities can be found. On the first sight, the morphological changes in both models seem in line with the expectation; erosion at the front and accretion at the lee side of the breakwater. Figure C-13 and C-14 present the bathymetry after 20 hours of simulation, whereas Figure C-5 and C-10 present the bathymetry from the video analysis merged with levelling results. Differences can be distinguished. It should be noticed that the form and the orientation of the underwater tombolo differs. The tombolo in the physical model seems not to be influenced by the oblique incident wave at all. Furthermore, the simulation shows an accretion of 11.5 cm just right behind the breakwater, whereas the physical model shows a much lower value (7.5cm). Erosion in both models at the downstream side of the breakwater due to the strong current velocity of combined gyre and longshore current is in line with the expectation.

Cross-section

The cross-sections in Figure D-5 and D-6 show a credible match between the merged images and the simulation in the area between \( x = 7 \) m and \( x = 14 \) m. Unfortunately, verifying this result with measurements can not be done because they are missing. We can also notice that the difference increases from \( x = 7 \) m to the waterline. Enormous erosion in this area may be caused by the fact that supplied sand in physical model was not equally spread across the beach: during the test we believed that the distribution of added sand was not appear to be necessary as the waves perform this function automatically. After the event it was obvious that equally distribution across the beach appeared to be very important.
Considering all cross-section figures in more detail, a great resemblance can be found: big difference between physical and computer model is found near the waterline.

### 6.7.3 Control area

The area (4.3m x 5.0m) behind the breakwater will be considered in more detail. The location and dimensions are given in Figure 6.2.

![Control area](image)

Figure 6.2 Control area.

According to the traditional levelling sand has been deposited right behind the breakwater. Furthermore, the waterline is also shifted in the offshore direction, the so called salient has been formed. Considering this area by subtracting the bathymetry with the initial profile it turned out that the control area has totally lost of 0.161 m³ of sand. This is somewhat counter intuitive, but can be easily explained. The severe erosion of the adjacent area results in a net loss of sand.

The same phenomenon can be found in the numerical simulation. Erosion also occurs in the adjacent area of the detached breakwater. Accretion right behind the breakwater is increasingly higher, 0.123 m³. The bottom developments of the control area after 20 hours in both models are shown in Figure E-1 and E-2.

When a smaller area is taken in considering (3.0 m x 5.0 m), it turns out that accretion according to the numerical model (0.198 m³) approximate ten times larger than physical model (0.018 m³). It can be concluded that the numerical model seems to overestimate the accretion behind the breakwater.

### 6.7.4 Longshore current

**Groyne**

Figure F-1 shows the computed longshore current profile and the experimental measurements. According to the measurement the peak of the longshore current profile is situated more in the offshore direction than of the computed profile. The magnitudes of the measured velocity are somewhat lower than the computed. This might be explained by the fact that not the proper bottom roughness is used in the simulation. At the tip of the groyne a lower velocity also has been found (see Figure F-2), but it is in line with the computed velocity.
Offshore breakwater

The longshore current profile across the beach presented in Figure F-3 shows that the distribution of both models are fairly matched. The peak of the maximum current is situated approximately at the same location in both models. However, it should be noticed that the computed values are larger than the measured values. Measurement behind the breakwater gives substantially lower value. (Figure F-4) According to the measurements and computational results the longshore currents are in line with respect to the magnitude. As we can clearly see in Figures F-4 and F-5 that the longshore velocity behind the breakwater is much higher than at the front, which is remarkable.

6.7.5 Concentration

The concentration profiles presented in Figures G-1 to G-6 are computed by DELFT3D-FLOW module with version 03.05.007. Comparing the results it can be said that the calculated values are considerably higher than the measurements. This might partly be explained as follows: it appeared that the sediment concentrations strongly depend on the orientation of the intake tubes. The sediment concentrations were substantially lower if the intake tubes were directed perpendicular to the current direction than when directed opposite the current direction.

It is interesting to note that the concentrations of the experimental measurements increase as approaching the waterline, whereas the computations shows the opposite. This only appears in case when Hs=15cm and Qp=187.5 l/s. This can be clearly seen in Figure G-7 to G-9.

6.7.6 Discharge

Groyne

Analysis of the applied and the measured discharge in the physical model is already done in Chapter 4. This section, the results produced by DELFT3D will be compared with the experimental results. Calculation of the numerical output for the total longshore discharge gives 154 l/s. This is approximate 29 l/s larger than the applied discharge in the physical model (125l/s). It is not surprisingly, since the measured longshore velocity is also lower than the computed velocity.

Offshore breakwater

In this case a significantly difference has been found between the physical and numerical value of the discharge. According to the computation the magnitude of the longshore discharge is 2.45 times larger than the pump discharge. We still do not have any idea yet how such an extremely high value exists between the physical and computer model. One of the explanation could be that the bottom roughness used in numerical model is not correct. In the numerical model the bottom roughness is assumed to be uniform, whereas in the physical model it is not.
6.7.7 Sediment transport

Groyne

As we have observed before in chapter 4, calculation of sediment transport using the measured longshore velocity and concentrations shows a remarkable low transport rate (42.5 g/s). Computational result produced by DELFT3D shows a higher value (134 g/s) than the measured quantity. Considering all uncertainties involved, it can be concluded that due to low concentration of the physical model results in a low transport rate.

Offshore breakwater

In this test we do not have the complete measurements of concentration across the beach. Only a rough estimation of the supplied sand at the upstream boundary is available. It appeared that the physical model required approximate 600 litre of sand per hour to remain stable. We have already seen in the previous section that the computational values of velocities and concentrations is substantial higher than the measurements. This also results in a much higher longshore transport rate (1600 l/h).

Total Longshore discharge and sediment transport of numerical model for the two wave conditions are presented in Figures H-1 and H-2.

6.7.8 Wave height

Groyne

Figure I-1 displays the computed and the measured wave heights in the groyne test. It can be said that the wave heights in both models show great similarity. Decreasing of the wave height in physical model is more gradually than the predicted by numerical model.

Offshore breakwater

This is also found in the test of the offshore breakwater. The waves outside the breaker zone sustain their height. In the breaker zone wave heights rapidly decrease because the lost of energy. The declining of the wave height in the breaker zone in physical model as well as in numerical model is similar (Figure I-2). A good agreement has also been found for the wave height in front of and behind the offshore breakwater (Figure I-3) and on the tripod downstream of the breakwater (Figure I-4).
A brief summary of initial comparison are given in table 6.1 and 6.2.

<table>
<thead>
<tr>
<th>subjects</th>
<th>physical</th>
<th>numerical</th>
</tr>
</thead>
<tbody>
<tr>
<td>bathymetry</td>
<td>irregular depth pattern</td>
<td>smooth depth pattern</td>
</tr>
<tr>
<td>accretion</td>
<td>upstream side groyne, beach toe</td>
<td>beach toe</td>
</tr>
<tr>
<td>erosion</td>
<td>adjacent beaches of groyne</td>
<td>at tip of groyne</td>
</tr>
<tr>
<td>scour hole</td>
<td>tip groyne, but minor</td>
<td>tip groyne, minor</td>
</tr>
<tr>
<td>wave height</td>
<td>in line with numerical model</td>
<td>in line with physical model</td>
</tr>
<tr>
<td>longshore current</td>
<td>low</td>
<td>high</td>
</tr>
<tr>
<td>concentration</td>
<td>low</td>
<td>high</td>
</tr>
<tr>
<td>longshore discharge</td>
<td>125 l/s</td>
<td>154 l/s</td>
</tr>
<tr>
<td>longshore transport</td>
<td>42.5 g/s</td>
<td>134 g/s</td>
</tr>
</tbody>
</table>

Table 6.1 Qualitatively and quantitatively intercomparison of physical and numerical model in case with the groyne

<table>
<thead>
<tr>
<th>subjects</th>
<th>physical</th>
<th>numerical</th>
</tr>
</thead>
<tbody>
<tr>
<td>bathymetry</td>
<td>irregular depth pattern</td>
<td>smooth depth pattern</td>
</tr>
<tr>
<td>accretion</td>
<td>beach toe, right behind breakwater</td>
<td>beach toe, behind breakwater</td>
</tr>
<tr>
<td>erosion</td>
<td>adjacent beaches of breakwater</td>
<td>at breakwater tip</td>
</tr>
<tr>
<td>control area</td>
<td>erosion 0.161m³</td>
<td>accretion 0.123m³</td>
</tr>
<tr>
<td>waterline</td>
<td>shifted in offshore direction</td>
<td>shifted in onshore direction</td>
</tr>
<tr>
<td>scour hole</td>
<td>large, but not deep at front breakwater</td>
<td>directly at breakwater tip and deep</td>
</tr>
<tr>
<td>wave height</td>
<td>in line with numerical model</td>
<td>in line with physical model</td>
</tr>
<tr>
<td>longshore current</td>
<td>low</td>
<td>high</td>
</tr>
<tr>
<td>longshore discharge</td>
<td>187.5 l/s</td>
<td>460 l/s</td>
</tr>
<tr>
<td>longshore transport</td>
<td>600 l/h</td>
<td>1600 l/h</td>
</tr>
</tbody>
</table>

Table 6.2 Qualitatively and quantitatively intercomparison of physical and numerical model in case with the offshore breakwater.
6.7.9 Computation with adapted depth

In the following computation another depth profile is used. This depth profile is equivalent to a bottom after four hours of simulation without structures. Computation with the adapted depth is more realistic than the former computation, because this is what happened in the physical model: the structures were built after couple of hours of wave action, which means that the initial bed level has been changed. The results of this computation will be discussed in the section below.

6.7.10 Results

In Figures F-1 and F-3 the results of longshore current are plotted for the computation with the new depth in case of 10 cm and 15 cm wave height. Figures G-1 to G-6 present the sediment concentrations. It can be said that these results are almost identical with the previous computations. Significant differences between the bathymetries of the groyne and the offshore breakwater with the previous simulation are not found.

Control area

Subtracting the final morphological development behind the offshore breakwater with the adapted depth profile of the control area (4.3mx5m) shows in this case a surprisingly good agreement. According to the simulation predicted by Delft3D 0.64m$^3$ of sand has been deposited in this area, which is in line with manual measurements (0.58m$^3$).

Conclusion

First comparison shows significant difference between the physical and numerical model, even when an adapted depth is applied in the computation. Only the wave heights and morphological development behind the offshore breakwater show an amazingly good agreement.
7 Conclusions and Recommendations

7.1 Conclusions

The pilot morphological model shows that despite of limited equipment and time, valuable results are obtained which proves that it is feasible to carry out such experiment in the near future.

Since the basin is not large it is important that the boundaries are designed in such a way that these can easily adapt to the changing beach profile, because if these boundaries are out of balance with the central section (interest area) the impact on the flow velocity is significant. The upstream boundary is extremely sensitive, especially when the construction has been built. After the re-adjustment of the upstream wave guide the flow condition throughout the model is significantly improved.

The model requires significant quantities of sand (0.6m³ per hour) to be supplied at the upstream boundary. This is not surprising, however the speed at which a failure in the supply of the correct quantity of sand, affects the flow rate in the model is surprisingly fast. An almost continuous supply of sand is required to keep the model running smoothly.

Sand can easily and effectively be supplied by simply tipping wheelbarrow loads of sand directly into the water near the upstream wave guide, with a little or no manually spreading of this sand. The waves and current should continue to run while the sand is dumped.

Using video to measure the bathymetry of a model that contained significant depressions would probably be ineffective. In our situation the 100 micron sand held water in any depressions for a long time. This prevented the video from capturing the details of these depressions. However, verification of image processing results with manual levelling gives qualitatively good agreement.

Initial comparison of the measurements with the computational results shows significant differences. In general, it can be concluded that numerical model produces much higher concentrations and thus longshore sediment transport rates than the physical model.
7.2 Recommendations

To execute such experiment successfully, it is recommended to plan more time in the execution. The model designer has to pay continuous attention on the test, especially at the beginning. Things can go terribly wrong when the modeller is not able to anticipate the situation.

With using a mobile carriage placed across the beach the bathymetry will be more accurately measured. This prevents footprints on the beach that might influence the bathymetry. Investigation of morphological changes, especially around the construction requires more advanced instruments, because manually measuring is not accurate enough. More velocity instruments are necessary in order to determine the longshore current and the return flow parallel to the beach correctly.

An efficient sediment supply system in the upstream boundary area across the beach allows the model to be easily controllable. The amount of dumped sand should be determined hourly. With a sophisticated sand trapping system the quantity of longshore sediment transport can easily be determined.

Placing the camera on a position that covers the whole basin could optimise the new waterline detection technique. Effective lighting arranged in more or less the manner originally envisaged (i.e. with the lights relatively low to the water surface, shining in the same direction as the camera view) prevents spurious reflections.

As quantitative results are not easily obtained from the video measuring technique we found it essential to manually measure at least one profile to obtain real-time information on the changes in beach profile. It is recommended that in any future tests a minimum of 3 profiles is manually measured, preferably using proper levelling equipment. The results of these measurements can be used for real-time interpretation of the functioning of the model as well as future calibration and validation of the video measuring technique.

The process of measuring the bathymetry by video taping while lowering the water level was workable, but did require a significant amount of time, during which the model could not be operated. We drained and refilled the model each day, with each complete process taking some 3 hours. Sophisticated drain and refill system could accelerate this process, which can save a lot of time.

Up till now the bottom roughness in numerical simulation is assumed to be uniform, whereas the wave-current related bottom roughness in the physical model locally varies (observed from physical model). It is recommended to re-run the simulations with varying bottom roughness.

In order to compare the results in proper way, we suggest to set up a numerical model, which is identical to the physical model: same conditions and dimensions. Additionally, using the adapted bottom profile as the input for the depth, morphological changes around the structures
References


Visser, P.J. (1982). The proper longshore current in a wave basin, Report no. 82-1, Communications on Hydraulics. Department of Civil Engineering Delft University of Technology.


A  New waterline detection technique

clear all
close all
cd d:\My_Documents\images\10-11;
blank=imread('set0001.bmp');
 fid = fopen('result.1011','w');

for n=1:63
  hulp=n+10000;
  t=((n-1)*40+1);
  hulp2=num2str(hulp)
  fname=strcat('set',hulp2(2:5),'.tif')
  if exist(fname)
    tifl=imread(fname);
    tifl(blank==255)=nan;
    h=figure(1);
    image(tifl);
    h3=double(rgb2gray(tifl));
    h3(300:end,:)=nan;
    b3=blank(:,3);
    h3(b3==255)=nan;
    h3(h3==0)=nan;
    h4 = zeros(size(h3));
    if n==1;
      h5 = zeros(size(h3));
    end
    h4(2:end-1,:)=.25*h3(1:end-2,:)+.5*h3(2:end-1,:)+.25*h3(3:end,:);
    h3=h4;
    a=figure(2);
    imagesc(h3);
    zoom;
    x=700;y=125;
    j1=y;
    dif=zeros(size(h3));
    dif(1:end-5,:)=h3(2:end-4,:)-h3(3:end-3,:); 
    [mxstep jmxstep]=max(dif);
    for i=x-1:61;
      pp(1:300)=0;
      in=1;
      j1=min(max(jmxstep(i),12),500);
      for j=j1-5:j1+5
        pp(j)=h3(j+1,i)-h3(j-1,i);
        if isnan(pp(j));
in=0;
end
end
if in;
  [mx(i),jmx(n,i)]=min(pp);
  h5(jmx(n,i),i)=7.1950e-010*t.^3 -3.3837e-006*t.^2 -8.992e-003*t +
               5.1872e+001;
  xx=i;
  yy=jmx(n,i);
  zz=(1.2410e-013*t.^4 - 7.1693e-010*t.^3 + 2.3585e-007*t.^2 - 1.1624e-
               002*t +
       5.2996e+001-2.996)/100;
  fprintf(fid,\%10.2f \%10.2f \%10.4fn',xx,yy,zz);
end
end
imagesc(h5);
title(zz)
f = getframe(a);
[X, map] = frame2im(f);
fname=strcat('set\',hulp2(2:5),'\',w,'tif')
imwrite(X,fname,'tif')
f=[];
end
end
fclose(fid);
close(figure(1));
figure;
info=load('result.1011');
x1=info(:,1);
y1=info(:,2);
plot(x1,y1,'r');
set(gca,'ydir','reverse')
r=0:60:3600;
poly=polynomial1011(r);
function z=polynomial1011(r)
z=(1.2410e-013*r.^4-7.1693e-010*r.^3+2.3585e-007*r.^2-1.1624e-002*r+5.2996e+001-
       2.996)/100;
figure;
plot(r,z,'b');
xlabel('time [s]');
ylabel('water level [m]');
title('10-11-2000');
axis([0 3000 0 0.55]);
grid on;
end
B Photographs
Photo B 1 Draining process of the groyne test

Photo B 2 Draining process of the offshore breakwater test
C  Bathymetries
Physical model with groyne
Image processing
Bathymetry 19 October; after 14 hours

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<th>Morph. pilot experiment</th>
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<td>DELFT HYDRAULICS</td>
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</table>
Physical model with groyne
Image processing
Bathymetry 20 October; after 20 hours

Delft3D
Morph. pilot experiment

WL | DELFT HYDRAULICS

Z2906
Fig. C 3
Physical model with offshore breakwater
Image processing
Bathymetry 8 November; after 8 hours

Delft3D
Morph. pilot experiment

WL | DELFT HYDRAULICS

Z2906 Fig. C 4
Figure C 8. 3-D Bathymetry 20 October
Figure C 10  3-D Bathymetry 10 November
Numerical model with groyne

Bathymetry; after 20 hours

Delft3D

Morph. pilot experiment

WL | DELFT HYDRAULICS

Z2906  Fig. C 11
Figure C 12. 3-D Bathymetry of numerical model; Groyne
Numerical model with offshore breakwater

Bathymetry after 20 hours

Delft3D
Morph. pilot—experiment

WL | DELFT HYDRAULICS

Z2906 Fig. C 13
Figure C 14  3-D Bathymetry of numerical model; Offshore breakwater
Physical model with groyne
Image processing 18 October; after 8 hours
Accretion Erosion [m]

Delft3D
Morp. pilot experiment

WL | DELFT HYDRAULICS

Z2906  Fig. C 15
Physical model with groyne
Image processing 19 October; after 14 hours
Accretion Erosion [m]

WL | DELFT HYDRAULICS

Delft3D
Morp. pilot experiment

Z2906
Fig. C 16
Physical model with groyne
Image processing 20 October; after 20 hours
Accretion Erosion [m]

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Physical model with offshore breakwater
Image processing 8 November; after 8 hours
Accretion Erosion [m]

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<td>Morp. pilot experiment</td>
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</table>

WL | DELFT HYDRAULICS

Z2906 | fig. C 18
Physical model with offshore breakwater
Image processing 10 November; after 20 hours
Accretion Erosion [m]

Delft3D
Morp. pilot experiment

WL | DELFT HYDRAULICS
Z2906 Fig. C 19
Numerical model with groyne

Accretion Erosion; after 20 hours

Delft3D

Morph. pilot experiment

WL | DELFT HYDRAULICS

Z2906 | Fig. C 20
Numerical model with offshore breakwater

Accretion-Erosion; after 20 hours

Delft3D

Morph. pilot experiment

WL | DELFT HYDRAULICS

Z2906  Fig. C 21
D  Cross sections
Figure D.1 Cross section at the beam; 18 October

Figure D.2 Cross section at the beam; 19 October

Figure D.3 Cross section at the beam; 20 October
Figure D.4 Cross section of the beach in groyne simulation

Figure D.5 Bed level measurements
Figure D 6 Cross section beach profile 8 November

Figure D 7 Cross section beach profile 10 November

Figure D 8 Cross section of the beach in Offshore breakwater simulation
Control area
Figure E.1 Control area of physical model

3D plot showing the control area with coordinates.
Figure E.2 Control area of numerical model
F  Longshore current
Figure F 1 Computed and measured longshore current at the beam; Hs=10cm

Figure F 2 Computed and measured longshore current at the tip of groyne; Hs=10cm
Longshore current in the upstream of the breakwater

Figure F 3 Computed and measured longshore current at the beam; $H_s=15\text{cm}$

Longshore current at the breakwater

Figure F 4 Computed and measured longshore current at the breakwater; $H_s=15\text{cm}$
G  Concentrations
Figure G.1 Computed and measured concentrations at position 1; Hs=10cm

Figure G.2 Computed and measured concentrations at position 2; Hs=10cm

Figure G.3 Computed and measured concentrations at position 3; Hs=10cm
Figure G 4 Computed and measured concentrations at position 4; Hs=10cm

Figure G 5 Computed and measured concentrations at position 5; Hs=10cm

Figure G 6 Computed and measured concentration at position 6; Hs=10cm
Figure G 7 Computed and measured concentration at position 3; Hs=15cm

Figure G 8 Computed and measured concentration at position 4; Hs=15cm

Figure G 9 Computed and measured concentration at position 5; Hs=15cm
H Longshore discharge and sediment transport
Figure H 1 Longshore discharge and sediment transport; $H_s=10\text{cm}$

Figure H 2 Longshore discharge and sediment transport; $H_s=15\text{cm}$
Figure H 3 Longshore current profile (linear extrapolated between x=9 and x=14m)

Figure H 4 Longshore discharge
I Wave heights
Figure 1.1 Computed and measured wave height at beam in case of 10cm wave height

Figure 1.2 Computed and measured wave height at beam in case of 15cm wave height
Figure I.3  Computed and measured wave height at breakwater

Figure I.4  Computed and measured wave height downstream of breakwater