The evolution of sandbars along the Colorado River downstream of the Glen Canyon Dam

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Cover photo: Andrew Pernick, Bureau of Reclamation
Aerial view of Glen Canyon Dam during high-flow release of water on March 5, 2008. All four jet tubes are open in the picture.
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Preface

This report contains the final results of my graduation research of the evolution of sandbars along the Colorado River downstream of the Glen Canyon Dam. This thesis concludes my Master of Science at the faculty of Civil Engineering and Geosciences at Delft University of Technology. This work was done in cooperation with Deltares.

Many people have contributed in different ways to the realization of this thesis. First I would like to thank my graduation committee, for the interesting discussions I had. I also thank Edwin Elias for introducing me to every one of the United States Geological Survey, USGS, in Santa Cruz and making me feel very welcome over there. Cor Zwanenburg of Deltares (Unit of Geo-Engineering) for helping me with the software package PLAXIS for simulating the erosion of the river banks.

Furthermore I would like to thank Deltares for the facilities they offered at the Rotterdamseweg and Stieltjesweg and the USGS for the facilities in Santa Cruz.

I thank my family who has supported me from the start. I want to thank my friends for their support and a wonderful time in Delft. However, most of all I want to thank Thijs for his interest, patience, support and help during my graduation.

Liz Kemp

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Abstract

The Colorado River in the Grand Canyon area has a lot of sandbars or beaches but they are eroding. Since the building of the Glen Canyon Dam and the Hoover Dam in the 1960s, sediment is trapped in the upstream lake. The inflow of water is clear, cold and nearly sediment free. The only noticeable remaining inflow of sediment in the Colorado River is by two tributaries which merge with the Colorado River. This sediment is mostly transported as bed load in downstream direction during normal flow conditions, and does not contribute to sandbar building.

The erosion of sandbars has been researched by the United States Geological Survey for decades. Large field experiments are part of this research. In 1996 the first high flow experiment was performed with the aim of creating new beaches or extending the existing ones along the river. Four large high-flow experiments have been executed in the river until now. The fourth one was the high-flow experiment of 2008. During the high-flow experiment of 2008 a dam release of 42000 cfs (1134 m$^3$/s) was bypassed through the dam for 60 hours. The experiment was planned for March 2008 since there was enough sediment in the system. The high velocities would bring sediment from the riverbed in suspension. The sediment is then deposited in low-velocity areas.

During the high-flow experiment of 2008 researchers surveyed multiple sites along the river. River mile 45 was one of the areas which was measured extensively. The data gained at that location are analyzed in this report. At this location the bed was surveyed twice a day using multibeam-soundings for the collection of the bathymetry data, flow velocity was measured, and samples of suspended sediment were taken. A camera was located in the Willie Taylor pool area capturing the sandbar in the Willie Taylor pool. This even continued for several months after the experiment.

The main question in this research is whether it is possible with a specified dam release, to create sandbars and how they can remain stable on the long term. To answer this question the software program Delft3D is used for the simulation of the sandbar creation. For the long term stability of the bars an image transformation was executed, and for the short term stability the geotechnical program PLAXIS was used.

The first part of the question is the creation of sandbars in the two consecutive pools, the Willie Taylor pool and the Eminence pool, located at river mile 45. The 3D flow fields in the pool areas are comparable to the measured flow fields. During the high discharge beaches were created in the sheltered areas. This was to some extent reproduced in Delft3D. Different hydrographs were used as inflow boundary conditions to analyze the influence of time on the deposited volume. A longer rise period has a positive effect on the deposited volume of sediment. In the first 20 hours of the peak most of the sediment is deposited. After the first 20 hours sediment is still deposited on the high levels but to a much lesser extent.

The second part of the question is whether the beaches can remain stable on the long term. Bank failure happens due to different forces. The erosion of the Willie Taylor bank was captured on camera. Every day photos were taken of the bar. With an image transformation the erosion rate was determined. The erosion rate is not a linear process: immediately after the flood the erosion process is very fast and it slows down over time.

When the peak of the high discharge was over, the water levels dropped 4 meters in 28 hours. This duration is too short for the bar to cope with the decrease in water level. The bar is not yet fully drained which has a negative influence on the stability of the bar. With PLAXIS a rapid
drawdown is simulated. When adjusting the drawdown rate to a slower pace the slope stability increases. When the water level is decreased 4 meters in 5 days the bars have enough time to drain and the slope stability is as high as possible. However, this also means that at other parts of the sandbar erosion processes (at high velocities) can continue for a longer time.

The dam release on an average day is coupled to the power usage upstream. The power usage has a daily fluctuating character. Therefore the dam release varies during the day by 0.5 meter. This fluctuation of the water level has a negative effect on slope stability. Different scenarios were developed to investigate the influence of a varying water level compared to a constant water level.

For the situation in the Willie Taylor and Eminence pools a dam release can be determined so sandbars will be created and remain stable for the long term. Sandbars can be created during a high discharge under sediment enriched conditions. The erosion of the sandbars can be set to a minimum when the drawdown of the water level takes 5 days, so the sandbars are almost fully drained. However, implications of erosion at other parts of the sandbar (other than the banks) should be considered using the Delft3D model. The daily fluctuations should be set to a minimum especially in the first period after the flood.
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1 Introduction

1.1 Background information

The Colorado River in the United States is one of the longer rivers in North America. The Colorado River has its source in a lake in the Rocky Mountains in northern Colorado and the river flows into the Gulf of California after having covered a distance of about 2330 km.

In this study focus is put on part of the Colorado River, the Grand Canyon area, Figure 1-1. In this area high flow experiments have been done for which the upstream boundary is Lake Powell which is created by the Glen Canyon Dam. Downstream of the Glen Canyon Dam the Paria River and the Little Colorado River merge with the Colorado River in Marble Canyon. The Colorado River flows into Lake Mead which is created by the Hoover Dam in the 1960s, this is the downstream boundary of the experiment. So the investigated area is restricted to the Marble and Grand Canyon reaches.

With the building of these dams sediment is trapped in the upstream lakes. As a result not enough sediment is transported into the Marble Canyon and Grand Canyon reaches which results in erosion of the existing sand bars.

The United States Geological Survey, USGS, is researching the erosion of the sandbars in the Grand Canyon. The USGS have been looking into this erosion process for years. The Colorado River is monitored closely and since 1996 four experimental high flow releases were imposed (1996, 2000, 2004, 2008). In March 2008 the last high flow experiment was done and a lot of measurements were executed, among which flow and bed topography of two pool areas near river mile 45 were measured with much detail. The measurements of these two consecutive pools, the Willie Taylor and the Eminence pool, are the foundation of this thesis.

Figure 1-1: Grand Canyon area, after (Kempthorne and Myers, 2007)
1.2 Problem description

The sediment transport into the Grand Canyon decreased dramatically after the completion of the Glen Canyon Dam in the 1960s. Before this, the spring snow melt had always swelled the river thus contributing to the river’s natural flood cycle. Large quantities of sand were transported which created and maintained the Grand Canyon sandbars. Before the dam was built, 25 million tons of sand entered the Grand Canyon annually. The Paria River added 1.7 million tons of sand. Downstream of the Marble Canyon the Little Colorado River and the Colorado River merge, adding a further 1.9 million tons. The total sand supply in the predam era was almost 29 million tons per year. Now the Paria River and the Little Colorado River are the main sand suppliers of the Grand Canyon. Other smaller tributaries also contribute a minor amount of sand to the Grand Canyon. Nowadays around 16% of the predam sand supply is provided to the Canyon (Wright et al., 2005). In Figure 1-2 this difference is depicted, the magnitudes of the arrows indicate the amount of sediment supply before (A) and after (B) the dams were built.

![Figure 1-2: There is sediment re-supply form the Paria and Little Colorado Rivers, after (Topping et al., 2000)](image)

The USGS has been researching the erosion process for years and one of the main causes they found is that the erosion of the sandbars in the canyon is primarily caused by the lack of suspended sediment. The water which enters the canyon at the Glen Canyon Dam is cold, clear and sediment free. The discharge at the dam has a daily fluctuation cycle as it is directly related to power usage upstream. This is thus a contributing factor to the erosion. The daily changes can be as large as 4500 cfs, roughly 120 m³/s, which is a water level change of around 0.5 meter a day. In the summer

![Figure 1-3: Mean discharge for the year 2009, (http://waterdata.usgs.gov)](image)
and in the winter the demand for power is higher than during the other seasons. In Figure 1-3 the mean discharge is plotted over the year 2009 in which the seasonal difference is clearly visible.

Creation of the sandbars is only possible with a high discharge and a rich availability of sediments. If no measures are taken all sandbars which are in the river will erode. When enough sediment has entered the Colorado River from the Paria and Little Colorado River a high discharge is bypassed at the Glen Canyon Dam so sediment will be in suspension and deposited in low velocity areas. This cannot be done too often as sediment enriched conditions are needed to create the sandbars. If a high discharge is released when there is not enough sediment in the system, erosion of the sandbars will be the result.

In the Grand Canyon these sediment deposition areas have been identified as an important environment for recreational, ecological and archeological resources. The sandbars are important for the rafters who can set camp on these beaches. The sandbars provide habitat for wildlife and fish, and the sand supply is needed for protection of archaeological sites.

To deal with the erosion problem the Glen Canyon Dam Adaptive Management Program GCDAMP was developed to “maintain or improve the quality of recreational experiences for users of the Colorado River Ecosystem, within the framework of the GCDAMP ecosystem goals” (2001). The following management objectives to maintain or improve recreational resources were developed:

1. Maintain or improve the quality and range of recreational opportunities in Glen and Colorado Canyons within the capacity of the Colorado ecosystem.
2. Increase the size, quality and distribution of camping beaches in critical and non-critical reaches.
3. Maintain or enhance the wilderness experience in the Colorado River ecosystem (Wright et al., 2005).

The program excludes constructive measures, and augmentation of sediment supply from bypassing sediment from Lake Powell.

1.3 Objectives

The erosion of the sandbars in the Colorado River has been a problem for many years now. Many initiatives have been taken to investigate this loss of sandbars along the river. With a high flow release sandbar volume can be increased but after a couple of months the high flow deposits are mostly eroded, so the following question arises:

*Is it possible to design a dam release such that sandbars will accrete and remain stable on the long term?*

1.4 Approach

To answer this question the focus is on the creation of the bars during the flood wave and on the minimization of erosion of the bars right after the high flow. To simulate and predict the details of the bar building process the high flow experiment has been simulated with Delft3D. This is a numerical modeling system for hydrodynamic and morphological simulations. The modeling approach is able to reproduce the complex eddy patterns from which the sand bars originate. Two consecutive pools located at river mile 45 were extensively monitored and analyzed. Therefore the modeling is executed with the data gained from these pools measured during the high flow experiment of 2008. The results of these simulations have been compared to the measured data from the high flow experiment and used to calibrate and validate the model to some extent. Different scenarios have been developed to find the best dam release for creating stable beaches.
At the end of the peak of the high flow experiment water levels dropped and newly deposited beaches appeared at certain locations. One of the main problems was that after the flood, these new beaches eroded fast due to geotechnical instability processes. The bank stability under rapid drawdown conditions have been simulated with the Finite Element Model PLAXIS. This program is a geotechnical program which can calculate the slope stability under rapid drawdown conditions. However, PLAXIS does not provide the actual erosion rate. To determine the erosion rate empirically we have used an approach based on digital photography. Time lapse pictures of the Willie Taylor pool were taken on site by the USGS. Those photos have been transformed to a horizontal reference to gain more insight into the erosion rate of the sandbar.

A two month visit to the USGS in Santa Cruz, California was planned. The aim was to gain more data of the high flow experiment and to learn from the experts who have been researching this erosion problem for many years.

1.5 Contents of the report

In chapter 2 the high flow experiments are described. The experiment of 2008 is specified in more detail. The creation of the sandbars is modeled using Delft3D and described in chapter 3. In chapter 4 the bank failure is described; first an image transformation is carried out to the time lapse photos of the Willie Taylor pool; and second the stability calculation of the slope of the newly created beaches using PLAXIS is presented. In chapter 5 the conclusions and recommendations are specified. This thesis also includes 5 appendices.
2 High flow experiment

In the last few decades a couple of major high flow experiments have been executed in the Colorado River, and studied by the United States Geological Survey, USGS. The goal of these experiments was to find out how to increase the size and quality of the beaches in the critical and the non-critical reaches. The researchers aim at gaining insight into the physical processes which take place in the river. A critical reach was defined as a continuous river section in which the number of available campsites is limited, because of geological characteristics, high demand, or other logistical factors. Non-critical reaches are sections of the river in which plentiful sandbars exist, (Gloss et al., 2005).

Sandbars particularly develop during floods because the strongest eddies occur during such high discharges. Plans have been made to create beaches by creating artificial floods on a more regular basis. The strongest eddies are particularly located along the banks just downstream of rapids formed by debris flows or fans from small tributaries. Behind various rapids along the river recirculation eddies may develop, in these sheltered areas sediment can settle since the velocity is lower with respect to the main channel, resulting in a lower transport capacity of sediment. After a high discharge, water levels drop again and the areas where sediment has settled emerge along the edges of the river as beaches and sandbars at various locations. In these eddies some sediment is trapped, such that sandbars can develop. However, in some time during floods some of the previous sediment is eroded from parts of the bars, and flushes out the Grand Canyon. Therefore only foods that occur under sediment enriched conditions can result in a positive mass balance and a net transfer of sediment from the channel to the eddies. When there is not enough sediment in the system the high discharge will transport sand from the existing beaches to the main channel.

Sediment mainly enters the Colorado River through the Little Colorado River and the Paria River. This is fine sediment with a $D_{50}$ of 0.1 mm, which settles in the main channel during normal low discharge conditions, or is transported downstream as bed load transport. Since the sediment does not accumulate on the riverbed, there is generally insufficient sediment available to be transferred from the river bed to sandbars. When there is enough sediment throughout the system a high discharge will cause sediment to be in suspension as is shown in Figure 2-1. All fine sediment which is located on the river bed at a low discharge, is in suspension at a high discharge.

Figure 2-1: Sand on the river bed will be suspended by the controlled flood and deposited in sand bars along the banks, after (Anderson et al., 1996).
2.1 Previous high flow experiments
The experimental release for the 1996 flood consisted of a low steady flow for 96 hours, then a rapid increase in 10 hours to a steady peak flow which continued for 167 hours. The falling limb took 46 hours and resulted in a steady state of 88 hours, see Figure 2-2. The 1996 experiment resulted in an overall reduction of sandbars. The high water flow continued too long which resulted in a lot of sediment ending up in Lake Mead.

![Figure 2-2: Left: Hydrograph for the 1996 flow event, after (Wiele, 1996), Right: Hydrograph for the 2000 flow event, after (Hazel et al., 2001)](image)

In 2000 an experiment was executed with a controlled Low Summer Steady Flows (LSSF) of 8,000 cubic feet per second (cfs) downriver from the Glen Canyon Dam, from March through June 2000, with a four-day spike of 30,000 cfs (940 m$^3$/s) in May. The LSSF was designed to test the benefits of low flows on native fishes, the spike was intended to improve aquatic habitat by rebuilding sandbars.

From the 1996 and 2000 events, scientists learned that tributary-supplied sand does not accumulate on the riverbed over long periods of time under typical dam conditions. Typical dam conditions are releases which depend on the power usage upstream and have a fluctuating character. So after the 1996 and 2000 event, high flow releases are only planned, when a lot of tributary floods occurred, supplying new sand in the Colorado River, (Melis et al., 2007).

The next experiment was executed in 2004. The hydrograph was similar to the one used in 2008, which is depicted in Figure 2-6. The duration of this high flow was a lot shorter than the 1996 event. In 2004 the experiment led to an increase in the total sandbar area in the upper half of Marble Canyon and Grand Canyon. However the beaches eroded quickly. One of the causes of this erosion was that after the high flood the agency which operates the dam continued to vary the volume of water being let through the dam on a daily basis to adjust power generation to match peak power consumption. The variable discharge had a bad impact on the survival of the beaches.

2.2 High flow experiment of 2008
The experiment in 2008 is different from the previous ones. One of the main lessons learned from the previous experiments was that more sediment was needed for the creation of the sandbars. Just before the 2008 experiment about three times the volume was available compared to the available volume in 2004. Demonstrable sand enrichment conditions prior to the flood were present in all reaches except between river-miles 61 and 88, Figure 2-3.
Before March 2008 there were 16 months of tributary inflows below the Glen Canyon Dam which created sediment enriched conditions. This was the main reason that the high flow experiment was planned for March 2008. For 60 hours a constant high flow was released at the Glen Canyon Dam. The water volume was designed to force sand and silt from the riverbed onto the eroding beaches and sandbars.

Data was collected at multiple sites along the river (e.g. at river miles 30, 45, 60 and 87). For this study, data has been used that was collected from river mile 45. This area contains two consecutive pools connected by a small rapid. In Figure 2-4 the model area is depicted. The left image shows the location of the study site which is around mile 45 in Marble Canyon. On the right the modeled area is depicted with the two consecutive pools, the Eminence pool and the Willie Taylor pool.
Before the high discharge

River mile 2 taken on March 3, 2008

River mile 6 taken on February 3, 2008

River mile 45 taken on March 4, 2008

River mile 65 taken on February 10, 2008

River mile 81 taken on February 5, 2008

After the high discharge

River mile 2 taken on March 28, 2008

River mile 6 taken on March 29, 2008

River mile 45 taken on March 14, 2008

River mile 65 taken on April 2, 2008

River mile 81 taken on April 6, 2008

Figure 2-5: Pictures before (left) and after (right) from the flow experiment 2008, (courtesy of Melis)
In Figure 2-5 a few images are plotted of sandbars in the river before and after the high flow experiment of 2008. On the left side the pictures are plotted that were taken before the high flow experiment and on the right side the pictures that were taken after the high flow experiment are plotted. At some upstream locations (river mile 2 and 6), loss of beaches had occurred. Around mile 45 and further downstream the beaches became either larger or no excessive volume changes occurred.

The reason why loss of beaches occurred at the upstream reach of the river is primarily the lack of extra inflow of sediment during the high discharge. For the upstream reaches there is not enough sediment present for the duration of the high discharge. At the start of the experiment beaches are created, but when there is no more inflow of sediment the beaches start to erode. However since these beaches have eroded during the flood, however they supply sediment for the downstream pool areas. Preliminary results of the 2008 experiment show that sandbars eroded in the upper most part of the Marble Canyon, but that there was a lot of deposition in parts of the lower Marble Canyon and eastern Grand Canyon.

Measurements were done by scientists and surveyors. Amongst other locations the Willie Taylor and the Eminence pool areas were measured extensively. Twice a day, before, during or after the flood, different cross sections were surveyed with different devices. The velocity was measured with an Acoustic Doppler Current Profiler, ADCP. This is an acoustic instrument which was attached to a small boat. Multiple cross sections were surveyed in the Willie Taylor and Eminence pool, to measure the flow velocity at different locations in the pool. Multibeam surveys were done twice a day for collection of the bathymetry data. Between the Eminence and Willie Taylor pool areas samples of the sediment at different depths were collected every day, using a traditional point sampler. With these samples a grain size analysis of the suspended sediment could be done in the lab after the high flow experiment.

![Figure 2-6: Discharge and sand concentration at mile 45 of the high flow experiment of 2008, (Sloff et al., 2009)](image-url)
The hydrograph and the sand concentration are plotted in Figure 2-6, the continuous line represents the discharge and the discontinuous line is the sand concentration. The symbols indicate the bathymetric survey times. The rising limb is 36 hours, the constant peak flow is 60 hours, and the falling limb is 28 hours. The increase in discharge causes a water elevation of 4 meters. The concentrations measured during the experiment are plotted in the same figure, the highest concentrations were measured during the start of the peak. After this the concentration of sediment decreases and is almost zero when the water discharge is at its normal level again. The high concentration of the suspended sediment measured is connected to the slumping which took place in the Eminence pool just before. The slumping is visible on the bathymetry data of the Eminence pool. In Appendix A, a table is depicted with the concentration of the suspended sediment for different grain sizes.

In Figure 2-8 the topography is plotted as derived by Kaplinsky (2008) from the multibeam sounding. The USGS references the water depth to the 8000 cfs discharge which is the water depth of 836.2 meter in the plots for the Willie Taylor and Eminence pool. The discharge of 8000 cfs is the reference discharge level, used by the USGS to compare wet and dry of the topography. The depth in the figures is referenced to the North American Datum of 1983 (NAD83) in meters. In the sheltered area of the Willie Taylor pool a sand bank is formed during the experiment. The return channel is clearly visible in the Eminence pool. In Appendix A, subsequent topography plots of the Eminence and Willie Taylor pool are depicted.
In Figure 2-9, the eddies are visible which were present during the high flow experiment. The results plotted here are from the experiment of 2008, derived from the ADCP data by Scott Wright. The plots show the depth averaged velocity vectors at the flood peak.

Figure 2-9: Flow field of the Eminence (left) and Willie Taylor (right) pool, (courtesy of Wright)

After the high flow experiment scientists visited the newly created beaches in May 2008. They dug trenches in the beaches to collect sediment samples, analyzed the bed layering, and studied the bed forms. The gradations of the sediment in the newly created beaches coincide with the sediment samples measured during the flood. In Appendix A the measured sediment in time for different diameters is plotted as is the fall velocity for each gradation. In Appendix D a table is plotted with the cumulative grain size distribution of the deposited sediment in the Willie Taylor pool.
3 Creation of sandbars

Sandbars are created by recirculation eddies below channel constrictions. The processes that contribute to the net erosion and deposition pattern in these eddies are of a complex 3D nature. Physically based computational models must therefore be 2DH or 3D with high resolution treatment of the processes. Previous work of (Wiele et al., 2007) have shown that such modeling approaches are capable of reproducing the time-dependent behavior of the sandbars during experimental flood. Presently (2009) USGS and Deltares are co-operating to apply the Delft3D modeling system for this purpose. Preliminary application of Delft3D showed that the observed sandbar development could be reproduced to some extent. Therefore we have chosen to explore this approach somewhat further and use is to assess the effect of the hydrograph and sand layer on the bar development.

The data gained from the high flow experiment of 2008 have previously been modeled in a preliminary Delft3D modeling study by Kees Sloff. The results of these preliminary runs are described in (Sloff et al., 2009). The Delft3D model created by Kees Sloff is the basis of the simulations in this research. The modeling results are explained and are followed by the presentation of the results which are compared to the measured data from the high flow experiment. This chapter concludes with various scenarios of the discharge for the optimal bar creation.

3.1 Modeling introduction

The basic equations and relevant features of the Delft3D modeling system are explained in Appendix B. For the simulation of the sandbars the Delft3D morphodynamic model is used, the model area is depicted in Figure 3-1.

The boundary conditions are determined for the upstream and the downstream side of the model area. The boundary conditions at the inflow boundary consist of the hydrograph, Figure 3-1, which was similar to the hydrograph of the experiment, and the sediment inflow which is coupled to the hydrograph. The inflow concentration used in the model is about 50% of what was measured during the experiment. This reduction gives a better match to the bar volume. The measurements of suspended sediment were done between the two eddies, so the data does not have to be representative of the upstream boundary.

Figure 3-1: Model area with grid, and discharge scheme – inflow boundary condition
The median grain size of the bed material was determined on multiple locations in the pool areas. With a camera with a macro lens pictures were taken of the bed layer. The median diameter was determined from those images. The median diameter varies from 0.2 – 0.5 mm, the average diameter found in the Eminence pool and Willie Taylor pool was 0.230 mm. The average diameter was used for the gradation of the initial bed layer. The gradation of the bed layer was determined on the gradation of the suspended sediment, since only the \( d_{50} \) value was known, (courtesy of Rubin).

The thickness of the bed layer was determined using the lowest measured depth ever from the pool areas and the bed level just before the high flow. Subtracting those layers resulted in a layer with various thicknesses. In the rapid areas no sand was available and in the pool areas a layer of sediment was present before the high flow.

The results of the Delft3D simulations are presented in different sections. First the hydrodynamic results are presented and compared with the measured data. Next the morphology results are presented. The morphology of the pools is compared in bed level changes and in the volume changes in the pool areas.

### 3.2 Hydrodynamic results

The considered part of the Colorado River, the Marble and Grand Canyon, is a steep reach, characterized by a sequence of riffles and pools. The riffles are formed by debris fans that are produced by small tributaries and partially block the river channel. Outflows from these tributaries are mostly by debris flows, which carry mostly very coarse sediment. The debris flow causes constrictions in the river with rapids as a result. Behind these rapids a recirculation zone occurs and depending on the local situation an area where sediment can settle may be created, Figure 3-2.

![Image](image_url)

**Figure 3-2:** Left: schematization of a flow field in the Colorado River, after (Schmidt and Graf, 1990), Right: flow pattern in eddy pool, (courtesy of Sloff)

The right picture of Figure 3-2 is of the bar in the Eminence pool after the high flow of 2008. At the location where the bar is, the primary eddy was located during the experiment. Since the flow velocity was lower in the primary eddy deposition of sediment took place at that location.

In Figure 3-3 the velocity profiles are plotted. The first plot is derived from the ADCP measurements. The second plot represents the flow field reproduced with Delft3D. The measured flow velocity in the main channel has the order of 3 m/s. In the Delft3D simulation flow velocities of 3 m/s are calculated for the main channel. The velocity on top of the bar is in both situations about 0.5 m/s and increases in the return channel. The flow velocity of the experiment and of the simulation is in the same order of magnitude.
3.3 Morphological results

In the Colorado River at certain locations sandbars were created. In the Willie Taylor pool and the Eminence pool sediment was deposited during the experiment. Delft3D reproduced the formation of the sand bars in both the Eminence and Willie Taylor pool. In Figure 3-4 the bed levels are depicted, on the left are the plots from Delft3D and on the right the bed levels measured during the experiment is plotted. Since the model has a relatively coarse grid the volume change in the pool will be compared to the measured volumes referenced to the depth.

There is a problem with the sigma grid when there are steep slopes in the topography. The computation of false gradients occurs when the horizontal diffusivity and horizontal pressure gradient forces are calculated. Near the transition area of the main channel and the shallow area,
the horizontal component of the sediment transport exceeds the vertical transport component. As a result the sediment is lifted upon the slope introducing an artificial vertical diffusion, (Stelling and van Kester, 1994).

In Figure 3-5 is visible that steep slopes occur in this Delft3D model. The first two images represent the change in bed level, the third image the location of the cross section. The right image presents a steep slope from the main channel to the sandbar. The slumping process which should take place does not occur. The erosion process is not complete in the model. For this reason most of the comparisons are done with respect to the deposition volumes in the pools.

### 3.3.1 Sand displacement

Another way to assess the outcomes of the morphological simulations is to consider the deposition and erosion volumes as function of elevation for the total pool area. In the following subsections, figures are presented in which the depth is plotted versus the volume of deposited sand per pool. These plots are divided in segments of time. The ‘rise’ coincides with the start of the flood until the high water level is reached. ‘Peak’ is the time period when the water is at the constant high level of 42,000 cfs (1134 m$^3$/s) for 60 hours. The ‘fall’ is the period in which the water level decreases until the normal water level has been reached. The entire flood summary is the sum of the rise, peak and fall. For every analysis the results are plotted with those four steps. In the left corner of each figure the volume of sedimentation and erosion for that time period is depicted. The elevation is plotted relative to 8000 cfs (216 m$^3$/s), which is the average discharge. So all volumes deposited above the zero level are part of a beach after the water levels have dropped again.

The area of the Eminence and the Willie Taylor pool used in the figures to display the volume changes, is the area between the rapids, so the surface of the rapids is not included in the plots. The volume changes in the pool area of the Delft3D simulations are compared to the figures created by S. Wright.
Sand displacement for Willie Taylor pool

| Figure 3-6: Deposition and erosion for the Willie Taylor pool gained from the experiment, the values in the corner are the sum of the volume over the depth |
|---|---|
| Rising limb summary | Peak summary |
| Volume change (m$^3$) in 25 cm bins | Volume change (m$^3$) in 25 cm bins |
| Elevation (m) relative to 8,000 cfs | Elevation (m) relative to 8,000 cfs |
| -16824.4478 | -7890.2797 |
| -1000 | -1000 |
| -500 | -500 |
| 0 | 0 |
| 500 | 500 |
| 1000 | 1000 |
| 1500 | 1500 |
| 2000 | 2000 |

| Figure 3-7: Deposition and erosion for the Willie Taylor pool from the Delft3D run |
|---|---|
| Falling limb summary | Entire flood summary |
| Volume change (m$^3$) in 25 cm bins |
| Elevation (m) relative to 8,000 cfs |
| -6356.3218 | -31071.0493 |
| -1500 | -1000 |
| -1000 | -500 |
| -500 | 0 |
| 0 | 500 |
| 500 | 1000 |
| 1000 | 1500 |
| 1500 | 2000 |

In Figure 3-6 and Figure 3-7 the deposition and erosion is plotted for the Willie Taylor pool. The first set of graphs represents the volume change during the experiment, in the second set of graphs the volume changes for the simulation with Delft3D are plotted. There are quite some similarities between these plots. The volume change during rise is clearly present. The volumes are on the sedimentation point of view during the rise and peak the same order of magnitude. Delft3D presents a large volume of sediment between the levels -5 and -10 meter, this is deposited in the shallow area right beside the main channel. In the experiment the deposition of sediment was more at the 0 till -5 range, while the -5 to -10 range was characterised by a net erosion.
3.3.2 Sand displacement for Eminence pool

Figure 3-8: Deposition and erosion for the Eminence pool during the flow experiment of 2008, the values in the corner are the sum of the volume over the depth

Figure 3-9:Deposition and erosion for the Eminence pool from the Delft3D run

In Figure 3-8 and Figure 3-9 the volume changes for the Eminence pool are plotted. Comparing the results of the depositions in the Eminence pool between the measured data and the simulations of Delft3D there appear to be similarities. The depth where the sediment is deposited is quite similar, however in the Delft3D model nothing happens below the -10 level. The volume of the deposited sediment is less in the simulation compared to the experiment.
3.3.3 Sedimentation and erosion during peak flow of the Willie Taylor pool

Peak 1: Mar-6 1700 to Mar-7 1200; 19 hours

Peak 2: Mar-7 1200 to Mar-7 1600; 4 hours

Peak 3: Mar-7 1600 to Mar-8 1100; 19 hours

Peak 4: Mar-8 1100 to Mar-8 1700; 6 hours

Figure 3-10: Sedimentation and erosion during the peak of the experiment, the values in the corner are the sum of the volume over the depth

Figure 3-11: Deposition and erosion results from Delft 3D for the peak flow

Since multibeams were done a couple of times a day the peak period can be split into four sections to see when most sedimentation and erosion takes place. These plots are depicted in Figure 3-10 for the Willie Taylor pool. In the first 19 hours most of the sedimentation takes place above the 8000 cfs level. These volumes are compared with the volumes deposited and eroded in the Delft3D simulation as can be seen in Figure 3-11. In this simulation there is also a peak of
deposited volume in the first 19 hours of the peak flow. After this part of the peak substantially less sediment is deposited or has eroded.

### 3.4 Different scenarios for a dam releases

Four different scenarios are developed to create beaches in the Delft3D model. The volume of deposited sediment is investigated and the rate of deposition during a certain period of time is examined. The various hydrographs are plotted in Figure 3-12, the first hydrograph plotted is the hydrograph used in the high flow experiment of 2008. The different scenarios are evaluated on the deposition rate and deposited volume. These results will be compared to the simulation of the experiment of 2008. The four scenarios are as follows:

1. Hydrograph (1) has the same rise as the reference run, but a shorter peak duration, the peak has a duration of 19 hours, which coincides with the duration in which most of the sediment is deposited during the experiment.
2. Hydrograph (2) has the same peak duration as hydrograph (1), the rise however has a longer duration as the time of the peak has shifted 20 hours so the effect of the deposition during the rise can be compared. The fall ends at the same time, however the duration is a little shorter compared to (1).
3. Hydrograph (3) has no real peak duration, the rise starts at the same time but the duration is 60 hours longer before the 42000 cfs discharge is reached and at that moment the fall sets in.
4. Hydrograph (4) has no real rise or fall, just a peak with the same duration as the reference run. So the effect of the rise prior to a 60 hour peak can be investigated.

![Figure 3-12: Various hydrographs](image-url)
Figure 3-13: Deposition and erosion pattern of hydrograph (1)

Figure 3-14: Deposition and erosion pattern of hydrograph (2)
Figure 3-15: Deposition and erosion pattern of hydrograph (3)

Figure 3-16: Deposition and erosion pattern of hydrograph (4)
In Figure 3-13, Figure 3-14, Figure 3-15 and Figure 3-16 the volume changes in depth are plotted for all four scenarios. The summary of the deposited volumes is presented in Table 3-1. For the hydrographs (1), (2) and (3) the start and end time is the same as the start and end time of the hydrograph of 2008. Hydrograph (4) has a shorter duration.

Table 3-1: Summary of erosion and sedimentation for different discharge scenarios

<table>
<thead>
<tr>
<th></th>
<th>Rise ER (m$^3$)</th>
<th>Rise SED (m$^3$)</th>
<th>Peak ER (m$^3$)</th>
<th>Peak SED (m$^3$)</th>
<th>Fall ER (m$^3$)</th>
<th>Fall SED (m$^3$)</th>
<th>NET (m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrograph 2008</td>
<td>-16824</td>
<td>18729</td>
<td>-17953</td>
<td>5106</td>
<td>-31071</td>
<td>41788</td>
<td>10717</td>
</tr>
<tr>
<td>Hydrograph (1)</td>
<td>-16777</td>
<td>18679</td>
<td>-8929</td>
<td>12339</td>
<td>-32296</td>
<td>39947</td>
<td>7651</td>
</tr>
<tr>
<td>Hydrograph (2)</td>
<td>-19756</td>
<td>23637</td>
<td>-5853</td>
<td>11064</td>
<td>-34322</td>
<td>44317</td>
<td>9995</td>
</tr>
<tr>
<td>Hydrograph (3)</td>
<td>-19107</td>
<td>25877</td>
<td>-380</td>
<td>6974</td>
<td>-26208</td>
<td>33343</td>
<td>7135</td>
</tr>
<tr>
<td>Hydrograph (4)</td>
<td>-4327</td>
<td>4834</td>
<td>-1255</td>
<td>20164</td>
<td>-16568</td>
<td>25100</td>
<td>8532</td>
</tr>
</tbody>
</table>

In Table 3-2 the volumes are printed for the deposited sediment in the Willie Taylor pool on the bar. This coincides with the positive elevation relative to 8000 cfs discharge plotted in the Figures 4-11, 4-12, 4-13 and 4-14. The increase of bar volume above the 8000 cfs level is the main purpose of the large dam release, since depositions at this level result in beaches under normal dam conditions. The deposited volumes for the different scenarios are compared to the reference run of the experiment of 2008. During the rise of hydrograph (2) and (3) more sediment is deposited compared to the reference run. During the peak period almost half of the deposited volume is reached by scenario (1) and (2) whilst the peak duration is only a third of the reference run. The deposited volume on the bar of the Willie Taylor pool is largest with the hydrograph of the reference run, followed by scenarios (2) and (4).

Table 3-2: Deposited volume on the bar in the Willie Taylor pool above the 8000 cfs level

<table>
<thead>
<tr>
<th></th>
<th>Rise</th>
<th>Peak</th>
<th>Rise + Peak</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m$^3$</td>
<td>%</td>
<td>m$^3$</td>
</tr>
<tr>
<td>Hydrograph 2008</td>
<td>2383</td>
<td>100</td>
<td>4339</td>
</tr>
<tr>
<td>Hydrograph (1)</td>
<td>2383</td>
<td>100</td>
<td>2202</td>
</tr>
<tr>
<td>Hydrograph (2)</td>
<td>2690</td>
<td>113</td>
<td>1939</td>
</tr>
<tr>
<td>Hydrograph (3)</td>
<td>3838</td>
<td>161</td>
<td>112</td>
</tr>
<tr>
<td>Hydrograph (4)</td>
<td>592</td>
<td>25</td>
<td>4127</td>
</tr>
</tbody>
</table>

In Table 3-3 the duration of the rise and peak period are printed. The rise and peak period are compared to the volume deposited in the given time span with a deposition rate in m$^3$/h. For the rise and peak duration the highest deposition rates are found when these periods are shortest.

What can be concluded from the deposition rates is that the deposition rate during the peak period is larger than during the period of rise, even when a peak period has a duration of 60 hours, or the period of rise should be really short which was the case in scenario (4).

Table 3-3: Comparison of deposited volume in duration of the rise or peak period

<table>
<thead>
<tr>
<th></th>
<th>Rise</th>
<th>Peak</th>
<th>Rise + Peak</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Duration (h)</td>
<td>Deposition rate (m$^3$/h)</td>
<td>Duration (h)</td>
</tr>
<tr>
<td>Hydrograph 2008</td>
<td>36</td>
<td>66</td>
<td>60</td>
</tr>
<tr>
<td>Hydrograph (1)</td>
<td>36</td>
<td>66</td>
<td>20</td>
</tr>
<tr>
<td>Hydrograph (2)</td>
<td>48</td>
<td>56</td>
<td>20</td>
</tr>
<tr>
<td>Hydrograph (3)</td>
<td>96</td>
<td>40</td>
<td>1</td>
</tr>
<tr>
<td>Hydrograph (4)</td>
<td>6</td>
<td>99</td>
<td>60</td>
</tr>
</tbody>
</table>
3.5 Optimizing the hydrograph for deposition

A hydrograph consists of a rise, a peak and a fall period. The bars are created during the rise and peak period of the hydrograph. When extending the rise period, more sediment is deposited above the 8,000 cfs level. However the deposition rate decreases as the period of rise of peak increases.

The peak is the important aspect of the hydrograph, since the highest deposition rates over a longer period of time are reached. The peak duration may not be too long otherwise all sediment will be ending up in Lake Mead. The peak may not be too short as a wave damps out and the high water levels will not reach the downstream area of the Colorado River.

The balance has to be found between the rise and peak duration. However this model is used for a very small part of the river so more investigation is needed to know whether these demands are valid Colorado River wide.

It is realized that chosen model approach is not an exact reproduction of the reality. As indicated by (Sloff et al., 2009) the model is very sensitive to initial sand layer and inflow concentration. Nevertheless it is believed that the tendencies shown here one reasonably representative for the actual events.
4 Failure of sandbars

During and after a flood, sandbars undergo erosion by various processes. In the previous section it has been shown that the growth of the sandbar is coming from a balance between deposition and erosion of different parts of the bar due to flow processes. The associated type of erosion has been reproduced with the Delft3D model. However, another important process for erosion of the sandbar is bank erosion induced by slope failure near the water surface. Particularly the decay of beaches after the flood is driven by this failure mechanism. In this chapter we have assessed this erosion process by analyzing observed bank retreat, and by modeling slope stability as function of different water level drawdown scenarios.

4.1 Introduction

Bank erosion can be caused by instability in an unprotected bank. Sandbars were created rapidly during the high flow experiment, but as the water levels dropped the newly formed beaches eroded fast. Photographic material illustrates this erosion process quite clearly as is shown in Figure 4-1 and Figure 4-2. These photos were taken right after the high flow of 2008. Vertical shear surfaces were visible along the newly gained beaches and after a couple of months much of these deposits have eroded. The erosion process of these beaches is comparable to bank erosion of unprotected banks in rivers.

Bank erosion in the Colorado River is mainly caused by currents (steady and unsteady) and differences in water levels. As the water level changes, it affects the internal and external forces on the slope. This results in both seepage-induced pore pressures due to transient flow and stress-induced excess pore pressures which are developed inside the slope.

Figure 4-1: Drainage of banks, (courtesy of Sloff)

Figure 4-2: Erosion of banks after the flood in the Colorado River, (courtesy of Draut)
The dissipation of the excess pore pressures and the decrease in seepage induced pore pressures depend on the drawdown rate and on the soil properties like permeability and compressibility. In highly permeable soils the pore pressures will dissipate immediately during drawdown, whereas in low permeable soils the pore pressures are not likely to dissipate at the same rate as the external water level changes. This results in totally or partially undrained soil behavior, which decreases the slope stability and may eventually lead to slope failure.

When the internal water level is not able to keep up with lowering of the external water level, we call this a rapid drawdown. This situation is shown in Figure 4-3 c. The other figures, a and b, show situations in which the soil is fully drained and partially drained during drawdown respectively. It is worth noticing that even a bank in a very permeable soil may fail if the drawdown is so fast that drainage cannot keep up with it.

There is a big difference between the failure mechanisms of non-cohesive and cohesive materials because of the significant differences in soil mechanics. In a non-cohesive bank, like a sand bank, shear strength increases more rapidly with depth than shear stress. This results in critical conditions more likely occurring at shallow depths. The shear strength describes the maximum strength of the soil at which significant plastic deformation occurs due to the applied shear stress. Erosion of cohesion-less soils depends on gravitational forces and particle characteristics such as size, grain shape, gradation and relative density. In a cohesive bank, like a clay bank, shear stress increases more rapidly than shear strength with increasing depth so that the critical surfaces are likely to be found within the deep layers of the bank. In the Colorado River, non-cohesive banks were created consisting entirely of sand.

After the flood in the Colorado River, the water level dropped very fast: 4 meters in 28 hours. The bars in the Colorado River are created during the high flow below the water surface in a very short period of time; they are completely saturated and not really consolidated. For the stability of the bars it is important to lower the water level not too fast so a pressure imbalance will be prevented. The stability of the banks can be given a safety factor (SF), the larger the safety factor the larger the stability:

\[
SF < 1 \quad \text{unstable}
\]

\[
SF \geq 1 \quad \text{stable}
\]

With respect to the Colorado River, the life span of the bars will be longer when a slow drawdown is executed compared to a rapid drawdown. Then there is more time to adjust to the external water level, which has a positive effect on the stability. This will be discussed more extensively in section 4.5.
4.2 Tracking bank movement by image transformation

The time dependent erosion process of the beaches after March 2008 was captured on camera. Six photos a day were taken of the Willie Taylor pool on previously programmed times. With these successive pictures the retreat of the bar is clearly visible, and it shows that the emerged part of the bar is almost completely gone after a couple of months. The camera was located in such a way that it overlooked the whole Willie Taylor pool as is shown in Figure 4-4. The main direction of the flow in the figures is indicated with the arrows. Using the coordinates in UTM from the left photo and the pixel coordinates from the photo on the right an image transformation can be done.

Figure 4-4: Left: world plane in Arizona State Plane coordinates; Right: image plane

The goal of the image transformation is to transform the time lapse photos of the Willie Taylor pool into the same coordinate system as the world plane. To start with, we have two planes, an ‘image’ plane and a ‘world’ plane. The ‘image’ plane has four coordinates \((x_0, y_0), \ldots, (x_3, y_3)\), the four corners of the first quadrilateral, and the world plane has the four coordinates \((X_0, Y_0), \ldots, (X_3, Y_3)\), which are the corners of the second quadrilateral. These four sets of coordinates for both planes are coordinates of the same visible landmarks. With these sets of coordinates the computation to transform the image into world plane coordinates can be started (Criminisi et al., 1997).

The coordinate system in which all pictures are plotted is the Arizona State Plane central zone grid (FIPS zone 0202) in meters, referenced to the North American Datum of 1983 (NAD 83). In the pictures taken of the bar a couple of clear landmarks are visible, the Northing and Easting in UTM are found for these points using the Grand Canyon Monitoring and Research Centre website. With a conversion tool the coordinates were transferred to Arizona State Plane coordinates. The pixel coordinates of the image are found using Matlab.

In Appendix C the transformation is explained in more detail. The transformation matrix at the end of the calculation is:

\[
T = \begin{pmatrix}
0.887 & 351.9914 & 9176.7158 \\
2.6461 & 961.5565 & 24165.8816 \\
0.0001 & 0.0395 & 1
\end{pmatrix}
\] (4.2)

Figure 4-5: Perspective transformation
The transformation of six consecutive photos is depicted in Figure 4-6, the same transformation matrix is used for each picture. Between each photo a time of around 25 days has passed. When comparing the transformed images to the ‘world’ plane it may seem as if the river is narrower in the transformed case, nevertheless this is vegetation which is in front of the camera. The camera is looking down on the bar, thus not having a complete view of the width of the river. The top of the image is somewhat stretched: this has to do with the transformation: all points outside the quadrilateral will not be correctly transformed.

Erosion of bar in time, T start is 10 March 2008

![Figure 4-6: Results from the image transformation of the Willie Taylor Pool area](image)

The retreating edge for the different time steps of the bar is shown in Figure 4-7. It is clearly visible that the erosion of this bar is not a linear process: immediately after the flood the erosion process is very fast and it slows down over time.
The surface in $m^2$ between the edge lines (shown in the figure above) is calculated and plotted in Figure 4-8. The data points present the sum of the calculated erosion in days [$m^2$/day]. The erosion process can be described with the following formula, and is also plotted in Figure 4-8.

$$E = \frac{at}{b + t} = \frac{4120r}{85 + t}$$

(4.3)

In this equation, $E$ is the erosion rate in $m^2$/day and $t$ is the time in days. This equation is only valid for this particular pool and this particular situation, since many other factors have an effect on the erosion process of a bar.

Note that this erosion rate is based on the surface area. Clearly from Figure 4-7 follows that also the shape of the beach is changing in time.

![Figure 4-8: Cumulative erosion of the Willie Taylor pool, t=0 is 10th of March 2008](image)
4.3 Topography changes

Figure 4-9: Photos of the Willie Taylor pool at the time when the surveys were done

On the 10th of March a survey was done just after the high flow. On the 31st of March the next survey was done and as can be seen in Figure 4-9 the bar in the Willie Taylor pool is eroding. The left image was taken on the 10th of March and the right image was taken on the 31st of March. Subtracting the measured surveys it presents loss of sediment for the period of 20 days in the Willie Taylor pool. When looking at the difference plot of these two surveys in Figure 4-10, the most extended part of the bar eroded, and the main channel deepened minimal. There are a few locations where sediment settled, but this does not equal the erosion which took place by far. Thus sediment is transported downstream during these 20 days. After the experiment a period of relatively high discharges were released at the dam, since there was a lot of rainfall upstream. However there were no enriched conditions after the flood, resulting in erosion when high discharges were released.

Figure 4-10: Cumulative erosion or deposition in meters of the topography between the 10th and the 31st of March 2008

4.4 Modeling tools for the stability analysis

The rapid drawdown was one of the criteria to determine which model could be used to simulate the stability problem. The following software tools were examined for the stability analysis of the sandbar during and after drawdown conditions.

CONservational Channel Evolution and Pollutant Transport System, CONCEPTS, is a computer model that simulates open-channel hydraulics, sediment transport and channel morphology.
CONCEPTS can determine whether a slope is stable or unstable but only under gradually varying water level changes. So it is not useful for the stability of the slope under rapid drawdown conditions, (Langendoen, 2000).

The Bank Stability and Toe Erosion Model, BSTEM, combines three limit equilibrium method models that calculate the factor of safety for multilayer stream banks. It has two main features, the failure by shearing of a soil block and the erosion by flow of bank and bank toe material. However the rapid drawdown cannot be simulated since the water table cannot vary in time, (Simon et al., 2008).

GEO-SLOPE is an international commercial design software tool for geotechnical solutions in a wide spectrum. With SLOPE/W which is part of the GEO-SLOPE software package, a stability analysis can be done during rapid drawdown conditions. However the student version is a limited version without the rapid drawdown tool. Nevertheless stability calculations with fully drained situations can be done, (GEO-SLOPE international, 1995-2010).

PLAXIS is a commercial Finite Element Method which can calculate the stability of slopes during rapid drawdown conditions, so this program is used for the stability analysis. PLAXIS is discussed in the following section.

4.5 PLAXIS

The stability problem is investigated with the 2D plane-strain models from PLAXIS. PLAXFLOW is one of the PLAXIS products. It is a stand-alone program for groundwater flow calculations. This program includes transient flows, unsaturated behavior and time-dependent boundary conditions. The PLAXFLOW results can be used in PLAXIS for the deformation and stability analysis.

4.5.1 The model

The PLAXFLOW program is used for the transient seepage analysis. The transient seepage analysis obtains seepage-induced pore pressures and a free ground water surface for different drawdown rates. PLAXIS is used for the deformation and stability analysis, in which drained soil behavior is modeled as a fully saturated and two-phased continuous medium. Stress induced pore pressures and the stresses and strains are calculated (Berilgen, 2007).

Elements

PLAXIS Version 8 can be used to carry out two-dimensional finite element analyses. Two dimensional models may be either plane strain or axisymmetric. Separate PLAXIS programs are available for 3D analyses but they are not used for this stability calculation. The plane strain model is used for the bank failure analysis of geometries with a (more or less) uniform cross section and corresponding stress state and loading scheme over a certain length perpendicular to the cross section (z direction). Displacement and strains in z direction are assumed to be zero. However normal stresses in z-direction are fully taken into account. It is also possible to use an axisymmetric model which is used for circular structures with a (more or less) uniform radial cross section and loading scheme around the central axis where the deformation and stress state are assumed to be identical in any radial direction.
Transient seepage analysis

In the simulation of the drawdown behavior of the slope the transient seepage analysis is done to obtain seepage-induced pore pressure and free groundwater surface. To model variation of the water level during drawdown a water level is determined and a time step in which this water level decreases has to be selected. The calculated groundwater flow parameters from the last time step are used in the deformation analysis. These groundwater flow calculations are executed with PLAXFLOW.

Gravity acceleration

The earth gravity acceleration, \( g \), is set to 9.8 m/s\(^2\) by default and the direction of gravity coincides with the negative \( y \)-axis. Gravity is implicitly included in the unit weights given by the user. In this way, the gravity is controlled by the total load multiplier for weights of materials. The weight is calculated in PLAXIS after the transient seepage analysis calculation.

Deformation analysis

The deformation analysis is performed using PLAXIS, in which drained soil behavior is modeled as a fully saturated and two-phased continuous medium and stress induced pore pressures and stresses and strains are calculated. The calculated groundwater flow parameters in the transient seepage analysis and the weight calculation are the input variables for the deformation analysis.

\( \Phi \)-\( c \)-reduction

The \( \Phi \)-\( c \)-reduction is an option available in PLAXIS to compute safety factors. In the \( \Phi \)-\( c \) reduction approach the strength parameters \( \tan \phi \) and \( c \) of the soil are successively reduced until failure of the structure occurs. The sliding surface does not need to be defined beforehand it is automatically found, therefore, a shear surface closer to the natural sliding surface is defined.

A more accurate definition of the factor of safety is therefore the ratio of the true strength to the computed minimum strength required for equilibrium. This deformation analysis is based on the Mohr Coulomb material model and the safety factor is:

\[
SF = \frac{c_{\text{input}} - \sigma_n \tan \phi_{\text{input}}}{c_{\text{reduced}} - \sigma_n \tan \phi_{\text{reduced}}} 
\]

Where the strength parameters with the subscript ‘input’ refer to the properties entered in the material sets and parameters with the subscript ‘reduced’ refer to the reduced values used in the analysis, \( \sigma_n \) is the actual normal stress component. \( \Sigma Msf \) is set to 1.0 at the start of the calculation to set all material strengths to their original values. In this approach the cohesion and the tangent of the friction angle are reduced in the same proportion:

\[
\frac{\tan \phi_{\text{input}}}{\tan \phi_{\text{reduced}}} = \frac{c_{\text{input}}}{c_{\text{reduced}}} = \Sigma Msf 
\]
The reduction of the strength parameters is controlled by the total multiplier $\Sigma M_{sf}$. This parameter is increased in a step-by-step procedure and, as a consequence, the strength parameters $\phi$ and $c$ of the soil are successively reduced until failure of the soil body occurs. The safety factor is then defined as the value of $\Sigma M_{sf}$ at failure, provided that at failure a more or less constant value is obtained for a number of successive load steps.

$$SF = \frac{\text{Strength available}}{\text{Strength at failure}} = \text{value of } \Sigma M_{sf} \text{ at failure}$$ \hspace{1cm} (4.6)

The safety factor calculation by using PLAXIS with Phi-c reduction procedure can be seen in more detail in (PLAXIS 2D, 2008).

### 4.5.2 Input parameters

In the following section the input parameters are discussed which were used for the bank stability simulations. There are a number of parameters unknown which are used in the PLAXIS calculation. For these values correlations are used to determine the unknown parameters with the known parameters of the sediment. These correlation tables and figures have been taken from (Koster, 2004) and have been compiled in Appendix D.

**Mohr Coulomb failure mechanism**

The Mohr Coulomb failure mechanism is a well-known model and is used as a first approximation of soil behavior in general. The Mohr Coulomb model is a first order model, and is used for the bank in the Willie Taylor pool. The model uses five parameters: Young’s modulus, $E$, Poisson’s ratio, $\nu$, the cohesion, $c$, the friction angle, $\phi$, and the dilatancy angle $\psi$.

In 1900 Mohr presented a theory for failure of materials. He stated that material fails because of a critical combination of normal stress and shearing stress, not just from either maximum normal or shear stress alone. The Mohr Coulomb failure criterion is described as follows:

$$\tau = c + \sigma \tan \phi$$ \hspace{1cm} (4.7)

In which $\tau$ is the critical shear stress, $c$ is the cohesion, $\sigma$ is the normal stress and $\phi$ is the internal friction.

The stresses in a material can be shown in a graph in which the shear stress and the normal stress are presented. The circle of Mohr intersects the horizontal axis (normal stress) at the smallest and largest possible normal stresses. When the Mohr Coulomb failure criterion intersects the circle of Mohr the soil will fail due to internal tension.

**Angle of repose**

The angle of internal friction is an engineering property of granular materials. It is the maximum angle of a stable slope determined by friction, cohesion and the shapes of the particles. The internal friction is usually determined for dry sands with a triaxial compression test. In this case the soil is completely saturated, and loosely packed. In the Appendix D the table is plotted where the angle of internal friction is taken from, which is $\phi = 30^\circ$. An angle of repose of 30° is an average value, there are however some circumstances in which this angle should be lower. When silt is present in the soil, the angle of repose decreases. When the percentage of silt is around 10 to 20 percent a reduction of 5° is done with respect to the angle of repose, (Koster, 2004). For the calculations $\phi = 30^\circ$ and $\phi = 25^\circ$ are used.
**Cohesion**

Cohesion is the component of shear strength of a rock or soil that is independent of the interparticle friction. The cohesion is almost zero for non-cohesive sediments. The value used in the PLAXIS calculations is \( c = 0.1 \).

**Poisson’s ratio**

The Poisson’s effect \((\nu)\) is the ratio when a sample object is stretched in one direction, it tends to contract in the other two directions perpendicular to the direction of stretch. And the other way round, when a sample of material is compressed in one direction it tends to expand in the other two directions. This phenomenon is called the Poisson effect see Figure 4-13.

Most materials range between 0 and 0.5 when compressed along the axial direction. For the stability calculations in PLAXIS \( \nu = 0.33 \) is used which is the default value. The table is plotted in Appendix D.

\[
\nu = -\frac{\varepsilon_{\text{trans}}}{\varepsilon_{\text{axial}}} = -\frac{\varepsilon_x}{\varepsilon_y} \tag{4.8}
\]

where
- \( \nu \) resulting Poisson’s ratio [-]
- \( \varepsilon_{\text{trans}} \) transverse strain [-]
- \( \varepsilon_{\text{axial}} \) axial strain [-]

**Young’s modulus**

Young’s modulus describes tensile elasticity, or the tendency of an object to deform along an axis when opposing forces are applied along the axis; it is defined as the ratio of tensile stress to tensile strain. Young’s modulus is the ratio of stress, which has units of pressure, to strain, which is dimensionless, therefore Young’s modulus itself has the unit of pressure.

\[
E \equiv \frac{\text{tensile stress}}{\text{tensile strain}} = \frac{\sigma}{\varepsilon} = \frac{F/A_0}{\Delta L/L_0} = \frac{FL_0}{A_0 \Delta L} \tag{4.9}
\]

where
- \( E \) Young’s modulus [kN/m²]
- \( F \) Force applied to object [kN]
- \( A_0 \) original cross-section area through which the force is applied [m²]
- \( \Delta L \) the amount by which the length of the object changes [m]
- \( L_0 \) original length of object [m]

Not all sand parameters are known, a correlation is used with the following formula to determine Young’s modulus:

\[
K = \frac{\nu}{1-\nu} = \frac{0.33}{1-0.33} = 0.5 \tag{4.10}
\]

\[
E = (1 - 2K\nu)E_y = (1 - 2 \cdot 0.5 \cdot 0.33)10 = 6.7 \text{ MPa} \approx 6700 \text{ kN/m}^2 \tag{4.11}
\]

**Drained or undrained behavior**

The drained behavior setting is used when no excess pore pressures are generated, this is the case for dry soils and for soils where drainage is possible due to a high permeability. For drained behavior the Mohr Coulomb failure criterion is valid.
The undrained behavior is used when pore pressures do develop, for instance in clay and peat soils. The permeability is low for these soils. However materials which usually have a drained behavior can also behave as undrained in situations when for instance the pore pressure can not dissipate fast enough. This is the case for these PLAXIS calculations.

**Soil weight**

The saturated and unsaturated weight refers to the weight of the soil including the fluid or air in the pores. The dry soil weight is the weight of the soil without water in the pores. The saturated weight is the weight of the wet soil below phreatic level without air in the pores. The weights are activated by means of the gravity loading in the calculation program, $\Sigma M_{\text{weight}}$. The saturated soil weight is determined from a table for loosely packed sands. The saturated weight is $\gamma_{\text{sat}} = 19 \text{ kN/m}^2$. The dry weight is taken from a correlation between completely saturated sand and the weight of this wet soil, this results in $\gamma_{\text{dry}} = 15 \text{ kN/m}^2$.

**Horizontal and vertical permeability**

The permeability is a measure of the ability of water passing through the soil. A low permeability implies that water passes very slowly through the pores of the medium. Sand usually has a permeability of a couple of meters a day. Since the permeability is unknown the following formula gives a value based on the sediment diameters. The sieve table of sediment diameters is plotted in Appendix D. In PLAXIS the horizontal and vertical permeability, $k_x$ and $k_y$ are expressed in m/day. The diameters in the following equation are in centimeters.

\[
k = \left( \frac{268}{D_{60}/D_{10} + 3.4} \right) (D_{10})^2
\]

\[
= \left( \frac{268}{0.0125/0.0053 + 3.4} \right) (0.0053)^2 = 0.00285 \text{ cm/s} \rightarrow 2.5 \text{ m/day}
\]

**The dilatancy angle**

Dilatancy is when compacted granular material expands in volume as it is sheared. The dilatancy angle of sand depends on the friction angle and the density for sands, the order of magnitude used in PLAXIS is $\psi = \varphi - 30^\circ$. When the friction angle is less, the dilatancy angle is zero. In this case the angle of repose is never larger than $30^\circ$ so the dilatancy angle is 0 for all simulations.

In the following table the input values are presented which were used in PLAXIS:

**Table 4-1: The input parameters for the run in PLAXIS**

<table>
<thead>
<tr>
<th>Material model</th>
<th>Mohr Coulomb</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of behavior</td>
<td>Drained</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material properties</th>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil unit weight above p.l.</td>
<td>$\gamma_{\text{unsat}}$</td>
<td>15</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>Soil unit weight below p.l.</td>
<td>$\gamma_{\text{sat}}$</td>
<td>19</td>
<td>kN/m$^3$</td>
</tr>
<tr>
<td>Horizontal permeability</td>
<td>$k_x$</td>
<td>2.5</td>
<td>m/day</td>
</tr>
<tr>
<td>Vertical permeability</td>
<td>$k_y$</td>
<td>2.5</td>
<td>m/day</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>$E_{\text{ref}}$</td>
<td>6700</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>$\nu$</td>
<td>0.33</td>
<td>-</td>
</tr>
<tr>
<td>Cohesion</td>
<td>$c_{\text{ref}}$</td>
<td>0.1</td>
<td>kN/m$^2$</td>
</tr>
<tr>
<td>Friction angle</td>
<td>$\varphi$</td>
<td>30</td>
<td>$^\circ$</td>
</tr>
<tr>
<td>Dilatancy angle</td>
<td>$\psi$</td>
<td>0</td>
<td>$^\circ$</td>
</tr>
</tbody>
</table>

p.l. = phreatic level
4.5.3 Results

From the measured data in March 2008 a cross section is taken which is the initial profile of the run in PLAXIS. The following cross section, Figure 4-14, is from the Willie Taylor pool at the location where the bank was formed. The cross section is from March 8th at the end of the day just before recession of the controlled flood takes place. This is the thin blue line in the figure. The largest slope in this cross section is 1:5. At the start of the simulation the water level is at the top of the bank. On top of the cross section from March 8th the cross section is drawn which is used in PLAXIS.

Figure 4-14: Cross section used in PLAXIS

From the Environmental Impact Statement (EIS) of 1995, (U.S. Department of the Interior, 1995), it is said that sandbars are initially deposited at angles ranging from 20 to 45 degrees with an average of 26 degrees. As the water level recedes, this slope may be unstable. Seepage induced erosion tends to reduce the slope to 11 degrees. On top of some sandbars, a rapid decrease in river stage sets up conditions for bar failure, this is depicted in Figure 4-15.

The slopes mentioned in the EIS are steeper than the ones measured on the 8th of March which is used for the calculations. The steepest slope measured with the multibeam is a slope of 1:5 which is 11 degrees and thus coincides with the ‘stable’ situation mentioned in the EIS report. For this reason more simulations are done with steeper slopes. The stability analysis is concentrated at the start of the receding limb of the flood and ends when the normal stage is reached.

Figure 4-15: Conceptual cross section of a sandbar affected by fluctuating flows, (U.S. Department of the Interior, 1995)
In the following section the results will be discussed of the PLAXIS runs. Different scenarios have been investigated. The slope of the bank is simulated in 3 different scenarios, a slope of 1:5, 1:4 and 1:3. The angle of repose has the following angles: 25° and 30°. All scenarios are done in the three drawdown cases which are the water level decrease of 4 meters in 28 hours which is the same drawdown rate as in the experiment of 2008, the water level decrease of 4 meters in 2*28 hours and the water level decrease of 4 meters in a fully drained situation.

In Table 4-2 the variations for the simulations in PLAXIS are described. The safety factor for each of the runs, 18 in total, is depicted in Figure 4-20. The run with a slope of 1:5, an angle of repose of 30° and the drawdown rate of 4 meters in 28 hours is described extensively in this chapter. For results concerning the other simulations reference is made to Appendix E.

<table>
<thead>
<tr>
<th>Slope</th>
<th>Angle of repose (°)</th>
<th>Drawdown rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:5</td>
<td>30°</td>
<td>28 hours</td>
</tr>
<tr>
<td>1:4</td>
<td>25°</td>
<td>56 hours</td>
</tr>
<tr>
<td>1:3</td>
<td>18.4°</td>
<td>fully drained</td>
</tr>
</tbody>
</table>

Water level decrease of 4 meters in 28 hours

The geometry and the soil parameters are determined so the stability calculations can start. The soil parameters are printed in Table 4-1, the cross section used in this calculation is plotted in Figure 4-14. At the end of the receding limb the pore pressures are highest, resulting in the lowest slope stability. The slope stability factors are therefore determined at the end of the receding limb.

In Figure 4-16 the deformed mesh is depicted. This mesh shows the failure state. With the Phi-c reduction approach a failure profile is always the outcome, it shows where the weakest points are in the given cross section. The blue line represents the water level inside the slope, and the water pressure on the slope.

![Figure 4-16: Deformed mesh after the Phi-c reduction calculation](image)

The effective stresses can be presented in the relative shear stresses $\tau_{rel}$. The relative shear stress option gives an indication of the proximity of the stress point to the failure envelope. The relative shear stress is defined as:

$$\tau_{rel} = \frac{\tau}{\tau_{\text{max}}}$$

(4.13)

Where $\tau_{\text{max}}$ is the maximum value of shear, i.e. the radius of the Mohr stress circle. In Figure 4-17 the effective relative shear stresses are depicted, where the geometry is colored red the slope fails.
Figure 4-17: Relative shear stresses

The groundwater flow module of PLAXIS calculates a pore pressure distribution. In Figure 4-18 the degree of saturation is plotted after the drawdown of 28 hours. The level of saturation is still very high in the slope.

Figure 4-18: Degree of saturation

The flow field is depicted in Figure 4-19, the highest flow velocity is found at the location where the water surface intersects the slope face after the drawdown.

Figure 4-19: Flow field

Stability factors for different scenarios

In Figure 4-20 the stability factors are plotted for the different scenarios. A couple of straightforward remarks can be made:

- The stability factor is larger when the angle of repose increases;
- The stability decreases when the steepness of the slope increases;
- The slope 1:3 (18.4°) is not stable in the situation for \( \varphi = 25^\circ \).

The main conclusion from this is that lowering the water level at a slower pace has a positive effect on the stability.

The discontinuous lines represent the runs where the water level is lowered at a pace so a fully drained situation occurs, there is no pressure imbalance in the slope. The fully drained situations show that the bank itself is stable regardless of the angle of repose or slope. However a fully drained situation is never the case since the drawdown of 4 meters has to be done in more than half a year as it takes that time to fully drain the bar. However in that period the beach has totally eroded due to various hydrodynamic forcing, as was seen from the photos taken from the Willie Taylor pool.
Figure 4-20: Stability factors for different scenarios

**Stability factor for various drawdown rates**

By varying the drawdown time for the reference cross section the stability factor increase is calculated. In Figure 4-21 the stability factor for the different drawdown rates is plotted. What can be observed from this plot is that the drawdown rate has a significant effect on the stability of the slope. Especially in the range from 0 to 5 days, the factor of safety increases a lot. The maximum stability factor, 2.8, is reached in a drawdown rate of 4 meters in 150 days.

---

Figure 4-21: Stability factor in time for different drawdown rates

The daily inequality

The dam releases water for power generation, this varies during the day. In Figure 4-22 (1) the dam release is plotted for a week in February of 2009. This variation is about 4500 cfs (120 m$^3$/s) and results in a water level fluctuation of 0.5 meter. With this fluctuation a stability calculation with PLAXIS is done. The hydrograph for this calculation starts with the falling limb of the high
flow experiment. This is followed by three days of the daily inequality dam release. To compare the effect of what a variable discharge will do to the stability of the bar a similar run is done but with a constant water level following the falling limb. These hydrographs are plotted in Figure 4-22 (2).

![Hydrograph](http://waterdata.usgs.gov)

**Figure 4-22: (1) Daily inequality of the dam release, (http://waterdata.usgs.gov), (2) Hydrographs used in PLAXIS**

The stability factor for the scenario based on the normal dam release is 1.9. The stability factor for the situation where the water level is kept constant is 2.7, this is a large increase in stability. Both runs have a larger factor of safety compared to the run where the stability factor is determined right after the falling limb, which was 1.6. The reason why this occurs is that PLAXFLOW calculates the pore pressures in time, but PLAXIS only uses the end values of the pore pressures. The pore pressures after a period of a constant water level are a lot smaller compared to the pore pressures after daily fluctuations of the water level. This results in a higher stability factor when the water levels are kept constant. It is worth noticing that the stability of the slope is lowest just after the falling limb.

### 4.6 Creating stable beaches

When a rapid drawdown is executed the stabilizing effect of the water on the upstream face is lost, but the pore water pressures within the slope may remain high. The dissipation of pore water pressure in the slope is largely influenced by the permeability and the storage characteristic of the bank material. The creation of the stable beaches is important as they undergo a lot of hydrodynamic forcing.

The drawdown rate has an effect on the stability of the slopes. The stability factors calculated with PLAXIS are relatively high, since nothing was left of the bar in the Willie Taylor pool after a couple of months. This implies that there are more hydrodynamic forces which have a negative effect on the slope stability. The eddies in the pools are transporting sediment from the bar into the main channel where it is located further downstream.

The erosion of the bar is not a linear process. Immediately after the flood the erosion process is very fast and it slows down over time. The daily fluctuations have a negative effect on the stability of the slopes so this should be decreased to a minimum.

When no constructive measures can be taken, the only thing to adjust is the dam release. The operators of the dam link the dam release to the power consumption. But if the daily fluctuation is set to a minimum for a period after the high flow the beaches have a better chance of a longer life. The conclusions drawn here are valid for all the bars in the Colorado River, not just the bar in the Willie Taylor pool.
5 Conclusions and recommendations

This chapter summarizes the main findings of this study. This study investigated the creation of sandbars along the Colorado River by releasing a lot of water at the Glen Canyon Dam. The main question was: Is it possible to design a dam release such that sandbars will accrete and remain stable on the long term? The question consists of two parts, first whether it is possible to create beaches with special regulations for the dam release. The second part of the question is under which conditions the sandbars which are created will be stable.

5.1 Conclusions

Creation of sandbars
The creation of the sandbars in the Eminence and Willie Taylor pool has been investigated using Delft3D. The sandbars are formed under high discharge conditions. The most important findings are:

- When an initial bed layer of sediment is used in the simulations, deposition and erosion patterns on the river bed are produced. The simulations give results that compare well with what was found during the experiment of 2008. In the simulation deposited volumes on the bars in the Willie Taylor pool amounted to about 80% of the deposited volume in the experiment. The main characteristics of the bars are reproduced by Delft3D, the shape of the bar and the return channel are both comparable to what was found after the experiment of 2008.
- Adjusting the rise and peak length of the hydrograph results into different volumes of deposition of sediment in the pool areas. Beaches are created during the rise and the peak of the high flow, when sediment is deposited into the relatively sheltered areas. Most of the sediment is deposited in the first 19 hours of the peak. However the deposition rate is still higher during the peak period opposed to during the rise.
- Short peaks may dampen too much to have sufficient submergence of bars in the downstream reaches. Long peak durations could transport all sediment downstream, causing more erosion than accretion.
- The inflow of sediment was decreased by 50% compared to the measured value. For the Willie Taylor pool the volume of deposition was the same order of magnitude. However, for the Eminence pool not enough deposition took place, but the bar extended more into the pool compared to what happened in the experiment.
- It is realized that chosen model approach is not an exact reproduction of the reality. The model is very sensitive to initial sand layer and inflow concentration. Nevertheless it is believed that the tendencies shown here one reasonably representative for the actual events.

Creation of the stable beaches
The creation of the stable beaches is difficult as they undergo a lot of hydrodynamic forcing. With the computer program PLAXIS stability calculations have been made and the time lapse photos of the Willie Taylor pool have been transformed and analyzed. The most important findings are:

- Lowering the water level at a slower pace has a positive result on slope stability. The sliding and eroding of the beaches starts at the end of the peak flow as the water level is dropping. When the water level drops too fast a bank cannot drain quickly enough, so a pressure imbalance will be developed which has a negative result on bank stability. This pressure imbalance is larger when the water level is drawdown faster.
- Daily fluctuations have a negative effect on the stability of the slopes, compared to a constant water level, so the fluctuation should be reduced to a minimum.
• The bank retreat of the bar is not a linear process: immediately after the flood the erosion process is very fast and it slows down over time. It follows a more or less exponential behavior.
• Apart from constructive measures, there is only one adjustable aspect that might be taken into consideration. If the discharge fluctuations after a flood are kept at a minimum the bars will exist longer.

*Designing a dam release so sandbars are created and stable*
Combining the results it seems possible to create a dam release scheme to enhance accretion and stability of the beaches. The resulting sandbars may resist the hydrodynamic forces for a longer time.

### 5.2 Recommendations
Collect more data on site during a next high flow experiment to allow more accurate model prediction:
- Collect more suspended sediment samples at various locations in the pool area, as the suspended sediment concentrations depend on local circumstances.
- Gathering more information on the sand geotechnical properties; important parameters are the angle of repose and Young’s modulus. The permeability is an important parameter for how fast a beach can be drained.

Further research on slope stability:
- Investigate to what extent breaching occurs on these newly developed beaches after a high flow. The breaching process may occur in natural circumstances due to a local slope collapse. It takes place below the water surface under the influence of gravity.
- Further investigation of the combination of processes in the river that causes bar erosion. With PLAXIS only the hydrodynamic forcing, rapid drawdown was applied, resulting in mostly positive stability factors. A different program is needed when other forces are taken into account.

Creation of bars:
- Delft3D is a model which reproduces a good result compared to what really happened. However, more research should be done on longer sections of the river. Different scenarios can be done to investigate the influence of the hydrograph on the bar building process, and the inflow of suspended sediment should be researched.
6 Literature

6.1 Books, reports and papers


6.2 Internet

US Geological Survey
www.usgs.gov

Water data for the Grand Canyon area
http://waterdata.usgs.gov

Grand Canyon Monitoring and Research Centre
www.gcmrc.gov

Coordinate Conversion
# List of symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
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<tbody>
<tr>
<td>A₀</td>
<td>original cross-section area through which the force is applied</td>
<td>m²</td>
</tr>
<tr>
<td>a</td>
<td>Van Rijn’s reference height</td>
<td>m</td>
</tr>
<tr>
<td>A</td>
<td>Rouse number</td>
<td>-</td>
</tr>
<tr>
<td>B</td>
<td>effect of bottom friction</td>
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</tr>
<tr>
<td>C</td>
<td>Chézy coefficient</td>
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<tr>
<td>Cᵦₚ₀</td>
<td>Barotropic Courant number</td>
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<td>concentration of sediment fraction</td>
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<td>mass concentration at reference height a</td>
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<td>f</td>
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<td>turbulent momentum flux in ξ and η-direction</td>
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<td>permeability of the medium</td>
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<td>truncation wave number</td>
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<td>Mₓ, Mᵧ</td>
<td>the contribution due to external sources of momentum</td>
<td>m/s²</td>
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<td>gradient hydrostatic pressure</td>
<td>kg/(m²s³)</td>
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<td>p</td>
<td>pressure head</td>
<td>m</td>
</tr>
<tr>
<td>Q</td>
<td>global sink or source per unit area</td>
<td>m/s</td>
</tr>
<tr>
<td>qᵢᵥ, qᵢₒ</td>
<td>local sources and sinks of water per unit volume</td>
<td>1/s</td>
</tr>
<tr>
<td>Sₓ, Sᵧ</td>
<td>components of the bed load transport</td>
<td>kg/ms</td>
</tr>
</tbody>
</table>
Specific storage $S$ -

Sum of the horizontal strain rates $S^*$ $1/s$

Dimensionless adaptation time for the vertical sediment concentration profile $T_{sd}$ -

Transformation matrix $T$ -

Time $t$ -

Effective bed shear velocity $u_*$ $m/s$

Depth averaged horizontal velocity $U$ $m/s$

Flow velocity components $u,v,w$ $m/s$

Horizontal eddy viscosity $v_H$ $m/s^2$

Local bed shear stress due to currents $u_{*,c}$ $m/s$

Sediment settling velocity $w_s$ $m/s$

Vector of the world plane $X$ -

Vector of the image plane coordinates $x$ -

Vertical coordinate in PLAXIS $y$ $m$

Vertical coordinate in the Delft3D model $z$ $m$

Direction perpendicular to the cross section in PLAXIS $z_b$ $m$

Gravity head $z$ $m$

Elevation above the bed $z$ $m$

**Greek**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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</thead>
<tbody>
<tr>
<td>$\alpha_s$</td>
<td>correction factor</td>
<td>-</td>
</tr>
<tr>
<td>$\beta_{eff}$</td>
<td>effective Van Rijn’s ‘beta’ factor fraction</td>
<td>-</td>
</tr>
<tr>
<td>$\beta$</td>
<td>Van Rijn's ‘beta’ factor of the sediment fraction</td>
<td>-</td>
</tr>
<tr>
<td>$\Delta L$</td>
<td>amount by which the length of the object changes</td>
<td>m</td>
</tr>
<tr>
<td>$\Delta z$</td>
<td>difference in elevation between the centre of the $k_{mx}$ cell and Van Rijn’s reference height</td>
<td>m</td>
</tr>
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<td>bed porosity</td>
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<td>$\varepsilon_{xx/yy/zz}$</td>
<td>eddy diffusivities of sediment fraction</td>
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<td>vertical fluid mixing coefficients calculated by the $k-\varepsilon$ turbulence closure model</td>
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<td>$\varepsilon_s$</td>
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</tr>
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<td>$\rho$</td>
<td>fluid density</td>
<td>$kg/m^3$</td>
</tr>
</tbody>
</table>
\( \phi \)  \quad \text{the total fluid potential} \\
\( \phi \)  \quad \text{friction angle} \\
\( \psi \)  \quad \text{Poisson’s ratio} \\
\( \rho_s \)  \quad \text{sediment density}  \quad \text{kg/m}^3 \\
\( \sigma_T \)  \quad \text{Prandtl Schmidt number} \\
\( \sigma_n \)  \quad \text{normal stress component}  \quad \text{N/m}^2 \\
\( \sigma \)  \quad \text{velocity in the } \sigma \text{-direction in the } \sigma \text{-coordinate system}  \quad \text{m/s} \\
\( \tau_c \)  \quad \text{bed shear stress due to currents}  \quad \text{N/m}^2 \\
\( \tau_w \)  \quad \text{bed shear stress due to waves}  \quad \text{N/m}^2 \\
\( \omega \)  \quad \text{vertical velocity in the } \sigma \text{-coordinate system}  \quad \text{m/s} \\
\( \zeta, \eta \)  \quad \text{coordinates of the curvilinear grid} \\
\( \psi \)  \quad \text{dilatancy angle} \\
\( \zeta \)  \quad \text{free-surface elevation above the reference plane}  \quad \text{m}
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Appendix A Data from the high flow experiment of 2008

Figure A-1: Sediment concentration for each time step for different size ranges, (courtesy of Wright)

The median grain size of the sediment on the river bed is: $D_{50} = 230 \mu m$

Table A-1: Suspended sediment for different grain size ranges, (courtesy of Wright)

<table>
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<tr>
<th>Survey Times</th>
<th>VFS (kg/m³)</th>
<th>FS (kg/m³)</th>
<th>MS (kg/m³)</th>
<th>CS (kg/m³)</th>
<th>VCS (kg/m³)</th>
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</thead>
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<tr>
<td>4-3-2008 17:20</td>
<td>0.02</td>
<td>0.01</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
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<td>0.15</td>
<td>0.05</td>
<td>0</td>
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</tr>
<tr>
<td>6-3-2008 09:08</td>
<td>1.63</td>
<td>1.08</td>
<td>0.31</td>
<td>0.03</td>
<td>0</td>
</tr>
<tr>
<td>7-3-2008 08:57</td>
<td>1.09</td>
<td>1.04</td>
<td>0.32</td>
<td>0.03</td>
<td>0</td>
</tr>
<tr>
<td>8-3-2008 08:53</td>
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<tr>
<td>9-3-2008 07:54</td>
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<td>0.02</td>
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<td>0.23</td>
<td>0.09</td>
<td>0.01</td>
<td>0</td>
</tr>
<tr>
<td>10-3-2008 10:12</td>
<td>0.08</td>
<td>0.04</td>
<td>0.01</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
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Figure A-3: Topography of the Willie Taylor pool: before, during and after the experiment, the depth is referenced to the 8000 cfs discharge, (courtesy of Kaplinski)
Appendix B  The Delft3D model

In this appendix a description is given of the Delft3D model. The basic equations are explained which were used in the model. At the end of the appendix the modeling results are plotted.

B.1 Flow computation

The software package Delft3D is a process based numerical model, which is under continuous development of Deltares. Delft3D can predict the flow for rivers, coastal areas, estuaries, shallow seas and lakes in two-dimensional (2D, depth-averaged) or three-dimensional (3D) unsteady flow and transport phenomena. Delft3D consists of different modules, which can interact with each other and each focuses on a specific process. The module Flow is used for the hydrodynamic computations. It describes the non-steady flow and transport phenomena. This concerns situations where the flow phenomena have a horizontal scale (both length and time) which is significantly larger than the vertical scale (depth), such as in rivers. The flow results from river discharge which is one of the boundary conditions.

In a morphological model, the Flow module is the main component, because that is the first step in modeling activities since every problem in river engineering concerns flows. The results of the flow calculation are used as input in other modules of Delft3D. For this study a three dimensional approach is used since there is significant variation in the transport in vertical direction.

Delft3D-Flow is based on a number of assumptions and approximations. A detailed description of these assumptions is provided in the Delft3D-Flow User Manual (Deltares, 2008).

B.1.1 Momentum equation in horizontal direction

Delft3D-Flow solves the Navier-Stokes-equations for an incompressible fluid under shallow water and Boussinesq assumptions, in two horizontal depth-averaged dimensions on a curvilinear grid.

For the general case of the curvilinear coordinates $\xi$ and $\eta$, the momentum equations in $\xi$- and $\eta$- direction are:

$$\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial \xi} + v \frac{\partial u}{\partial \eta} + \frac{\omega}{H} \frac{\partial u}{\partial \sigma} + \frac{v}{\sqrt{G_{\xi\xi} G_{\eta\eta}}} \left( \frac{u \cdot \frac{\partial}{\partial \xi} \sqrt{G_{\xi\xi}}}{\frac{\partial}{\partial \eta}} - \frac{v \cdot \frac{\partial}{\partial \eta} \sqrt{G_{\eta\eta}}}{\frac{\partial}{\partial \xi}} \right) - f v = - \frac{1}{\rho_0 \sqrt{G_{\xi\xi}}} P_{\xi} + F_{\xi} + \frac{1}{H^2} \frac{\partial}{\partial \sigma} \left( v \frac{\partial u}{\partial \sigma} \right) + M_{\xi}$$

(B.1)

and

$$\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial \xi} + v \frac{\partial v}{\partial \eta} + \omega \frac{\partial v}{\partial \sigma} + \frac{u}{\sqrt{G_{\xi\xi} G_{\eta\eta}}} \left( \frac{v \cdot \frac{\partial}{\partial \xi} \sqrt{G_{\xi\xi}}}{\frac{\partial}{\partial \eta}} - \frac{u \cdot \frac{\partial}{\partial \eta} \sqrt{G_{\eta\eta}}}{\frac{\partial}{\partial \xi}} \right) + f u = - \frac{1}{\rho_0 \sqrt{G_{\eta\eta}}} P_{\eta} + F_{\eta} + \frac{1}{H^2} \frac{\partial}{\partial \sigma} \left( v \frac{\partial v}{\partial \sigma} \right) + M_{\eta}$$

(B.2)

where

1. Inertia
2. Advective terms
3. Terms created by transforming the equation from a rectilinear grid into curvilinear grid
4. Coriolis force
5. Baroclinic pressure term
6. Unbalance of horizontal Reynold’s stresses
7. Vertical eddy viscosity
8. External sources or sinks of momentum

where

\( u, v, w \) flow velocities [m/s]
\( f \) Coriolis parameter [1/s] in this case \( f=0 \)
\( G_{\xi \xi}, G_{\eta \eta} \) coefficients used to transform curvilinear into rectangular coordinates [m]
\( \xi, \eta \) coordinates of the curvilinear grid [-]
\( \omega \) vertical velocity in the \( \sigma \)-coordinate system (computed from the continuity equation) [m/s]
\( \sigma \) velocity in the \( \sigma \)-direction in the \( \sigma \)-coordinate system [m/s]
\( P_{\sigma}, P_{\eta} \) pressure gradients [kg/(m\(^2\)s\(^2\)]
\( M_{\sigma}, M_{\eta} \) the contribution due to external sources of momentum [m/s\(^2\)]
\( F_{\sigma}, F_{\eta} \) Turbulent momentum flux [m/s\(^2\)]

### B.1.2 Vertical velocities

The vertical velocity \( \omega \) in the adapting \( \sigma \)-coordinate system is computed from the continuity equation:

\[
\frac{\partial \xi}{\partial t} + \frac{1}{\sqrt{G_{\xi \xi} G_{\eta \eta}}} \frac{\partial}{\partial \xi} \left[ \left( d + \xi \right) u \sqrt{G_{\eta \eta}} \right] + \frac{1}{\sqrt{G_{\xi \xi} G_{\eta \eta}}} \frac{\partial}{\partial \eta} \left[ \left( d + \xi \right) v \sqrt{G_{\xi \xi}} \right] + \frac{\partial \omega}{\partial \sigma} = H \left( q_{in} - q_{out} \right)
\]  

(B.3)

1. Water level gradient as a function of time
2. Specific discharge gradient in \( \xi \)-direction
3. Specific discharge gradient in \( \eta \)-direction
4. Gradient of vertical velocity relative to moving \( \sigma \)-plane
5. Local sources and sinks of water

### B.1.3 Hydrostatic pressure

Under the shallow water assumption (for \( \sigma \)-grid), the vertical momentum equation is reduced to a hydrostatic pressure equation. Vertical accelerations due to buoyancy effects and due to sudden variations in the bottom topography are not taken into account. So:

\[
\frac{\partial P}{\partial \sigma} = -g \rho H
\]  

(B.4)

### B.1.4 Numerical aspects

Delft3D is a numerical model based on finite differences. Therefore the shallow water equations have to be discretized. The shallow water equations are discretized via the staggered grid approach, see paragraph B.1.6, where the water level points are defined in the cell centers and the velocity components perpendicular on the middle of the grid cell faces.

For each model in Delft3D the numerical stability is checked with the Courant number. This number gives an indication of numerical stability and accuracy of a model. The Courant number can be adjusted by changing the time step. The downside of adapting the time step is that the computational time of the model will increase. For two dimensional models the Courant number is defined as (Stelling, 1984):

...
\[ C_r = 2 \Delta t \sqrt{\frac{gh}{\Delta x^2} + \frac{1}{\Delta y^2}} \]  
\textbf{(B.5)}

The Courant number is smaller than \( 4\sqrt{2} \).

\section*{B.1.5 Horizontal Large Eddy Simulation}

The standard numerical model computes a flow field until it reaches a steady state, which means that the velocity difference of two consecutive time steps is smaller than a preset value. What follows is the calculation of the sediment transport which results in an update of the bed topography. With this new topography a new flow field is calculated until a steady state is reached. When using the standard numerical model for this research study the morphological development in the ‘groyne’ field is poorly reproduced. The occurrence of the time varying turbulent eddies is the reason for this poor reproduction. These turbulent eddies are not resolved by this modeling technique. An eddy resolving approach is needed, such as Horizontal Large Eddy Simulation (HLES), (Yossef and Uijttewaal, 2003).

(Van Schijndel and Jagers, 2003) compared observations of model experiments and results of a 2D numerical simulations using Delft3D-FLOW with HLES. A good resemblance was found for the flow field as well as the sediment pattern in the case of emerged groynes. Van Schijndel and Jagers emphasized to use HLES when turbulent eddies have to be reproduced.

With the use of HLES the turbulent velocity field is decomposed into turbulence at large scales and small scales. The large scale turbulent structures interact with the main flow and the small scale turbulent structures interact with the large ones. With a coarse grid Delft3D is able to solve the large scale turbulent structures, however the smaller eddies cannot be resolved with this grid. A sub-grid-scale (SGS) model is necessary to model the effects of the unresolved small-scale turbulence, since the interaction between motions at all scales have to be taken into account.

According to (Uittenbogaard and Van Vossen, 2003), the proper use of HLES requires three criteria:

- The most restrictive criteria for the time step is the limitation of the barotropic Courant number \( C_{BT} \) which is imposed to make sure that the conservation of energy and enstrophy (=square of vorticity) is approximated well:

\[ C_{BT} = 2\Delta t \sqrt{\frac{gH}{\Delta x}} \leq 4\sqrt{2} \]  
\textbf{(B.6)}

Where \( g \) is the gravitational acceleration [m/s\(^2\)], \( \Delta t \) is the computational time step [s], \( \Delta x \) is the grid cell width in m-direction (flow direction) [m], and \( H \) is the water depth [m].

- In order to follow the temporal evolution of the vortex-vortex-interactions, the time step should be sufficiently small. This requirement is also called space-time consistence and the criterion to be met is the CFL- criterion for advection:

\[ C_U = \frac{|U| \Delta t}{\Delta x} \leq 1 \]  
\textbf{(B.7)}

Where \( U \) is the mean velocity in the main channel [m/s].

- The horizontal grid has to be sufficiently fine (at least in the order of the water depth) for being able to resolve the most energetic eddies as well as their interactions.

\[ \Delta t \leq \left( \frac{\Delta x}{2v^{(SGS)}} \right)^2 \]  
\textbf{(B.8)}

This is a purely numerical criteria and is rarely violated.
The add-on HLES with the SGS model computes the horizontal eddy viscosity as a function of time and position in the model domain. As the flow is shallow and quasi-2D, this computed horizontal eddy viscosity can then be used in the depth averaged momentum equation.

\[ v_{HS}^{(SGS)} = \frac{1}{k_s} \left( \sqrt{\gamma \sigma_T} S^* \right) + B^2 - B \]  

(B.9)

- \( v_{HS}^{(SGS)} \) sub-grid horizontal eddy viscosity computed by HLES [m²/s]
- \( k_s \) truncation wave number [1/m]
- \( \gamma \) coefficient [-] depending on the dimensionality of the turbulence 2D or 3D
- \( \sigma_T \) Prandtl Schmidt number [-], 0.7 is the default parameter for the SGS model
- \( S^* \) sum of the horizontal strain rates [1/s]

For more information about HLES reference is made to the Delft3D FLOW User Manual.

**B.1.6 Grid**

Models constructed in Delft3D work from a rectilinear or curvilinear boundary fitted grid. As mentioned before Delft3D uses a staggered grid approach, which means that different quantities are defined at different locations in a numerical grid cell (Figure B-1). Therefore the staggered grid approach gives a different number of numerical grid cells than one would expect based on the size of the modeled area. An advantage of the staggered grid approach is that boundary conditions can be implemented on the grid in a rather simple way. Boundaries are defined on different locations, closed boundaries are defined through u- or v points, as are velocities, but water levels are defined at water level points (+ points).

![Figure B-1: Staggered grid of Delft3D, after (Deltares, 2008)](image)

**B.1.7 The \( \sigma \)-coordinate system**

For most modeling the sigma grid is used. This grid divides the vertical in a number of layers independent of the water depth, see Figure B-2. This leads to a smooth representation of the topography instead of when the z-grid is used. The water depth can be divided in any number of layers with various thicknesses in percentages.

\[ \sigma = \frac{z - \zeta}{d + \zeta} = \frac{z - \zeta}{H} \]  

(B.10)

in which
- \( \sigma \) \( \sigma \)-coordinate [-]
- \( z \) depth below mean (still) water level [m]
- \( \zeta \) free-surface elevation above the reference plane [m]
- \( d \) water depth below the reference plane [m]
- \( H \) total water depth [m]
B.2 Morphological computation

B.2.1 Sediment transport

The advantage of the online-sediment add-on in Delft3D-FLOW is that the bed level is updated every half time step which is simultaneously with the flow computations, see Figure B-3. The add-on is very useful in case of complex hydrodynamic situations such as in the pool areas along the Colorado River.

Figure B-3: Morphological modeling procedure when using Delft3D-FLOW

The description of sediment transport in this paragraph is given with respect to the model set up and sediment transport formulations applied here. For the sediment transport calculations Van Rijn’s (1984) sediment transport model is used. This transport relation is commonly used for fine sediments in situations without waves. Van Rijn makes a distinction between bed load and suspended load transport. Note that for non-uniform sediment the loads are multiplied with their availability in the bed (locally).

Bed load transport

Bed load transport ($S_b$) consists of sediment transported over the bed layer [kg/ms].

$$S_b = \begin{cases} 0.053 \sqrt{D_g D_{s0}^3} D_s^{0.3} T^{2.1} & \text{for } T < 3.0 \\ 0.1 \sqrt{D_g D_{s0}^3} D_s^{0.3} T^{1.5} & \text{for } T \geq 3.0 \end{cases} \quad (B.11)$$

$T$ is the transport parameter which is following from the bed-shear stress $\tau_b$ related to grains and critical shear stress $\tau_{cr}$ as follows:

$$T = \frac{\tau_b - \tau_{bcr}}{\tau_{bcr}} \quad (B.12)$$

In this model value of $\tau_b$ is computed from the actual shear using a form factor $\mu$: 
\[ \tau_0' = \mu \cdot \tau_b \quad \text{with} \quad \mu = \left( \frac{C}{C'} \right)^2 \]  

(Equation B.13)

In which \( C \) is computed for the actual roughness height (equal to \( k_s \) value) and \( C' \) computed for a roughness height equal to 3\( D_{90} \) of the mixture.

The critical shear stress is written according to Shields:

\[ \tau_{bcr} = \rho_w g D_{50} \theta_{cr} \]  

(Equation B.14)

In which \( \theta_{cr} \) is the Shields parameter which is a function of the dimensionless particle parameter \( D_* \):

\[ D_* = D_{50} \left( \frac{\Delta g}{v^2} \right)^{\frac{1}{3}} \]  

(Equation B.15)

**Suspended transport**

In Van Rijn’s transport model the reference concentration near the bed is important for the suspended sediment calculations.

\[ c_{a,i} = 0.015 \frac{D_{50,i}}{\xi_c} T_i^{1.5} \frac{T_i^{0.3}}{D_{50,i}} \]  

(Equation B.16)

In which \( D_{50,i} \) is the dimensionless particle diameter and \( \xi_c \) is the reference level above the bed (associated to roughness height, e.g. \( k_s \) value of bed forms). \( T \) is the transport parameter which follows from the bed shear stress as is described in equation B.11.

In Delft3D simulations the value of \( c_a \) is used to determine the net-sediment flux near the bed. This entrainment-deposition flux follows from the difference between the actual concentration rates and the value of \( c_a \).

The suspended sediment concentration is computed by solving the Delft3D advection-diffusion equation, using the entrainment / deposition flux as a source term. The entrainment / deposition flux is scaled according to the availability of sediment in case of non-uniform sediment or non-erodible layers.

The exchange of sediment between the bed and the water column is modeled using sink and source terms acting on the near bottom layer that is entirely above Van Rijn’s reference height. This reference height is called the kmx-layer. The sediment concentrations in the layers that lie below the kmx layer are assumed to rapidly adjust to the same concentration as the reference layer.

**Figure B-4: Schematization of deposition and erosion in Delft3D, after (Deltares, 2008)**

Every half time step, source and sink terms model the quantity of sediment entering the flow due to upward diffusion from the reference level and the sediment dropping out of the flow due to
sediment settling. The sink term is solved implicitly in the advection – diffusion equation, whereas a source term is solved explicitly.

The main advantages of this online approach are the following (Lesser et al., 2004):

1) Three dimensional hydrodynamic processes and the adaptation of non-equilibrium sediment concentration profiles are automatically accounted for in the suspended sediment calculations;
2) The density effects of sediment in suspension are automatically included in the hydrodynamic calculations;
3) Changes in bathymetry can immediately be fed back to the hydrodynamic calculations;
4) Sediment transport and morphodynamic simulations are simple to perform and do not require a large data file to communicate results between the hydrodynamic, sediment transport, and bottom updating modules.

Figure B-5: Approximation of concentration and concentration gradient at bottom of kmx layer, after (Deltares, 2008)

To determine the required sink and source terms for the kmx layer, the concentration and concentration gradient at the bottom of the kmx layer need to be approximated. A standard Rouse profile between the reference level \( a \) and the centre of the kmx layer is assumed.

\[
c = c_a \left[ \frac{a(H-z)}{z(H-a)} \right]^A
\]

where
- \( c \) concentration of sediment fraction \([\text{kg/m}^3]\)
- \( c_a \) reference concentration of sediment fraction \([\text{kg/m}^3]\)
- \( a \) Van Rijn’s reference height \([\text{m}]\)
- \( H \) water depth \([\text{m}]\)
- \( z \) elevation above the bed \([\text{m}]\)
- \( A \) Rouse number \([-]\)

Now the reference concentration and the concentration in the centre of the kmx layer, \( c_{kmx} \) are known the exponent \( A \) can be determined:

\[
c_{kmx} = c_a \left[ \frac{a(h-z_{kmx})}{z_{kmx}(h-a)} \right]^A \Rightarrow A = \frac{\ln \left( \frac{c_{kmx}}{c_a} \right)}{\ln \left( \frac{a(h-z_{kmx})}{z_{kmx}(h-a)} \right)}
\]

The concentration at the bottom of the kmx layer is:
\[ c_{\text{kmx}(\text{bot})} = c_a \left[ \frac{a(h - z_{\text{kmx}(\text{bot})})}{z_{\text{kmx}(\text{bot})}(h - a)} \right]^{A} \]  

(B.19)

This concentration is expressed as a function of the known concentration \(c_{\text{kmx}}\) by introducing a correction factor \(\alpha_1\):

\[ c_{\text{kmx}(\text{bot})} = \alpha_1 c_{\text{kmx}} \]  

(B.20)

The concentration gradient of the Rouse profile is as follows:

\[ \frac{\partial c}{\partial z} = A c_a \left[ \frac{a(h - z)}{z(h - a)} \right]^{A-1} \cdot \left( \frac{-ah}{z^2(h - a)} \right) \]  

(B.21)

The concentration gradient at the bottom of the kmx layer is:

\[ c'_{\text{kmx}} = A c_a \left[ \frac{a(h - z_{\text{kmx}(\text{bot})})}{z_{\text{kmx}(\text{bot})}(h - a)} \right]^{A-1} \cdot \left( \frac{-ah}{z^2_{\text{kmx}(\text{bot})}(h - a)} \right) \]  

(B.22)

This gradient is expressed as a function of the known concentrations \(c_a\) and \(c_{\text{kmx}}\) corrected with a factor \(\alpha_2\):

\[ c'_{\text{kmx}(\text{bot})} = \alpha_2 \left( \frac{c_{\text{kmx}} - c_a}{\Delta z} \right) \]  

(B.23)

**Erosive flux due to upward diffusion**

The upward diffusion of sediment through the bottom of the kmx layer is given by the following expression for the erosive flux, which is split in a source and sink term:

\[ E = \varepsilon \frac{\partial c}{\partial z} = \alpha_2 \varepsilon_1 \frac{(c_a - c_{\text{kmx}})}{\Delta z} = \alpha_2 \frac{\varepsilon_c c_a}{\Delta z} - \alpha_2 \frac{\varepsilon_c c_{\text{kmx}}}{\Delta z} \]  

(B.24)

Where \(\varepsilon_a\) and \(\frac{\partial c}{\partial z}\) are evaluated at the bottom of the kmx layer.

\[ \Delta z = z_{\text{kmx}} - h \]

(B.25)

**Deposition flux due to sediment settling**

The settling of sediment through the bottom of the kmx cell is given by the expression:

\[ D = w_c c_{\text{kmx}} \]  

(B.26)

We set:

\[ c_{\text{kmx}(\text{bot})} = \alpha_1 c_{\text{kmx}} \]  

(B.27)

The deposition flux is approximated by:

\[ D \approx \alpha_1 c_{\text{kmx}} w_c \]  

(B.28)

This results in a simple deposition sink term:

\[ \text{Sink}_{\text{deposition}} = \alpha_1 c_{\text{kmx}} w_c \]  

(B.29)

The total source and sink terms is given by:
\[
\text{Source} = \alpha_s c \left( \frac{e_s}{\Delta z} \right) \\
\text{Sink} = \left[ \alpha_s \left( \frac{e_s}{\Delta z} \right) + \alpha_s w_s \right] c_{kne}
\]

These source and sink terms are both guaranteed to be positive.

**B.2.2 Vertical mixing**

The vertical mixing coefficient can be calculated directly from the vertical fluid mixing coefficient calculated by the turbulence closure model:

\[
\varepsilon_s = \beta_{\text{eff}} \varepsilon_f
\]

where

- \( \varepsilon_s \) vertical sediment mixing coefficient of sediment fraction
- \( \beta_{\text{eff}} \) the effective Van Rijn’s ‘beta’ factor fraction. As the beta factor should only be applied to the current related mixing this is estimated as:

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

For non-cohesive sediment fractions:

- \( \beta \) Van Rijn’s ‘beta’ factor of the sediment fraction.
- \( \tau_c \) bed shear stress due to currents
- \( \tau_w \) bed shear stress due to waves
- \( \varepsilon_t \) vertical fluid mixing coefficients calculated by the k-\( \varepsilon \) turbulence closure model

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

- \( w_s \) settling velocity of the non-cohesive sediment fraction
- \( u_{*c} \) local bed shear stress due to currents

This implies that the value of \( \beta \) is space (and time) varying however it is constant the depth of the flow.

**B.2.3 The morphological scale factor**

The morphological scale factor up scales the velocity of morphological changes. It is a multiplication factor, with which the morphological changes which take place during the computed flow time, are multiplied. This method allows computing morphological changes over a long period in moderate computing times:

\[
\Delta t_{\text{morphology}} = \int_{\text{run}} \Delta t_{\text{hydrodynamic}}
\]

This factor is used because morphological changes often take place over a much longer period as compared to the hydrodynamic changes. The morphological changes due to the changes in the hydrodynamics are often very small and do not affect the hydrodynamics and sediment transport pattern much. There are limits to the morphological scale factor. Small bed level changes are exaggerated by this method and by varying hydrodynamic conditions are exaggerated by this method and the results can be unrealistic.

**B.2.4 Spin up time**

The spin up time is the time before morphological changes are taken into account. The purpose of using a spin up time is to prevent that initial instabilities in the sediment transport have effect on the bed level. During this time no morphological changes are made to allow hydrodynamics to adjust to the initial bathymetry.
### B.3 Sediment fractions used in Delft3D

The thickness of the bed layer was determined using the lowest measured depth ever from the pool areas and the bed level just before the high flow. Subtracting those layers resulted in a layer with various thicknesses. In the rapid areas no sand was available and in the pool areas a layer of sediment was present before the high flow.

In the following tables the boundary conditions used in the Delft3D simulations are plotted. In Table B-1 the sediment fractions are plotted which were used in the model. In Table B-2, Table B-3, Table B-4, Table B-5 and Table B-6 the inflow conditions for the different runs in Delft3D are plotted. The discharge, the water level, and the concentration for the various sediment fractions are given for the different times.

#### Table B-1: Sediment fractions and fall velocity applied in the model

<table>
<thead>
<tr>
<th>Type</th>
<th>Lower size [mm]</th>
<th>Upper size [mm]</th>
<th>Fall velocity [m/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very fine sediment</td>
<td>0.062</td>
<td>0.125</td>
<td>0.0060</td>
</tr>
<tr>
<td>Fine sediment</td>
<td>0.125</td>
<td>0.25</td>
<td>0.0185</td>
</tr>
<tr>
<td>Medium sediment</td>
<td>0.25</td>
<td>0.5</td>
<td>0.0484</td>
</tr>
<tr>
<td>Coarse sediment</td>
<td>0.5</td>
<td>1</td>
<td>0.109</td>
</tr>
<tr>
<td>Very coarse sediment</td>
<td>1</td>
<td>4</td>
<td>0.283</td>
</tr>
</tbody>
</table>

#### Table B-2: Inflow boundary conditions for reference run

<table>
<thead>
<tr>
<th>Time</th>
<th>Discharge [m³/s]</th>
<th>Water level [m]</th>
<th>VFS [kg/m³]</th>
<th>FS [kg/m³]</th>
<th>MS [kg/m³]</th>
<th>CS [kg/m³]</th>
<th>VCS [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-3-2008 05:00</td>
<td>312</td>
<td>836.54</td>
<td>0.01</td>
<td>0.02</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6-3-2008 12:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.6</td>
<td>0.4</td>
<td>0.15</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>8-3-2008 23:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.25</td>
<td>0.2</td>
<td>0.1</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>9-3-2008 18:00</td>
<td>312</td>
<td>836.54</td>
<td>0.1</td>
<td>0.1</td>
<td>0.05</td>
<td>0.01</td>
<td>0</td>
</tr>
</tbody>
</table>

#### Table B-3: Inflow boundary conditions hydrograph 1

<table>
<thead>
<tr>
<th>Time</th>
<th>Discharge [m³/s]</th>
<th>Water level [m]</th>
<th>VFS [kg/m³]</th>
<th>FS [kg/m³]</th>
<th>MS [kg/m³]</th>
<th>CS [kg/m³]</th>
<th>VCS [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-3-2008 05:00</td>
<td>312</td>
<td>836.54</td>
<td>0.01</td>
<td>0.02</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6-3-2008 12:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.6</td>
<td>0.4</td>
<td>0.15</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>7-3-2008 02:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.25</td>
<td>0.2</td>
<td>0.1</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>9-3-2008 18:00</td>
<td>312</td>
<td>836.54</td>
<td>0.1</td>
<td>0.1</td>
<td>0.05</td>
<td>0.01</td>
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#### Table B-4: Inflow boundary conditions hydrograph 2

<table>
<thead>
<tr>
<th>Time</th>
<th>Discharge [m³/s]</th>
<th>Water level [m]</th>
<th>VFS [kg/m³]</th>
<th>FS [kg/m³]</th>
<th>MS [kg/m³]</th>
<th>CS [kg/m³]</th>
<th>VCS [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-3-2008 05:00</td>
<td>312</td>
<td>836.54</td>
<td>0.01</td>
<td>0.02</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7-3-2008 02:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.6</td>
<td>0.4</td>
<td>0.15</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>7-3-2008 22:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.25</td>
<td>0.2</td>
<td>0.1</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>9-3-2008 18:00</td>
<td>312</td>
<td>836.54</td>
<td>0.1</td>
<td>0.1</td>
<td>0.05</td>
<td>0.01</td>
<td>0</td>
</tr>
</tbody>
</table>

#### Table B-5: Inflow boundary conditions hydrograph 3

<table>
<thead>
<tr>
<th>Time</th>
<th>Discharge [m³/s]</th>
<th>Water level [m]</th>
<th>VFS [kg/m³]</th>
<th>FS [kg/m³]</th>
<th>MS [kg/m³]</th>
<th>CS [kg/m³]</th>
<th>VCS [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-3-2008 05:00</td>
<td>312</td>
<td>836.54</td>
<td>0.01</td>
<td>0.02</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>8-3-2008 23:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.6</td>
<td>0.4</td>
<td>0.15</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>9-3-2008 00:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.25</td>
<td>0.2</td>
<td>0.1</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>9-3-2008 18:00</td>
<td>312</td>
<td>836.54</td>
<td>0.1</td>
<td>0.1</td>
<td>0.05</td>
<td>0.01</td>
<td>0</td>
</tr>
</tbody>
</table>
Table B-6: Inflow boundary conditions hydrograph 4

<table>
<thead>
<tr>
<th>Time</th>
<th>Discharge [m³/s]</th>
<th>Water level [m]</th>
<th>VFS [kg/m³]</th>
<th>FS [kg/m³]</th>
<th>MS [kg/m³]</th>
<th>CS [kg/m³]</th>
<th>VCS [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-3-2008 06:00</td>
<td>312</td>
<td>836.54</td>
<td>0.01</td>
<td>0.02</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6-3-2008 12:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.6</td>
<td>0.4</td>
<td>0.15</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>8-3-2008 23:00</td>
<td>1211</td>
<td>840.17</td>
<td>0.25</td>
<td>0.2</td>
<td>0.1</td>
<td>0.025</td>
<td>0</td>
</tr>
<tr>
<td>9-3-2008 00:00</td>
<td>312</td>
<td>836.54</td>
<td>0.1</td>
<td>0.1</td>
<td>0.05</td>
<td>0.01</td>
<td>0</td>
</tr>
</tbody>
</table>

B.4 Topography results of the Eminence pool
Figure B-6: Results from Delft3D Eminence pool, the depth is referenced to the 8000 cfs discharge

B.5 Topography results for the Willie Taylor pool

Figure B-7: Results from Delft3D Willie Taylor pool, the depth is referenced to the 8000 cfs discharge
Appendix C  Image transformation

Following is the elaboration of the projective image transformation:
The camera model could be written as
\[ X = Hx \]  \hspace{1cm} \text{(C.1)}

where:
- \(X\) is the vector of the world plane
- \(H\) is the matrix transform
- \(x\) is the vector of the image plane coordinates

In more detail this is:
\[
\begin{pmatrix}
XW \\
YW \\
W
\end{pmatrix} = \begin{pmatrix} a & b & c \\ d & e & f \\ g & h & 1 \end{pmatrix} \begin{pmatrix} x \\ y \\ 1 \end{pmatrix}
\]  \hspace{1cm} \text{(C.2)}

\(W\) is actually:
\[
W = gx + hy + 1
\]

We can rewrite the equation in a way that exposes its true non-linear form where the numerator supplies the parameters needed for affine transformation, and the denominator allows for the non-linear effect of perspective:
\[
\begin{pmatrix}
X \\
Y \\
1
\end{pmatrix} = \begin{pmatrix} a & b & c \\ d & e & f \\ g & h & 1 \end{pmatrix} \begin{pmatrix} x \\ y \\ 1 \end{pmatrix}
\]  \hspace{1cm} \text{(C.3)}

This is equivalent to the non-vector form of the perspective transform for the coordinate sets \((x_0, y_0)\) and \((X_0, Y_0)\):
\[
X_0 = \frac{ax_0 + by_0 + c}{gx_0 + hy_0 + 1} \quad \text{and} \quad Y_0 = \frac{dx_0 + ey_0 + f}{gx_0 + hy_0 + 1}
\]  \hspace{1cm} \text{(C.4)}

Multiplying both sides with the denominator gives:
\[
X_0 (gx_0 + hy_0 + 1) = ax_0 + by_0 + c
\]
\[
Y_0 (gx_0 + hy_0 + 1) = dx_0 + ey_0 + f
\]  \hspace{1cm} \text{(C.5)}

Adding some zero values:
\[
X_0 = ax_0 + by_0 + c + d \cdot 0 + e \cdot 0 + f \cdot \frac{0 - gx_0 X_0 - hy_0 Y_0}{gx_0 + hy_0 + 1}
\]
\[
Y_0 = dx_0 + ey_0 + f - gx_0 Y_0 - hy_0 X_0
\]  \hspace{1cm} \text{(C.6)}

With 4 sets of coordinates the following matrix is created:
\[
\begin{pmatrix}
x_0 & y_0 & 1 & 0 & 0 & 0 & -x_0 X_0 & -y_0 X_0 \\
0 & 0 & x_0 & y_0 & 1 & -x_0 Y_0 & -y_0 Y_0 \\
x_1 & y_1 & 1 & 0 & 0 & 0 & -x_1 X_1 & -y_1 X_1 \\
0 & 0 & x_1 & y_1 & 1 & -x_1 Y_1 & -y_1 Y_1 \\
x_2 & y_2 & 1 & 0 & 0 & 0 & -x_2 X_2 & -y_2 X_2 \\
0 & 0 & x_2 & y_2 & 1 & -x_2 Y_2 & -y_2 Y_2 \\
x_3 & y_3 & 1 & 0 & 0 & 0 & -x_3 X_3 & -y_3 X_3 \\
0 & 0 & x_3 & y_3 & 1 & -x_3 Y_3 & -y_3 Y_3
\end{pmatrix} \begin{pmatrix} a \\ b \\ c \\ d \\ e \\ f \\ g \\ h \end{pmatrix} = \begin{pmatrix} X_0 \\ Y_0 \\ X_1 \\ Y_1 \\ X_2 \\ Y_2 \\ X_3 \\ Y_3 \end{pmatrix}
\]  \hspace{1cm} \text{(C.7)}

The transformation matrix is:
The transformation matrix I used is the following:

\[
T = \begin{pmatrix}
a & b & c \\
d & e & f \\
g & h & 1
\end{pmatrix}
\]  

(C.8)

\[
T = \begin{pmatrix}
0.887 & 351.9914 & 9176.7158 \\
2.6461 & 961.5565 & 24165.8816 \\
0.0001 & 0.0395 & 1
\end{pmatrix}
\]  

(C.9)
Appendix D  Tables and charts from the sand manual

The figures are taken from (Koster, 2004).

Table D-1 - For determining the $D_{10}$ and the $D_{60}$ of the sediment sample

Figure D-1 - Determining Poisson’s ratio for loosely packed sand

Figure D-2 - Used to determine the triaxial modulus

Figure D-3 - Used to determine the saturated soil weight

Figure D-4 - Used to determine the angle of repose

Figure D-5 - Used to determine the dry soil weight

Figure D-6 - Young’s modulus is determined using this correlation figure

Table D-1: Grain size distribution of deposited sediment on the bar of the Willie Taylor pool

<table>
<thead>
<tr>
<th></th>
<th>Cumulative grain-size distribution of the entire sample</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>%&lt; 0.037 mm</td>
</tr>
<tr>
<td>Silt</td>
<td>%&lt; 0.044 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.053 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.063 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.074 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.088 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.105 mm</td>
</tr>
<tr>
<td>VFS</td>
<td>%&lt; 0.125 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.149 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.177 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.210 mm</td>
</tr>
<tr>
<td>FS</td>
<td>%&lt; 0.250 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.297 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.354 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.420 mm</td>
</tr>
<tr>
<td>MS</td>
<td>%&lt; 0.500 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.595 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.707 mm</td>
</tr>
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</tr>
<tr>
<td>CS</td>
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<td>%&lt; 1.19 mm</td>
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<td></td>
<td>%&lt; 1.41 mm</td>
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<td></td>
<td>%&lt; 0.044 mm</td>
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<td></td>
<td>%&lt; 0.053 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.063 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.074 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.088 mm</td>
</tr>
<tr>
<td></td>
<td>%&lt; 0.105 mm</td>
</tr>
</tbody>
</table>
Figure D-1: Poisson’s ratio

Figure D-2: Triaxial modulus

Figure D-3: Saturated soil weight
### Figure D-4: Angle of repose

<table>
<thead>
<tr>
<th>Hoofdnaam</th>
<th>Bijmengsel</th>
<th>Consistentie</th>
<th>C_e</th>
<th>C_w</th>
<th>E_wat</th>
<th>q'</th>
<th>c'</th>
<th>f_se</th>
</tr>
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<td>Grond</td>
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<td>32.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<td>35</td>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Dicht</td>
<td>0</td>
<td>0.001 of 0</td>
<td>150 of 200</td>
<td>37.5 of 40</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Zwart siltig</td>
<td>Los</td>
<td>0</td>
<td>0.003</td>
<td>50</td>
<td>30</td>
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### Figure D-5: Determine dry soil weight from wet soil weight

#### Voorbeeld:
- ρ_0 = 1,50 t/m³ ofwel P_0 = 1,93 t/m³
- via 1: n = 43.4 %
- via 2: e = 0.77
- w = 28.0 %
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Figure D-6: Correlation table for different modulus
Appendix E  PLAXIS output

In the following figures the relative shear stresses are plotted for the various runs. Where the relative shear stress is 1, failure occurs.

slope 1:5, $\varphi = 25^\circ$, drawdown in 28 hours

slope 1:5, $\varphi = 25^\circ$, drawdown in 56 hours

slope 1:4, $\varphi = 25^\circ$, drawdown in 28 hours

slope 1:4, $\varphi = 25^\circ$, drawdown in 56 hours

slope 1:3, $\varphi = 25^\circ$, drawdown in 28 hours

slope 1:3, $\varphi = 25^\circ$, drawdown in 56 hours

Figure E-1: Relative shear stress for different slopes and drawdown rates with $\varphi = 25^\circ$
slope 1:5, $\varphi = 30^\circ$, drawdown in 28 hours

slope 1:5, $\varphi = 30^\circ$, drawdown in 56 hours

slope 1:4, $\varphi = 30^\circ$, drawdown in 28 hours

slope 1:4, $\varphi = 30^\circ$, drawdown in 56 hours

slope 1:3, $\varphi = 30^\circ$, drawdown in 28 hours

slope 1:3, $\varphi = 30^\circ$, drawdown in 56 hours

Figure E-2: Relative shear stress for different slopes and drawdown rates with $\varphi = 30^\circ$