Flooding

Classification & Simulation

M.J. de Vries

June 1999
Flooding

Classification & Simulation

M.J. de Vries
Preface

This report has been written in accordance to my Master’s research project for the study of Civil Engineering at the Delft University of Technology in Delft, The Netherlands. The goal of this study was to make an inventory of the numerical important aspects of flooding and to verify the correct implementation of these aspects in the simulation program DELFT-FLS.

I would like to thank the Strategic Research and Development department at WL | DELFT HYDRAULICS for giving me the opportunity to do this project and the people of ‘De Keet’ for the pleasant working atmosphere. I would especially like to thank my supervisor at WL | DELFT HYDRAULICS, Prof. dr. ir. Guus Stelling for his advice, enthusiasm and encouragement during this project.

Of course I would like to thank my family and friends for their support.

Mark de Vries
June, 99
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<table>
<thead>
<tr>
<th>Symbol</th>
<th>Meaning</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>C</strong></td>
<td>Chézy coefficient</td>
<td>m$^{1/2}$/s</td>
</tr>
<tr>
<td>$C_f$</td>
<td>Friction constant</td>
<td>-</td>
</tr>
<tr>
<td>$f$</td>
<td>Infiltration intensity</td>
<td>m/day</td>
</tr>
<tr>
<td>$g$</td>
<td>Gravitational constant</td>
<td>m/s$^2$</td>
</tr>
<tr>
<td>$h$</td>
<td>Water depth</td>
<td>m</td>
</tr>
<tr>
<td>$h_0$</td>
<td>Initial water level difference</td>
<td>m</td>
</tr>
<tr>
<td>$h_{sm}$</td>
<td>Maximum scourhole depth</td>
<td>m</td>
</tr>
<tr>
<td>$h_o$</td>
<td>Original scourhole depth</td>
<td>m</td>
</tr>
<tr>
<td>$K(S)$</td>
<td>Permeability</td>
<td>m/day</td>
</tr>
<tr>
<td>$k_s$</td>
<td>Nikuradse roughness length</td>
<td>m</td>
</tr>
<tr>
<td>$n$</td>
<td>Manning roughness</td>
<td>s/m$^{1/3}$</td>
</tr>
<tr>
<td>$R$</td>
<td>Hydraulic radius</td>
<td>m</td>
</tr>
<tr>
<td>$t$</td>
<td>Time</td>
<td>s</td>
</tr>
<tr>
<td>$u$</td>
<td>Flow velocity in x-direction</td>
<td>m/s</td>
</tr>
<tr>
<td>$u_c$</td>
<td>Critical velocity of incipient motion</td>
<td>m/s</td>
</tr>
<tr>
<td>$u_t$</td>
<td>Vertically averaged velocity at time = t</td>
<td>m/s</td>
</tr>
<tr>
<td>$v$</td>
<td>Flow velocity in y-direction</td>
<td>m/s</td>
</tr>
<tr>
<td>$v_i$</td>
<td>Initial flow velocity</td>
<td>m/s</td>
</tr>
<tr>
<td>$v_d$</td>
<td>Dynamic flow velocity</td>
<td>m/s</td>
</tr>
</tbody>
</table>

**GREEK**

- $\alpha$: Dimensionless amplification factor
- $\Delta$: Relative density $(\rho_s - \rho_w)/\rho_w$
- $\kappa$: Von Karman coefficient ($\approx 0.4$)
- $\rho_s$: Mass density of sediment
- $\rho_w$: Mass density of water
- $\tau_b$: Bottom shear stress
- $\Psi_f$: Pressure head at moisture front
- $\zeta$: Water level above reference

Subscripts $x$, $y$, and $z$ denote a variable with a direction along the $x$-, $y$- or $z$-axis respectively. Subscripts $x$, $y$, and $z$ denote if a variable is on a real or virtual point. Subscripts $x$ and $y$ denote if a variable is perpendicular or parallel to a line or area. Combinations are possible and the subscripts will then be separated by a ‘,’ (comma).
Abstract

Flooding of an area can be described mathematically and by the use of scale models. The different flow regimes can be discerned using the scale model and then these regimes can be modelled using mathematical relations. This mathematical description can then be used to design simulation computer code for such a flooding.

The first part of this report describes the different flow regimes that can occur after a defence work fails. For all regimes estimations were done of the dependence of that regime on measured data and of calculation methods. This report shows that the dynamic terms in the mathematical hydraulic descriptions have the upper hand during the first parts of the flooding. Later friction terms become more important. The dynamic terms prove to be highly dependent on the correct implementation of the mathematical relation in the simulation computer code. Bottom friction depends more on correct data from the field.

The second part of the report describes the verification of the capabilities of DELFT-FLS to simulate dynamic flow accurately. An improvement on the correct handling of straight closed bevelled boundaries was implemented. With the simulation of and comparison to a dam-break laboratory experiment the correct implementation of mathematical descriptions of flow is shown.
I Introduction

Floods can happen almost everywhere in the world, at almost any time. Main causes are heavy precipitation, dam failure, earthquakes, storms and human causes. Some recent examples of such floods can be found in China, where in July 1998 the Yangtze River burst its banks and flooded large areas for weeks. In the morning of 18 July 1998 a seven metre high Tsunami hit Aitape in Papua New Guinea, killing thousands of lives, after a magnitude 7 earthquake off the coast. The 1953 flooding of the south-western parts of the Netherlands, through more than 200 dike breaches, caused the deaths of 1835 people. After this disaster the construction of the Delta works was initiated.

Governments and local authorities are interested in simulating possible scenarios of these catastrophes before they happen, so early precautions can be taken, like evacuations, counter measures and the design of disaster plans. These scenarios can be simulated with flooding models like DELFT-FLS. However, the user must be convinced that the model simulates scenarios correctly. Therefore it must be known what happens when a flooding occurs, which factors are important to take into account when simulating a flooding, and whether these factors are implemented correctly into the simulation program. These aspects have been considered in the present study.

This thesis is divided into two main parts. The first part consists of determining the normative hydrodynamic properties of flooding. In chapter 2 the problem analysis, problem description and research objectives are presented. Chapter 3 describes the way flooding prevention and flooding damage control is handled in the Netherlands. A discussion on hydraulic defence works is given in chapter 4. This chapter includes an inventory of types, building materials and of failure causes and mechanisms of floods. The hydrodynamic aspects of flooding are discussed in chapter 5.

The second part of this thesis consists of the verification of the capabilities of DELFT-FLS on the normative hydrodynamic aspects of floods in the Netherlands. The DELFT-FLS program and the choice of the test-cases which will be used for the verification are described in chapter 6. In chapter 7 a 1Dh dam-break test case is presented to validate DELFT-FLS on the handling of non-parallel boundaries. The CADAM 45° Dam-Break test case is presented in chapter 8 to validate DELFT-FLS on dynamic flow. Finally chapter 9 presents some conclusions and recommendations.
2 Problem analysis

2.1 Problem description

Floods occur now and then, with different initial causes. As a flood usually causes a threat to people, livestock and properties, authorities want to decrease the risk of floods. Both in preventing them from happening at all and by controlling the damage from floods with proper disaster plans when they occur. Disaster plans are designed by evaluating different flooding scenarios. From the outcome of these evaluations proper measures can be taken, and thus the risks involved with floods are lowered. One method of evaluation that has gained much ground during the last few years is computer simulation.

Much research is being done to accurately simulate flooding. As a result of this research many programs and models have been constructed. Some programs are very specialised and can simulate one specific problem, like the simulation of the initial phase of a dam breach, very well, others are broader in application. These programs have different levels of accuracy. The accuracy depends highly on the amount of simplification of the real world and the implementation of the resulting model in a simulation program. Secondly the processing power to compute a case in an acceptable amount of time and the accuracy of the input data influence the outcome of simulations.

Developers of simulation programs have little influence on the accuracy of input data and the available processing power. They do have influence on the accuracy of the calculation model and the implementation of this calculation model in their simulation programs. Both the developers and the clients want an indication of the accuracy of this calculation model. To that end the calculation core of the simulation program needs verification on accuracy. This can be done by comparing the results of physical world test cases with the results of simulations of those test cases. These test cases have to incorporate the properties that influence the results of the simulation the most.

Before any verification can be done, first it has to be determined which properties have much influence of the results of the simulation and which have little influence. Then it has to be determined which of the influencing properties are dependant of the calculation model. Next some test cases have to be found that incorporate these properties and last the verification can be done by comparing the data from these test cases with the results of the simulation of these test cases using the simulation program.

DELT-FLS, which stands for DELFT FLOODING SYSTEM, is a simulation program being developed by WL | DELFT HYDRAULICS to simulate flooding. The main target group are the Dutch national and local governments which will use DELFT-FLS to make an inventory of the consequences and possible countermeasures of floods in The Netherlands. The calculation method in DELFT-FLS has been verified for normal flow regimes. However, the calculation method has been adapted to also compute extreme circumstances correctly. The calculation method of the program will have to be verified to this specification.
2.2 Problem definition and research objective

WL | DELFT HYDRAULICS wants to verify whether the extensions to simulate extreme flow circumstances in the calculation core of DELFT-FLS are implemented correctly. To that end they need an inventory of the hydrodynamic aspects of flow that are of importance for the correct workings of the extensions in the calculation core of DELFT-FLS.

The aim of this project is the verification of the extensions of DELFT-FLS, by making an inventory of relevant and normative hydrodynamic mechanisms of flow and by verifying DELFT-FLS using test cases that incorporate these mechanisms.

2.3 Approach

First an inventory will be made of flooding types known in the world by means of a literature study. For these flooding types an inventory will be made of the normative hydrodynamic mechanisms that influence flow and the extent of that influence. Then test cases will be sought that incorporate the hydrodynamic mechanisms that are influenced by the calculation core of DELFT-FLS.

These test cases will be simulated using DELFT-FLS and the results compared to the data of the test cases. If necessary simple modifications will be designed and implemented in the calculation core of DELFT-FLS. Because DELFT-FLS is still in development stage, for larger improvements only recommendations will given to be incorporated in later stages of the program.
3 Practical experience

Knowledge on defence against floods has reached high levels. Much is known about forecasting, defence work strengths and flow regimes. Defence works are designed to withstand water levels with a certain return period, though nature is still unpredictable and defence works do not always behave as predicted. Furthermore, an ever growing population has led to an increased use of land. This land is often low lying, on the banks of rivers and coasts or even land reclaimed from the water. These low lying areas have a high vulnerability for floods and even with good protection flooding can happen now and then.

Authorities want to know what happens when such a flood occurs, so damage can be controlled as much as possible. For evacuation plans, damage assessments and timetables they need to know the propagation speed and extent of a flood. The best way to do this is to come up with relevant scenarios for which plans can be made. These plans can then be used as a guideline for the local authorities.

In The Netherlands, responsibility for policies against and after floods is divided between local and national authorities.

The national government is responsible for the legislation and funding concerning the defence against disasters. It has a national scope and works on a more general basis. For example, it decides on the return periods for which defence works are designed. The government is also responsible for the international aspects of national defence against disasters and for communication with neighbouring countries.

In contrast to the national government, the local governments and authorities are more involved with the execution of the nationally made decisions. They have two major responsibilities: One, the management of the system of defence works and two, the mitigation of the effects of (local) flooding. The management of the system of defence works consists of designing, building and maintenance of levees and the design and operation of sluices, weirs, pumping stations, etc. The mitigation of effects of flooding consists of preparation of disaster plans, instruction of local authorities, like police and fire departments, and the co-ordination during floods.

For the preparation of disaster management plans, information is needed. In the case of floods, information is needed on available evacuation periods, expected water heights, flow regimes, etc. This information is needed for different possible scenarios. Usually most likely places of levee breaches are chosen and for those floods, disaster plans are made. In case a flood really occurs, the most fitting plan is chosen and adapted to accommodate the then current information demands.

Nature cares little about political boundaries, so the actions should also be communicated across political boundaries. Therefore good communication is needed between the different authorities. When local pumping station capacity proves insufficient, maybe neighbouring pumping capacity can be used. Also neighbouring information is
needed for evacuation plans. It is not useful to evacuate people to areas where the same flooding might occur, or maybe already has. In order to be able to communicate the authorities should be able to read and interpret the information provided by other authorities. With the increased use of computers the best way to interpret available information is to synchronise the information storing and transfer formats. This is best done by using the same tools for information gathering and processing. DELFT-FLS is being developed by WL | DELFT HYDRAULICS as a tool for simulating different types of scenarios.
4 Hydraulic defence works

4.1 Introduction

In this chapter an inventory will be made of different types of defence works and their failure mechanisms. First the chains of failure will be discussed, then the structural aspects of defence works and last breaching mechanisms.

4.2 Chains of failure

Every type of flood can be traced back to its incipient cause(s). Sometimes a failure is caused by a combination of incipient causes. The 1953 flooding disaster in the southern parts of the Netherlands e.g. was caused by a combination of a storm surge together with a normal spring tide and insufficient maintenance. Because of the rebuilding efforts of houses and businesses after the war, not enough attention was given to the maintenance of the dikes, which caused the dikes to have insufficient strength against the water.

Five conditions can be found that lay the foundations of failure. In table 4.1 the chains of failure are shown from the incipient cause down to the failure itself. This list may not be complete, but it does give an indication of the types of floods that are considered relevant for this research.

<table>
<thead>
<tr>
<th>Incipient cause</th>
<th>Direct cause</th>
<th>Consequences</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heavy precipitation or melting snow</td>
<td>High water levels in river</td>
<td>Flooding of flood plains</td>
</tr>
<tr>
<td></td>
<td>High water levels in reservoirs</td>
<td>Dike/ levee breaches Dike breaches</td>
</tr>
<tr>
<td>Storms</td>
<td>High water levels at sea</td>
<td>Dune/ dike breaches Flooding of delta's</td>
</tr>
<tr>
<td>Earthquakes</td>
<td>(at sea) tsunamis</td>
<td>Dunes/ dike/ coast overtopping/ collapsing</td>
</tr>
<tr>
<td></td>
<td>(on land) destabilisation of foundations</td>
<td></td>
</tr>
<tr>
<td>Construction faults or insufficient maintenance</td>
<td>Lower resistance to forces</td>
<td>Dam/ dike failure</td>
</tr>
<tr>
<td>Human actions</td>
<td>Damage to dams/ dikes/ levees and dunes</td>
<td>Breaching</td>
</tr>
</tbody>
</table>

Figure 4.1: Chains of failure leading to floods.
4.3 Dam types

Hydraulic defence works can be divided into three main types: Earth- or rockfill dikes and levees, concrete and masonry (reservoir) dams and dunes.

4.3.1 Earth- and rockfill dikes and levees

Dikes and levees are most commonly earthfill dams and rockfill dams, also called embankment dams. Embankment dams are used for river dikes and reservoir dams. It is estimated that 79 percent of the dams built in the United States of America are embankment dams. Almost all Dutch river defence works are of the embankment type. These dams and levees are made out of compacted embankment material. The difference between earthfill and rockfill dams, is the finer grained materials of which earthfill dams are made. Most embankment dams have an impervious core of clay, concrete or asphaltic concrete (Froehlich, 1990).

![Figure 4.2: Earth- or rockfill dam](image)

4.3.2 Concrete and masonry (reservoir) dams

Concrete and masonry dams come in four major varieties: Gravity, buttress, arch and arch-gravity dams (Froehlich, 1990).

The gravity dam derives its stability against overturning and sliding from its weight. It is usually made of blocks of masonry or unreinforced concrete, joined with mortar or flexible seals. A sloping slab of concrete resting on vertical buttresses is called a buttress dam. The water forces on the concrete slab are transferred to the foundation through the buttresses. If good lateral abutments are available an arch dam can be constructed. Arch dams can be recognised by their distinctive circle or ellipse shape. Water forces are transferred to the lateral abutments. The arch-gravity dam is a combination of the arch dam and the gravity dam. It transfers its loads to both the lateral abutments and the foundation. Typically, they are thin at the top and thick at the bottom.
4.3.3 Mountains

Mountains as hydraulic defence works come into view especially when reservoir dams are constructed. Not only the stability of the reservoir dam must be investigated, but also the stability and height of the surrounding mountains. Strength as a foundation to withstand the forces of the dam is a fairly obvious one. Stability of the side slopes inside the reservoir is also important. The slopes have been formed when they were dry. When the new reservoir fills the side slopes get submerged and when stability in not enough wholet parts of the earthen sides of the mountain can slide into the reservoir. The resulting wave can pose a threat to the reservoir dam.

The height of the surrounding mountains is important to make sure the reservoir does not spill water into surrounding valleys. To that end the permeability of the topsoil of the mountain has to be known.

4.3.4 Dunes

Dunes differ from the other dams in several ways. First, dunes are mostly formed by nature. Secondly, dunes are dynamically stable contrary to the statically stable dams, like the ones discussed before. During the winter large parts of the outer slope are eroded by storms. This does not pose a threat to the hinterlands as long as a minimum dune depth exists. The material is deposited on the near shore. During the summer most of this material is deposited back into the dunes, making them strong enough for the next winter.
Depending on various factors, coasts can be dynamically stable, progressive or regressive. If a coast is dynamically regressive, it may be necessary to add beach material every now and then to make sure the ongoing erosion does not threaten the hinterlands. Another (better) way is to take away the cause of the erosion, however this is often not possible, because this influences neighbouring coastal areas.

Dunes consist of non-cohesive, fine-grained sediment. They can resist wave attacks and overtopping if they are overgrown with plants like marramgrass, which binds the grains with their roots.

Figure 4.7: Indiana Dunes National Lakeshore, Indiana, with Marramgrass, USA © David Muench, 1981

4.4 Dam classification

Singh (1996) classifies reservoir dams by size and hazard potential. The (United States based) size classification depends on height and storage capacity.

<table>
<thead>
<tr>
<th>Size</th>
<th>Capacity ($10^6 m^3$)</th>
<th>Height (m)</th>
<th>Stored Potential Energy ($10^9 J$) (calculated with triangle-shaped reservoir $\nabla$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small</td>
<td>&lt; 1</td>
<td>&lt; 8</td>
<td>&lt; 52</td>
</tr>
<tr>
<td>Medium</td>
<td>1 - 3</td>
<td>8 - 15</td>
<td>52 - 294</td>
</tr>
<tr>
<td>Large</td>
<td>3 - 20</td>
<td>15 - 30</td>
<td>294 - 3920</td>
</tr>
<tr>
<td>Major</td>
<td>&gt; 20</td>
<td>&gt; 30</td>
<td>&gt; 3920</td>
</tr>
</tbody>
</table>

Table 4-1: Dam size classification.

The hazard potential after dam failure is based on an assessment of the risk to life and economic losses.

<table>
<thead>
<tr>
<th>Hazard potential</th>
<th>Loss of life</th>
<th>Economic Loss</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low</td>
<td>Impossible</td>
<td>Minimal</td>
</tr>
<tr>
<td>Low</td>
<td>Improbable</td>
<td>Marginal</td>
</tr>
<tr>
<td>Moderate</td>
<td>Possible</td>
<td>Appreciable</td>
</tr>
<tr>
<td>High</td>
<td>Probable</td>
<td>Excessive</td>
</tr>
</tbody>
</table>

Table 4-2: Dam hazard potential classification.
Singh (1996) combines these two to give the dam class.

<table>
<thead>
<tr>
<th>Hazard \ Size</th>
<th>Small</th>
<th>Medium</th>
<th>Large</th>
<th>Major</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Low</td>
<td>4</td>
<td>3</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Low</td>
<td>3</td>
<td>2</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Moderate</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>High</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 4-3: Dam classification by size and hazard potential.

These dam classes can then be used to formulate return periods for design flood and peak flood. These design discharges are calculated statistically. This will be different for most countries as they give different importance to life and economical losses. Table 4-4 gives sample return periods for the dam classes from Singh (1996). A ‘normal flood’ consists of a flood with normal dry-free board allowance. The ‘peak flood’ is the design flood which occurs without overtopping the dam.

<table>
<thead>
<tr>
<th>Class</th>
<th>Normal flood (year)</th>
<th>Peak flood (year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2,000</td>
<td>10,000</td>
</tr>
<tr>
<td>2</td>
<td>500</td>
<td>2,000</td>
</tr>
<tr>
<td>3</td>
<td>250</td>
<td>750</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>250</td>
</tr>
</tbody>
</table>

Table 4-4: Return periods per dam class.

The data in these tables are constructed for dams in the United States of America and is only one way of classifying (reservoir-) dams. One can take into account the probability of failure of the dam as well. For different types of dams, like levees, of course different criteria have to be taken into account. In The Netherlands, where design- and peak floods are primarily dimensioned for rivers and levees, the values are different. For the Dutch coast, the normative values lie between 2,000 and 10,000 years, the rivers are designed with return periods of 1250 years.

4.5 Dam failure

Failure modes are based on the properties of the materials a hydraulic defence work is constructed of and the way forces are handled. Human actions, like sabotage, war and research related influences, can cause all hydraulic defence works to fail.

4.5.1 Earth- and rockfill dams and levees

Earth- and rockfill dams and levees have four basic modes of failure: overtopping, piping, sliding and flow erosion. All four modes cause failure due to erosion of embankment material (Froehlich, 1990, Singh, 1996).

Overtopping occurs when water levels rise above the crest of the dam. This is usually caused by inadequate spillway discharge capacity or a crest height which is too low. The stream that flows over the crest of the dam erodes material from the, mostly, less protected inner slope. At a weak point a breach will form and more material will be eroded from this
breach, thus causing failure of the dam. The speed at which the breach grows depends on the erodibility of the embankment material.

**Piping** can occur when a hydraulic head difference over the dam is present. When the dam is not impervious enough, seepage of water through the dam can happen. This in itself is not a problem, it becomes a problem when the forces in the porous flow becomes larger than the resistance against erosion due to the weight of the material. Material will then be transported and conduits or pipes can be created. If a pipe is large enough, it will collapse and the crest of the dam will settle. A breach can form and cause failure. Piping can also be started by animals like muskrats. Singh (1996) estimates that more than 60 percent of the earth dam failures are caused by overtopping or piping.

If for long periods of time high water levels occur on the upstream side of a dam, high pore pressures in saturated parts can lower the stability of the dam, thus causing **sliding** of the up- or downstream side of the dam. This will then cause the crest to settle or breach, which is usually followed by failure. Another cause of sliding due to instability is rapid drawdown of the reservoir water level, so that the water inside the dam cannot follow quick enough, causing high pore pressures. High pore pressures can cause the sand skeleton to lose stability, after which the dam can fail. Earthquakes, causing abrupt movements in the soil can ‘liquefy’ the embankment material.

### 4.5.2 Concrete and masonry (reservoir) dams

Gravity dams mostly fail due to **overtopping** or **sliding**. Foundations and the mortar joints can deteriorate in time and create increased internal upward forces through seepage of water. Thus the stability of the dam is lowered. Especially older mortar joints, made in times when lime mortar was still popular, can deteriorate when in contact with water for longer periods, thus loosening the bonds between the masonry blocks.

Buttress dams fail mostly due to excessive forces on the footings of the buttresses, **crushing the foundation** and lowering the stability of the dam against sliding and overturning.

Arch dam failure can be caused by three modes: **Overtopping**, **movement of the abutments**, thus creating large internal stresses in the concrete, which will then crack, and cause the **abutments to devolve** completely.

Most concrete dams can survive overtopping for a long time. Masonry dams are more susceptible to erosion due to overtopping. The downstream foundation though can be eroded, thus causing failure of the dam due to toppling or sliding into the scour hole.

### 4.5.3 Dunes

The main mode of failure for dunes is **erosion**. This can happen in a long or short duration. If a dune erodes heavily during winter and can not recuperate during the summer, the coastline will regress slowly over the years until the hinterland is threatened by the sea. If a storm is long and severe enough and the dune depth is not enough, a breach can erode
into the dunes which effectively fails at that point. The sea can then rush into the hinterlands and flood the plains. The two modes of attack on a dune are shown in figure 4.8, with the solid line the position of the coast and the dotted line the location of the foremost economic activity. $R$ is the area that should be preserved against economic activities. During a storm dune depth $r$ is eroded. After the storm a dynamically stable coast will slowly recupeate from the attack and the original coastline will be re-established. The regressing coast in figure 4.8b is a danger to all economic activities near the coast and action should be taken against this situation.

![Diagram](image)

Figure 4.8: Position of 'the coastline' on a) a dynamically stable coast and b) a dynamically unstable coast according to Velden (1995)

### 4.5.4 Classification of dam failure

The main failure modes for different hydraulic defence works can be put together in a table. Some modes are more common than others. These are denoted xx as opposed to the less common modes, which are denoted with a single x.

<table>
<thead>
<tr>
<th>Dam type</th>
<th>Failure mode</th>
<th>Piping</th>
<th>Overtopping</th>
<th>Sliding</th>
<th>Overturning</th>
<th>Foundation failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earth fill</td>
<td>xx</td>
<td>xx</td>
<td>x</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Rock fill</td>
<td>xx</td>
<td>xx</td>
<td>x</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Gravity</td>
<td>xx</td>
<td>xx</td>
<td>xx</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Buttress</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arch</td>
<td>xx</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td>xx</td>
</tr>
<tr>
<td>Arch-gravity</td>
<td>xx</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td>xx</td>
</tr>
<tr>
<td>Dunes</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>x</td>
</tr>
</tbody>
</table>

Table 4-5: Classification of dam failure.
4.6 Breach development

When concrete or masonry dams fail, failure is quick. The breach is usually trapezoid in shape with practically vertical sides. Due to the nature of the material, the whole dam usually topples into the stream in a matter of minutes. For simulation purposes it is common practice to assume the breach to develop rapidly and the size of the breach to be equal to the entire dam in case of an arch dam. In case of a gravity dam the size of the breach is mostly assumed to be a part of the dam.

Initial triangular breach

Figure 4.9: Shape of breach during development.

An embankment dam will almost always fail with initially a triangular breach. This breach will grow laterally, with side slopes of about 2V:1H (see figure 4.9), depending on the flow through the breach. This is found to depend on the height of the water level above the crest of the breach. When a less erodible material layer is encountered, mostly the original foundation, the breach will keep growing laterally, but will slow or stop growing downward, thus forming a trapezoid breach shape.

The process of failing can be divided into five stages. According to Visser (1998) these stages are:

Figure 4.10: The five stages of breach development according to Visser (1998).

I: $t_0 < t < t_1$; Steeping of the channel of the inner slope. Due to gravitational forces the flow velocity and sediment transport capacity increase downward. The erosion at the toe of the slope is larger than at the top. This phase continues until at $t = t_1$ the slope angle becomes critical.
II: $t_1 < t < t_2$: The flow continues to erode the inner slope, thus decreasing the width of the crest. The sides also develop laterally, widening the breach. The inner slope angle stays critical.

III: $t_2 < t < t_3$: Continuing erosion causes the top of the dike to lower. The breach gets larger and so does the discharge through the breach. This phase continues until at $t = t_3$ the dike in the breach is completely washed away.

IV: $t_3 < t < t_4$: The dike in the breach is completely gone. The breach continues to grow laterally and vertically (scour hole). The downstream water level rises, until the flow through the breach shifts from supercritical ($Fr > 1$) to subcritical ($Fr < 1$). This happens at $t = t_4$, with the flow being critical ($Fr = 1$) at $t = t_4$. In small polders with a small water-surface area the transition from supercritical to subcritical can occur for $t < t_4$.

V: $t_4 < t < t_5$: The breach continues to grow laterally, until at $t = t_5$ the flow velocity becomes too small to be able to transport sediment (criterion of incipient motion).

The erosion process is repetitive. Erosion always starts around a less erodible element, until this element stands free into the flow and collapses. The flow will then erode sediment around another less erodible element, and so on. The increase of the volume of sediment eroded is almost linear in time, only towards the end of the erosion process the curve tends to flatten. This is caused by a decreasing discharge (Singh, 1996).

Downstream of a hydraulic defence work scour holes can form, when the bed is not protected properly against erosion. Scour holes can pose a threat to a hydraulic defence work by eroding the foundation downstream of that defence work. The defence work can then topple or slide into the scour hole.

Development of a scourhole can be seen as a special case of sediment transport. The depth of the scourhole was described with the Breusers-equation in v. Aalst et al. (1987).

$$h_{sm}(t) = \frac{(\alpha \cdot u_e - u_i)^{1.7} \cdot h_w^{0.2}}{10 \cdot \Delta^{0.7}} \cdot t^{0.4}$$

with:
- $h_{sm}(t)$: Maximum scourhole depth [m]
- $h_w$: Original water depth [m]
- $t$: Time [hr]
- $u_e$: Critical velocity of incipient motion [m/s]
- $u_i$: Vertically averaged velocity at time $t$ [m/s]
- $\alpha$: Dimensionless velocity amplification factor [-]
- $\Delta$: Relative density $(\rho_i - \rho_w)/\rho_w$ [-]
- 10: Constant [m·hr/s]

However, during experiments (Caan, 1996) it was found that the scourhole became much deeper than was found using the Breusers-equation, with $\alpha = 3$. He found it possibly also to be dependent on the packing of the sediment. In the experiment with the highest porosity he found the lowest scourhole-depth. This parameter is not included into the Breusers-equation. He suggested more research into the phenomena, as he could not determine an exact relation.
5 Flow after hydraulic defence work failure

5.1 Introduction

The development of a breach is largely dependent on often unknown structural properties, like packing and erodibility of the building materials. The initial width of the breach is often not known, because the period of high water e.g. is not known beforehand. For this sort of unknown properties a sensitivity analysis will have to be done.

For correct flooding simulation it is more important to know the properties of flow. The results of simulation depend both on the correct simplification and discretisation of the physical world and on the input of boundary conditions. To describe the governing influence on the simulation result, input data or numerical description, the hydrodynamic aspects of flow in different phases of flooding have to be known. In this chapter the hydrodynamic aspects will be discussed.

First hydrodynamic aspects of flow will be discussed. An inventory will be made of how these hydrodynamic aspects are influenced by numerical description on the one hand and by input data on the other hand. Then these aspects will be applied to the different phases of flooding.

5.2 Hydrodynamic aspects of flow

Hydrodynamics is the study of the (averaged) movement of fluids caused by external forces. These movements can be described by applying balances over a well chosen volume for properties, like mass, momentum or energy. In 2 dimensional (2Dh) hydrodynamics some simplifications are usually made:

- The pressure distribution is supposed to be hydrostatic,
- Every property is considered averaged over the depth.
- The gravitational acceleration is constant.
- The fluid is Newtonian, with constant density ($\rho$) and viscosity ($\nu$).
- Within small areas Coriolis forces are neglected.

With these simplifications equations are derived from those balances to describe the flow. A set of equations, which is often used, and also in this research project, is called the depth interpolated Saint-Venant equations. In a two dimensional model it consists of a continuity equation and two momentum equations, one in x- and one in y-direction. The equations are:

$$\frac{\partial \xi}{\partial t} + \frac{\partial (uh)}{\partial x} + \frac{\partial (vh)}{\partial y} = 0 \quad (2)$$
\[
\frac{\partial (uh)}{\partial t} + \frac{\partial (hu^2)}{\partial x} + \frac{\partial (vuh)}{\partial y} + gh \frac{\partial \zeta}{\partial x} + C_f u \|u\| = 0 \\
\frac{\partial (vh)}{\partial t} + \frac{\partial (vuh)}{\partial x} + \frac{\partial (hv^2)}{\partial y} + gh \frac{\partial \zeta}{\partial y} + C_f v \|u\| = 0 \\
\|u\| = \sqrt{u^2 + v^2}
\] (3)

with \( \zeta \) : the water level [m],
\( u, v \) : the velocities in X-, respectively Y-direction [m/s],
\( h \) : The water depth [m],
\( g \) : gravitational constant [m/s²],
\( C_f \) : Friction coefficient [-].

The momentum equation consists of four dynamic terms which balance each other:

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{\partial (uh)}{\partial t} )</td>
<td>( \frac{\partial (hu^2)}{\partial x} + \frac{\partial (vuh)}{\partial y} )</td>
<td>( gh \frac{\partial \zeta}{\partial x} )</td>
<td>( C_f u |u| )</td>
</tr>
</tbody>
</table>

The Rate of change of momentum (RoC), the Transport of momentum (ToM) and the Integrated normal hydrostatic pressure (Pres) are highly dependent of the way they are described numerically. The Bottom Friction (Fric) is more directly dependent on the input data. The abbreviations between the brackets match those in the figures.

When large gradients of the water surface \( (\Delta \zeta / \Delta x) \) exist the momentum equation will lead to a balance between [1], [2] and [3]. The bottom friction will have relatively little influence. However, when the gradients in the flow are small, with over land flow e.g., the relative influence of the bottom friction [4] will increase, and will balance mainly [3]. This consideration could lead to ‘diffusive wave approximation’
The relative influence of the different terms can be shown with a simple numerical experiment. A dam-break situation with upstream water level boundary was simulated in 1Dh. One cell equals ten metres in the x-direction. The channel was chosen one hundred cells long, or 1000 metres. The dam was located at 10 m, or 1 cell, from the left boundary. The upstream water level was set to 1.0 m and kept at that level during the entire
simulation. In the first (uppermost) graph both the water level and the velocity was plotted. The water level values can be found on the left axis and the velocity values on the right axis. The second (middle) graph shows the influence of each term in the momentum equation as a percentage of the sum of the absolute values of the terms. This relative influence was calculated per cell. In the third (lowermost) graph the values of each term of the momentum equation are shown.

Two areas can be discerned in the graph of the different terms. They are denoted [I] and [II] (see figure 5.1). Some conclusions can be drawn from the graphs:

- At the front of the bore (area [I]) that forms when bottom friction is present, the flow is very dynamic, with large gradients of the water surface. Here the rate of change of momentum and the transport of momentum have relatively large influence. Friction can be effectively be neglected. The small friction peak at the very front of the flow exists due to the very shallow water depths at the front of the flow.

- Behind the bore the flow very quickly becomes dominated by the friction and the pressure gradient. It can be seen that the relative influence the rate of change of momentum and the transport of momentum becomes very small.

- From the third graph it can clearly be seen that the absolute values of the properties become very small compared with those at the front of the flow in area [I]. The dynamic processes of a dam-break flow prove very important in comparison to the normal flow.

### 5.3 Initial flow

Lauber (1997) did some laboratory experiments on front propagation after a dam break. A channel set-up was used with a width of 50 cm, a depth of 70 cm and a length of 14 m. At 4 m. downstream of the channel a gate was located that could be lifted fast enough to prevent effects of opening time on the dam break wave. Close examination of the video-captured dam break flow revealed two positive waves; an initial wave an a dynamic wave.

Initial flow occurs during the first moments of the breach process, the time it takes a particle from the free surface to reach the channel bottom. During this period the shallow water equations are not valid as the effects of streamline curvature are dominant. The wave propagates with velocity \( v_i = \sqrt{\frac{10}{9} gh_0} \). The initial phase has a duration of \( t = \sqrt{2h_0/g} \) seconds.

The dynamic wave propagates with velocity \( v_d = 2\sqrt{gh_0} \), which is faster than the initial wave velocity. After a short time the dynamic wave overtakes the initial wave. From this moment the shallow water equations can be used.

Stansby et al. (1998) did some experiments on the initial stages of dam-break flow. They found that a horizontal jet formed during this initial phase. This jet cannot be simulated using the Saint-Venant relations for shallow water flow. Only when the assumption of a hydrostatic pressure distribution is dropped this horizontal jet can be reproduced in simulations. It was found however that the results from 1D computations using the Saint-Venant equations, effectively excluding the initial flow, gave qualitatively highly similar
results to the measurements for times beyond the initial flow. Only for the very first instances the differences were significant. This corresponds to the findings of Lauber, that for the first instants the hydrostatic pressure simplification does not apply.

Because of the small time period involved and the fact that a dam will never breach 'perfect' - in a negligible time- initial flow will not exist like in a perfect breach. The first moments the flow is not free, like in a perfect breach, but still influenced by the breaking dam. It is thus assumed acceptable to use the shallow water equations with an instantaneous breaking of the dam. When the time, needed for the breaking of the dam, is significant, this breaking process should of course also be included in the calculation.

5.4 Near field flow

The near field flow is characterised by a small space scale and is dominated by inertial and pressure forces. This results in rapid variations in flow patterns. The flow can be mathematically described by shallow water equations.

Special attention should be given to the propagation of the waterfront over dry bed and wet bed (Gallati, 1988; Braschi, 1988). Braschi did some physical model tests to verify his model. He found a distinct difference between near field flow over the dry and the wet bed. The first has the shape of a pancake, flowing evenly to all directions. The latter has a distinctive wave front and progresses as a bore.

![Figure 5.3: Front shape flow (Stelling, 1998)](image)

5.5 Sheet flow/ overland flow

Secondly the 'far field' flow occurs. This flow occurs further away from the breach and is influenced by characteristics of the area. These are:

1. Geometry (slopes, sizes)
2. Land use (vegetation, buildings, streets, railroads, ditches)
3. Soil saturation (infiltration capacity)
4. Soil build-up (defined by the geological history)

These characteristics influence flow in different ways. Geometry mainly gives some conditions about the direction a flood will flow. Land use influences mostly the resistance a flood encounters. As simulations are typically done on 25x 25 m² to 100x100 m² grid size,
different types of land use, like agriculture, bushes and forests, can be considered as bottom friction. Soil build-up and saturation influence the capacity of the topsoil to drain and store water from the flood.

Several ways are known to model bottom-roughness. Shear stress on the bed induced by a 2D turbulent flow can be described by:

$$\tau_b = \rho C_f u |u|$$  \hspace{1cm} (5)

with $\rho$ : The mass density [kg/m$^3$],
$u$ : The velocity [m/s],
$C_f$ : A (bottom-) friction constant [-].

Bottom friction, however, is not the only friction term. Also so called sub-grid effects are usually put into the friction term. A sub-grid effect is an effect that has influence on the flow, but cannot be implemented directly due to resolution restrictions. An example of a sub-grid effect is the flow around and between bridge piers. Here not only bottom friction is present, but also a drag force, contraction of the flow and turbulence due to the mixing of the flow behind the piers. These flow phenomena mostly have a fairly small size compared the sides of the grid cells. Some integrated description can be inserted into the friction term to cope with these sub-grid effects.

When a flow propagates over initially dry plains water can infiltrate into the topsoil, depending on the degree of saturation $S$ [-] of that topsoil. $S$ is the ratio between the volume of water in the topsoil $V_{water}$ and the pore volume $V_{pores}$. This can be of importance when simulating flooding of initially dry plains.

Infiltration intensity $f$ [LT$^{-1}$] is mostly denoted as a speed. More precisely it is a volume that infiltrates a unit area in unit time [(L$^3$T$^{-1}$)L$^{-2}$] which is shortened to [LT$^{-1}$]. The Green-Ampt model describes infiltration with a sharp moisture front. The infiltration intensity can be written as:

$$f = -K(S) \frac{\psi_f - (z_f + H)}{z_f}$$  \hspace{1cm} (6)

At the moisture front the pressure head is $\psi_f$ when the depth of the front is taken as reference level. $z_f$ is the increasing length over which water has already infiltrated. $H$ is the water level above ground level and $K(S)$ is the permeability of the topsoil as a function of the saturation $S$.

![Figure 5.4: Sharp moisture front](image-url)
Thus, when a sharp moisture front is applicable, like in sandy bottoms, the infiltration intensity can be calculated. The permeability is dependent on the, often uncertain, saturation of the topsoil. For different sediment types the order of permeability for fully saturated soil is given in the following table (Akker, 1995):

<table>
<thead>
<tr>
<th>Sediment type</th>
<th>Permeability [m/day]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>$10^{-7} - 10^{-4}$</td>
</tr>
<tr>
<td>Silt</td>
<td>$10^{-4} - 10^{-1}$</td>
</tr>
<tr>
<td>Sand</td>
<td>$10^{-1} - 10^{3}$</td>
</tr>
<tr>
<td>Sandstone</td>
<td>$10^{5} - 10^{0}$</td>
</tr>
<tr>
<td>Igneous rock</td>
<td></td>
</tr>
<tr>
<td>with cracks</td>
<td>$10^{-5} - 10^{-1}$</td>
</tr>
<tr>
<td>without cracks</td>
<td>$10^{9} - 10^{5}$</td>
</tr>
</tbody>
</table>

Table 5-6: Indication of permeability for different sediment types.

How does this relate to the characteristics of a flood? Two important characteristics of a flood are the water level and the arrival times at certain points. Infiltration is a slow process, in the order of millimetres to metres per day, compared to the speed a flood propagates with, in the order of metres per second. Propagation speed will not be influenced significantly.

This also goes for the water levels. With a porosity of around 40% and a saturation ranging from 0 to 90%, the storage volume of a m$^3$ top soil ranges from 0.4 - 0.04 m$^3$. In the Netherlands ground water levels are kept at 0.1 to 1 metres under ground level. This leads to a volume of water stored in the topsoil in the order of $10^{-1} - 10^{3}$ m$^3$ per m$^2$ flooded area. The water levels can drop in the order of decimetres in very favourable conditions (low GWL, low saturation, high porosity) to negligible in worst case conditions (high GWL, high saturation, low porosity). In average the water level drop will be in the order of centimetres, highly dependent on:

- Ground water levels. The location of the ground water levels can give an indication of the total volume of water that can be stored in the top soil. In The Netherlands these data are pretty well known by polder boards;
- Saturation of the topsoil. Saturation can be deducted to a certain level with, e.g. the infiltration equation of Horton (1939), which takes into account precipitation intensity and saturation history;
- Build-up of the topsoil. This gives an indication of the porosity and permeability of the topsoil.

Infiltration lowers the end water levels of a flooding. The actual value of this drop however, is highly dependent on terrain conditions. Because omitting infiltration will only lead to a conservative (i.e. higher) estimation of the end water level, it should only be incorporated into simulations if the variables are well known and have significant effects. Then infiltration can be inserted into the simulation by adding a sink term to the continuity equation to draw mass from the flow.
6 Description of DELFT-FLS

6.1 Introduction

Verification of the correct implementation of a numerical model can be done in different ways, on different parts of the simulation program. During the research it was found that few accurate data are available on floods. Most data are compiled after the water has receded, from watermarks on houses and victim stories. Sometimes the location of the waterfront can be given more precisely. When an electricity substation or telephone station is submerged, it mostly shuts down, and such events are usually accurately logged. This produces data on specific times, though usually not on water levels or velocities.

Experimental data are more widely available. Many researchers have done experiments to validate their hydraulic models. These experiments can be used very well for the validation of the numerical principles, used in the simulation model. Proper mass or energy conservation is very important and this does not change with scale.

At the beginning of the research project, DELFT-FLS was found not capable of simulating the tests correctly. When a polder flooding was simulated the flow looked correct. When a more detailed look was taken to flow near a breach, the flow velocity proved to be too high and the front of the flow was not represented correctly. Also the instantaneous breaking of a dam was not simulated correctly. The breaking process was divided over at least two time steps, so when only one time step was available, a part of the dam would remain standing. A smaller issue was the restriction that the computation time step could only be input in hours as opposed to seconds e.g.

DELFT-FLS is still under development and some variables had been programmed rigidly into the code, instead of changeable through input variables. It proved necessary to change parts of the code. The official DELFT-FLS code is managed by another department at WL | DELFT HYDRAULICS and because they could not make the necessary changes to the code within the time frame of this project it was decided to take out part of the code, the calculation core. The calculation core was reprogrammed to the requirements of the research project. This led to a specific program, that could easily be adapted to further needs in simulation capabilities. The code, resulting from this research project, can be re-implemented in the DELFT-FLS code after this project. After the most obvious bugs had been corrected the verification process could continue. When simulating one of the test cases, which are described in the following chapters, a major bug was found. As this bug and its solution is interesting for this project it will be discussed together with that test case.

The next section (section 6.2) the DELFT-FLS model will be discussed with the equations that were implemented in the computer code. The selection of the test cases that were to be used as verification case for DELFT-FLS will be discussed in section 6.3.
6.2 The DELFT-FLS model

DELFT-FLS is being developed as a 2Dh model to simulate flooding in The Netherlands. It is intended to be a fast, easy to use simulation tool. To that end the grid, on which the computations are made has been chosen rectangular. This makes it very easy for a user to incorporate information from geographical information systems (GIS's) into DELFT-FLS. Geographical information systems mostly store and represent their data on rectangular grids. Therefore the step of generating a (curvilinear) grid and adapting the data to that grid can be discarded. The same simple data transfer applies after the simulation. Data from the simulation program can easily be imported into off-the-shelf post-processing software.

With a so called master definition file (*.mdf) all the different options of DELFT-FLS can be described. This can be done by describing the input of the simulation, like bottom profiles and initial conditions. Run-time options can be given like time step, start and end time, and the breaking of dams, precipitation, and boundary conditions. Output can be requested at specific times, locations and/or with specific intervals.

For the calculation core of DELFT-FLS the following continuity and depth-integrated shallow water equations in two dimensions are used:

\[
\frac{\partial \zeta}{\partial t} + \frac{\partial (uh)}{\partial x} + \frac{\partial (vh)}{\partial y} = 0
\]  
\[\frac{\partial (uh)}{\partial t} + \frac{\partial (hu^2)}{\partial x} + \frac{\partial (vuh)}{\partial y} + gh \frac{\partial \zeta}{\partial x} + g \frac{\parallel u \parallel}{C^2} = 0 \tag{2} \]

\[
\frac{\partial (vh)}{\partial t} + \frac{\partial (vuh)}{\partial x} + \frac{\partial (hv^2)}{\partial y} + gh \frac{\partial \zeta}{\partial y} + g \frac{\parallel v \parallel}{C^2} = 0
\]  
\[\parallel u \parallel = \sqrt{u^2 + v^2} \]

with

- \( \zeta \) water level above reference \([\text{m}]\),
- \( d \) water level below reference \([\text{m}]\),
- \( h \) Total water depth, \( h = \zeta + d \) \([\text{m}]\),
- \( u \) velocity in x-direction \([\text{m/s}]\),
- \( v \) velocity in y-direction \([\text{m/s}]\),
- \( C \) Chezy coefficient \([\text{m}^{1/2}/\text{s}]\) and
- \( g \) gravitational acceleration term \([\text{m/s}^2}]\).

For more detailed information on the discretisation for flow over large gradients see appendix A.
As can be seen from equations 8 and 9, the shear stress for depth-averaged flow in DELFT-FLS is assumed to be given by the quadratic friction law:

\[ \tau_b = \frac{1}{2} \frac{\nu |u|}{C^2} \]  

(4)

The Chézy smoothness value can be described in DELFT-FLS with the following three formulations:

- Chézy: \( C = \text{Chézy coefficient [m}^{1/2}\text{/s]} \)

- Manning: \( C = \frac{\sqrt{H}}{n} \)

where

\( H \) is the total water depth [m]
\( n \) is the Manning coefficient [s/m\(^{1/3}\)]

- White Colebrook: \( C = 18 \log_{10} \left( \frac{12H}{k_s} \right) \)

where

\( H \) is the total water depth [m]
\( k_s \) is the Nikuradse roughness length [m]
18 is a constant [m\(^{1/2}\)/s]

For some materials and bottom covers values for the Manning coefficient \( n \) and Nikuradse roughness length \( k_s \) are given.

<table>
<thead>
<tr>
<th>Material characteristics of the area</th>
<th>Manning roughness value ( n ) [sm(^{-1/3})]</th>
<th>Nikuradse roughness length ( k_s ) [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PVC, Plexiglas</td>
<td>0.01</td>
<td>3·10(^{-4})</td>
</tr>
<tr>
<td>Glass</td>
<td>0.01</td>
<td>3·10(^{-4})</td>
</tr>
<tr>
<td>Lucite</td>
<td>0.009</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>0.010–0.013</td>
<td>7·10(^{-4})–2·10(^{-2})</td>
</tr>
<tr>
<td>Masonry</td>
<td>0.025</td>
<td>6·10(^{-3})–7·10(^{-2})</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.020–0.030</td>
<td></td>
</tr>
<tr>
<td>Cobbles with large boulders</td>
<td>0.050</td>
<td></td>
</tr>
<tr>
<td>Natural waterways:</td>
<td></td>
<td>4·10(^{-2})–0.15</td>
</tr>
<tr>
<td>Straight, well maintained</td>
<td>0.025–0.030</td>
<td></td>
</tr>
<tr>
<td>Winding, well maintained</td>
<td>0.035–0.040</td>
<td></td>
</tr>
<tr>
<td>Winding, with vegetation</td>
<td>0.040–0.050</td>
<td></td>
</tr>
<tr>
<td>With stones and vegetation</td>
<td>0.050–0.060</td>
<td></td>
</tr>
<tr>
<td>Pastures, meadows</td>
<td>0.035</td>
<td>3·10(^{-2})–4·10(^{-2})</td>
</tr>
<tr>
<td>Agriculture</td>
<td>0.040</td>
<td></td>
</tr>
<tr>
<td>Shrub, bushes</td>
<td>0.050</td>
<td>0.4</td>
</tr>
<tr>
<td>Dense shrubs</td>
<td>0.070</td>
<td></td>
</tr>
<tr>
<td>Dense forest</td>
<td>0.100</td>
<td></td>
</tr>
</tbody>
</table>

6.3 Selection of the verification test cases

After the classification was completed the verification of DELFT-FLS was started. First relevant test cases had to be found. As DELFT-FLS is being developed as a tools to simulate floods in The Netherlands it was preferred to use data from levee breaches, with flooding of polders. It was found however that little accurate data was available. Some detailed measurements were done on the breaching process of levees, but no measurements were found of the polders behind the levee.

However, a European Commission funded program, called CADAM or Concerted Action on Dam-break modelling, has been under way since February 1998. Some detailed experiments were done on the flow after a reservoir dam breach. Even though these experiments do not represent a polder flood, they do represent the dynamic part of flow very well. In chapter 5 it was found that dynamic flow is governed by a balance between the rate of change of momentum, the transport of momentum and the pressure term. As this is the most difficult to implement correctly into computer code, the CADAM data could be used to verify the correct workings of DELFT-FLS. Sheet flow is hardly present in the CADAM experiments so that verification on this part of flow still had to be done.

Another experiment was chosen as a basic test of the two-dimensional capabilities of DELFT-FLS. The question asked was if the results of simulation were dependent on the direction of the flow compared to the direction of the grid. The best way to test this was to construct a channel parallel and diagonal through the grid (see figure 6.1).

![Diagram](image)

Figure 6.1: Top view of channel oriented a) parallel and b) diagonal to the grid. Thick lines represent walls, grey area is the flow area.

The results of the simulations were to be compared to an analytical exact one-dimensional solution. As one-dimensional flow is equal to two-dimensional flow with unit width, this flow could be simulated by a prismatic channel with frictionless closed side boundaries.

Finally it was decided to focus on two experiments:
1. One-dimensional dam-break flow simulated on a 2Dh grid. This test case will be described in chapter 7.
2. The CADAM test case with a 45° channel bend. This test case will be described in chapter 8.
7 1Dh test cases

7.1 Introduction

The 1Dh test-case is a basic test. The 1D part is implemented by simulating a prismatic channel of several cells wide with frictionless side boundaries. Flow direction, flow velocity and water levels should be exactly the same over any orthogonal cross-section of the channel.

7.2 Set-up of the test cases

1-Dimensional dam-break flow simulated on a 2Dh grid is done by designing a channel with constant width of a few cells. This test case is divided into two parts: 1Dh dam-break flow over an initially dry bed and over an initially wet bed. For both parts a comparison was made between a parallel case and a diagonal case. With the diagonal case the channel lies diagonally through the grid, while with the parallel case the channel lies parallel to the x-axis.

![Diagram showing channel orientations](image)

Figure 7.1: Top view of channel oriented a) parallel and b) diagonal to the grid. Thick line represent walls, grey area is the flow area.

Verification of the results produced with DELFT-FLS has been done with respect to two criteria. The first criterion is a comparison with an analytical solution of the case (1-dimensional). The second is on (wall-) boundary handling. For accurate 1Dh flow, wall friction of the channel in the 2Dh model was assumed to be non-existent. Therefore the front of the flow should be straight and perpendicular to the walls of the channel.

In a prismatic channel a dam was constructed. The test consisted of removing the dam at \( t = 0 \) in negligible time. From that moment the water will flow. This flow is followed in time. Because an analytical exact solution for this problem -based on the method of characteristics- is known, good verifications can be made of the results produced with DELFT-FLS.
The channel is chosen with a length of 500 m, divided into 100 cells of 5 m. The dam was located in the middle, at $x = 250$ m. The reference level is taken at the channel bottom. The reservoir level is 1.0 m. The downstream water level is considered 0.0 m for the dry bed case and 0.1 m for the wet bed case. Wall and bottom friction are neglected. At $t = 0$ the dam is removed and the water level is monitored during outflow of the reservoir for 30 s.

![Diagram of channel design]

Figure 7.2: 1D channel design. The second dimension of DELFT-FLS is perpendicular to the page.

This case was designed to test the following:
1. Correct propagation of the front and back of the wave,
2. Correct wave front shape both parallel and diagonal to the grid.

The first property is a basic implementation test of the model. If this test is not passed, the model used in the software is incorrect or implementation errors (bugs) have surfaced.

The second property is a good test for flow diagonally through the grid. When a cell is completely surrounded by active cells no problem will be met. However, (wall-) boundary cells could prove to be a problem. As DELFT-FLS calculates on a rectangular grid, no diagonal boundaries can be defined. The boundary will always look stepped.

![Diagram of grid with boundaries]

Figure 7.3: Top view of grid: Real versus simulated boundary

This could introduce boundary effects, which will influence the results in other cells and thus the general outcome of the simulation. A good implementation of the physical world will result in (almost) identical results for the parallel tests and the diagonal tests.
Two DELFT-FLS simulations were performed and compared to the analytical exact solution:

1. **The parallel test.** A prismatic channel was designed parallel to the rows on a rectangular grid (see figure 6.1a) with sides of 5 m. The total length of the channel was 500 m and the dam was a row of cells at column 51. When the dam was removed, the wall of water would then have its boundary exactly at \( x = 250 \) m. The channel was chosen 10 cells wide, or 50 m. This way a good estimate of the shape of any cross-section of the channel can be made.

2. **The diagonal test.** A prismatic channel was defined diagonally through the rectangular grid (see figure 6.1b). The sides of the grid were set to 5 m. This way the diagonal would be \( 5m \cdot \sqrt{2} \approx 7.07m \), leading to a 7 cell wide channel. The length of the channel was designed at 70 cells.

### 7.3 The analytical exact solution

For a good comparison of the two test cases, the analytical exact solution had to be calculated first. The results of these calculations have been stored over a period of 30 s with 5 second intervals. This was done for both the dry bed case and the wet bed case. To give an indication of the shape of the water level, results are given in figure 7.3a and figure 7.4a for \( t = 0, 10, 20 \) and 30 s. From figure 7.3b it can clearly be seen that, although from the water levels the dry bed case may seem smooth, a shock in the solution exists at the front of the flow. The analytical exact solution is based on Stoker's method (Stoker, 1957). For detailed information on the solution method of the numerical exact solution see in appendix B.

![Figure 7.4: Analytical exact solution of dam-break test, dry bed, with a) the water levels and b) the velocities.](image)
7.4 Results

The second step was to simulate the test cases using DELFT-FLS. The computation of the parallel test case gave no real problems. From figure 7.6 it can be seen that the front of the propagation flow is straight and perpendicular to the walls. From the graphs in figure 7.7 it can be seen that the propagation velocity of the front is also correct. This leads to the conclusion that this test case is represented correctly in DELFT-FLS.

The diagonal test case gave more problems however. The front of the wave was not straight anymore, but became more curved as the test proceeded in time, although it should be straight when no wall friction is present (see figure 7.8).
Figure 7.8: Top view of 1D dam-break test case at $t = 15\, \text{s}$ (left) and $t = 30\, \text{s}$ (right) with curved wave front.

Figure 7.9: Water levels of 1D diagonal wet bed test case at $t = 15\, \text{s}$ (left) and $t = 30\, \text{s}$ (right) without boundary improvement.

The cause seemed to be a drag at the sides of the channel. As wall friction was neglected however, this drag should not have been present. This drag gradually influenced the rest of the channel, which can be seen in the propagation of the front in the centreline of the flow. At first the front of the flow can keep up with the exact solution, but later it falls behind. This showed that in a two dimensional case the alignment of the grid can make a difference in the calculation of front propagation. At boundary cells, and subsequently at nearby cells, a decrease in front propagation speed can be found.

Let us consider a 1-dimensional dam break case on a 2-dimensional grid where the flow is diagonally over the grid. See figure 6.1b. First we will look at the mass balance for two cases:

1. A cell which is completely surrounded by other active cells
2. A boundary cell with two closed sides.
Figure 7.10: Grid with 1) a centre cell and 2) a boundary cell

If we take a mass balance over the cell we get with a bit of algebra

$$\frac{\Delta \zeta_1}{\Delta t} + \frac{\Delta (u_i h_i)}{\Delta x} + \frac{\Delta (v_j h_j)}{\Delta y} = 0 \quad \text{for cell 1}$$

and

$$\frac{\Delta \zeta_2}{\Delta t} - \frac{u_i h_i}{\Delta x} + \frac{v_j h_j}{\Delta y} + \frac{\Delta (v_j h_j)}{\Delta y} = 0 \quad \text{for cell 2}$$

We want to compare the two cases. In case of diagonal flow over a rectangular grid with the same velocities and gradients in both x- and y-direction, we can assume that:

$$u_1 = u_2 = v_1 = v_2 = u, \ h_1 = h_2 = h \quad \text{and} \quad \frac{\Delta \zeta}{\Delta y} = \frac{\Delta \zeta}{\Delta x}. $$

The mass balances can then be rewritten to:

$$\frac{\Delta \zeta_1}{\Delta t} = -2 \frac{\Delta (uh)}{\Delta x} \quad \text{for cell 1}$$

and

$$\frac{\Delta \zeta_2}{\Delta t} = - \frac{\Delta (uh)}{\Delta x} \quad \text{for cell 2.}$$

Now it can clearly be seen that

$$\frac{\Delta \zeta_2}{\Delta t} = \frac{1}{2} \frac{\Delta \zeta_1}{\Delta t}$$

This will introduce a lag in the propagation of the front near the boundaries of the flow. It can be compensated by halving the volume to be filled. To accomplish this the active area of a boundary cell should be halved. It should then also be proven, that also the momentum equation is fulfilled, when the active area of a boundary cell is halved.

First a method has to be found to describe the momentum equation over a cell with halved active area. In the DELFT-FLS method this is done by introducing so called virtual
velocity points on closed boundaries. These virtual velocities are dimensioned such, that the closed boundary is put in the diagonal of the cell. This way the boundary cell is equal to a non-boundary cell, concerning the momentum equation. See figure 7.11.

![Diagram showing real and virtual points](image)

Figure 7.11: Real and virtual points.

To model the real boundary, virtual velocities are assumed on the simulated boundaries. To accomplish this the velocities that are projected parallel to the real boundary have to be equal on both sides of the real boundary and the velocities that are projected perpendicular to the real boundary have to cancel each other out. The parallel and perpendicular projections of the real and virtual velocities become as follows:

**Parallel vector-projected velocities:**

\[
w_{r_{||}} = w_{v_{||}}
\]

with \( w_{r_{||}} = u_r \cos(\alpha) + v_r \sin(\alpha) \)

\( w_{v_{||}} = u_v \cos(\alpha) + v_v \sin(\alpha) \)

**Perpendicular vector-projected velocities:**

\[
w_{r_{\perp}} = -w_{v_{\perp}}
\]

with \( w_{r_{\perp}} = -u_r \sin(\alpha) + v_r \cos(\alpha) \)

\( w_{v_{\perp}} = u_v \sin(\alpha) - v_v \cos(\alpha) \)
Thus leading to the following set of equations:
\[
\begin{align*}
    u_c \cos(\alpha) + v_c \sin(\alpha) &= u_r \cos(\alpha) + v_r \sin(\alpha) \\
    -u_r \sin(\alpha) + v_r \cos(\alpha) &= u_r \sin(\alpha) - v_r \cos(\alpha)
\end{align*}
\]

For a structured rectangular grid cell with a closed diagonal boundary ($\alpha = 45^\circ$), like in figure 6.5, these two equations become:
\[
\begin{align*}
    u_r + v_r &= u_v + v_v \\
    -u_r + v_r &= u_v - v_v
\end{align*}
\]

It can now simply be seen that:
\[
\begin{align*}
    u_v &= v_r \\
    v_v &= u_r
\end{align*}
\]

With the virtual velocities known on the boundaries, (wall-) boundary cells can be treated the same way as non-boundary cells. Thus the active area of the cell is halved without disrupting the momentum equation over that cell.

This solution was implemented into the code of Delft-FLS as follows: The code checks if a cell is a diagonal boundary cell, by checking if two adjacent sides of that cell are closed. This is done by assigning each side a value 1, 2, 4 and 8 respectively when that side represents a closed boundary and zero when that side is an open boundary (see figure 7.12). The cell is a diagonal boundary cell only when the sum of the values is 3, 6, 9 or 12.

![Diagram](Figure 7.12: Top view diagonal boundary value assignment.)

With this adapted closed boundary treatment some new simulation runs have been done. From figure 7.13 it can be seen that the diagonal 1D channel test case now propagates with a straight front. This is the correct representation when no wall friction is present.
Figure 7.13: Front propagation after dam breach. $t = 60$ s.
8 CADAM Test cases

8.1 Introduction

8.1.1 The CADAM test case with a 45° channel bend

In February 1998 CADAM - the European Concerted Action on Dam-break Modelling - was started. Intended as a study programme running for two years it is funded by the European Commission and has the aim, among others, to exchange information on dam-break modelling between Universities, Research Organisations and Industry. One of the specific objectives is the creation of a database of test cases (analytical, experimental and real life) available for reference (CADAM, 1998). Different approaches to the numerical simulation of flow after dam failure have been compared to the experimental data in the database. The experiments combine different properties of dam-break flow that have proven difficult to simulate accurately.

(Hydro-) dam-break flow differs from levee-break flow. The most important differences are:

- Water level differences are smaller in case of levee breach than in case of (hydro-) dam breach.
- Downstream flow is usually into a wide flat polder after levee breach as opposed to flow into a highly channelled valley after (hydro-) dam breach. The levee-break flow will thus be more 2Dh of nature on a length scale than the dam-break flow.

The numerical principles, however, are largely the same for both levee-break flow as for dam-break flow. The front of the flow will be highly dynamic. This is the most difficult part of the flood to simulate, and this is what most researches have focused on in the past. This research project focuses on one of the more extensive test-cases: Dam-break flow into a channel with a 45° bend.

8.2 Set-up of the test case

This model combines a square-shaped reservoir upstream and a prismatic channel with a 45° bend downstream of a dam. Flow will be essentially 2D in the reservoir and in the bend region between the two reaches of the channel. This test-case has some interesting characteristics of the flow resulting from the dam-break. The first is the effect of the corner and the second is the upstream-moving hydraulic jump which forms at the corner. The step of the bottom profile at the inlet of the channel and the flow in the second leg of the channel diagonally through the grid may also be difficult to model.
The geometry of the test-case is as shown in figure 8.1:

![Plan View](image)

![Side View](image)

Figure 8.1: Geometry of the CADAM test-case. Actual dimensions for exact grid in cm (Soares, 1998a).

The gate is of the guillotine-type and can be raised in a very short time. Nine gauges have been used for water level measurement. One in the reservoir, three in the first leg of the channel, three on a cross-section just downstream of the bend and two in the second leg of the channel. Three gauges G5, G6 and G7 have been put in one cross-section to show the skewing of the flow in the bend section. The other six gauges have been set up in the centreline of the channel. Measurements from these gauges have been collected with a sampling rate of 0.1 s for a duration of 40 s. The exact data of the measurements can be found in appendices E and F.

For the simulation a rectilinear grid of 186 by 80 cells was designed. The sides of the cells were set to 0.0495 m, making the first leg of the channel ten cells wide. The end of the channel is designed as a chute. Bottom friction was omitted in the simulations. The gauges have been located in the following points of the grid:

<table>
<thead>
<tr>
<th>Gauge</th>
<th>X-location [cells right from bottom left]</th>
<th>Y-location [cells up from bottom left]</th>
<th>Gauge</th>
<th>X-location [cells right from bottom left]</th>
<th>Y-location [cells up from bottom left]</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>33</td>
<td>15</td>
<td>G6</td>
<td>136</td>
<td>18</td>
</tr>
<tr>
<td>G2</td>
<td>56</td>
<td>15</td>
<td>G7</td>
<td>134</td>
<td>20</td>
</tr>
<tr>
<td>G3</td>
<td>87</td>
<td>15</td>
<td>G8</td>
<td>144</td>
<td>26</td>
</tr>
<tr>
<td>G4</td>
<td>117</td>
<td>15</td>
<td>G9</td>
<td>165</td>
<td>47</td>
</tr>
<tr>
<td>G5</td>
<td>138</td>
<td>16</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 8-1: Location of gauges
8.3 Results

The CADAM test cases proved to be very interesting. At first results were terrible, later with the improved numerical scheme implemented for the 1D test case, results got a lot better. First the dry bed case will be discussed, and then the wet bed case. The results will be discussed, first generally and then more focused onto the measurements of the different gauges.

For better readability in the following paragraphs ‘the dry bed case’ and ‘the wet bed case’ will be used, meaning ‘the CADAM test case with initially a dry, respectively wet, bed in the channel’.

8.3.1 The dry bed case

When the gate is opened at $t = 0$, the water starts to flow through the channel. At the entrance of the channel a forward and a backward propagating wave develops. The back wave bounces off the upstream wall of the reservoir after some time. The forward wave arrives at the bend and be forced around the corner. At this point an upstream travelling wave is produced, which propagates as a bore. This bore is clearly seen in the experiments (Soares, 1998b). After about 20 seconds this bore enters the reservoir and dissipates. Through the bend the water level skews, with the higher side to the outside of the bend. Arriving at the end of the channel, the water drops off the chute.
Figure 8.2: Comparison of the time evolution of the water level in nine gauging points between the experiments (---) and the DELFT-FLS model (-----) with initial dry bed. For the locations of the gauges see figure 8.1.

In the reservoir the water level drops gradually. It can be seen from figure 8.2a that the reservoir water level drops faster in the simulation than in the experiment. For about 5 seconds this leads to a higher flux through the entrance of the channel. After these five seconds the flux lowers and the water level drop speed becomes about the same for the simulation as for the experiment. A possible cause of this effect is that the simulation is 2Dh and has all variables averaged over the depth. Therefore, when the flow separates vertically, this is not simulated accurately by DELFT-FLS. In addition to this the width of the channel is represented by only 10 cells which can lead to an underestimation of horizontal contraction at the channel entrance in the simulation. The result is that more water can pass the entrance of the channel in the channel. Further in time, this effect is compensated by the lower head difference between upstream and downstream water levels.

From the graph of gauge 2 (figure 8.2b) it can be seen that even after about 5 seconds the water level stays too high in comparison to the experimental water levels. The higher
water levels are present until the bend-generated bore arrives. Several possibilities can be found to explain this phenomenon:

1. Assumptions of uniform vertical velocity distribution and hydrostatic pressure do not apply in regions with strong gradients of the water surface. However, the numerical method used in DELFT-FLS is designed to cope with these large gradients (Stelling, 1998). In addition to this Soares (1998b) found that visual and pressure-gauge measured water levels agreed. This means that the hydrostatic pressure assumption seems to hold at the location of gauge 2.

2. The progressive development of turbulence along the channel is not taken into account. This should however not be influencing the water levels significantly at the location of gauge 2.

3. Soares (1998b) found that the downstream travelling flow near gauge 2 in the experiment is near critical. The numerical method used in DELFT-FLS can handle transitions between sub- and super critical flow. In this case the downstream travelling flow near gauge 2 seems to stay about critical, until the bend-generated bore arrives. Near the critical stage of a flow small errors in energy values result in high errors of water level but not so high levels of discharge (Soares, 1998b).

4. Because of the step in the bottom profile, vertical separation of the flow will occur. The velocity in the resulting jet is higher than without vertical separation. In the simulation, with vertically averaged velocities and consequently no vertical separation, the velocities will be lower and thus the water levels higher.

The water levels at the location of gauge 3 (figure 8.2c) show very good agreement with the experiments, except for the upstream moving bore. The bore arrives at gauge 3 some four seconds late. This lag is also present at gauge 4 (figure 8.2d) and continues at least until gauge 2, and thus probably until it reaches the reservoir. The lag must have been created in the last part of the first leg of the channel, between gauges 4 and 5 (figures 8.2d and 8.2e). From the graph of gauge 5 it can be seen that the front of the flow in the simulation arrives at the bend a bit earlier than the front of the flow in the experiment. This means the front of the flow propagates too fast in the simulation, which in turn means the bore will arrive at gauge 2 later. This higher propagation velocity can be explained by the higher flux in the first seconds after the opening of the gate. Later the flux lessens towards the value found in the experiment, and the flow velocity also slows down, because it is near critical. From then on the propagation velocity of the bore from the simulation is about the same as that from the experiment.

A quick sensitivity test was done on the influence of bottom friction. Soares (1998) gives Manning coefficients for bottom and wall friction. These are $0.0095 \text{ s/m}^{1/3}$ and $0.0195 \text{ s/m}^{1/3}$ respectively. The rather high wall roughness is due to joints between glass plates of which the wall is constructed. The bottom roughness can directly be input in DELFT-FLS. Wall friction is not taken into account in DELFT-FLS, but by increasing the value for the bottom friction, this could roughly be compensated.
Figure 8.3: Sensitivity test of the influence of bottom friction on CADAM test case.

From figure 8.3 it can be seen that bottom friction has influence on the propagation velocities in the channel. This can be seen from the arrival times of the upstream travelling bore at gauge 3. With bottom friction the downstream particle velocity is lower and the upstream travelling bore subsequently reaches gauge 2 faster than without bottom friction. A Chézy value of about 60 m$^{1/2}$/s seems a good average value, which is also in fairly good agreement with the value found through the Manning formula using the given Manning roughness value.

In the bend region, the flow skews. This can clearly be seen when comparing the water levels at gauges 5 and 7 in figures 8.2e and 8.2g respectively. The water level at the outside of the bend (gauge 5) is higher than at the inside of the bend (gauge 7). Also it can be seen that the skewing of the flow in the simulation is more than in the experiments for the first seconds. At first it is much higher, after about ten seconds it slowly drops to about the values of the experiment and after about 25 seconds the water levels from the simulation drop below those from the experiment.

Gauges 8 and 9 (figures 8.2h and 8.2i) show good agreement with the values found in the experiment. At first the water levels are about the same as found in the experiments,
later they drop a bit below those found in the experiments. The water level differences between the simulation and the experiment later in time are comparable with those in the reservoir later in time.

Another major difference between the experiment and the simulation is that the simulation does not reproduce the undulations that are clearly present in the experiment. One of the possible reasons can be that the raising of the gate in the experiment is not instantaneous which could influence the flow in the channel while in the simulation the raising of the gate can be instantaneous.

All in all, five major differences can be found between the simulation and the experiment:
1. The water level in the reservoir drops too quickly during the first few seconds,
2. The water level calculated at the location of gauge 2 is too high,
3. The arrival times of the bore are lagging compared to the experiments when bottom friction is omitted,
4. The skewing of the flow surface is too high during the first seconds of the simulation,
5. The undulations are hardly reproduced by the simulation.

Nevertheless, the results are satisfying because the flow is simulated correctly in a qualitative sense. The water level that is produced by the simulation follows the mean water level of the experiment for large parts of the test case. This was done without ‘turning of knobs’ or any other fine tuning of this test case. With some modifications, like squeezing of the entrance of the channel for a short time, the results will probably get better quantitatively. This however can not be done in real world cases, where no measurements are available for comparison.

8.3.2 The wet bed case

The test with the initial wet bed was done with 1 cm water in the channel at the moment the gate opens. Figure 8.4 shows the results from this experiment.

![Graphs showing water level over time for Gauge 1 and Gauge 2](image-url)
Figure 8.4: Comparison of the time evolution of the water level in nine gauging points between the experiments (---) and the DELFT-FLS model (---) with initial wet bed. For the locations of the gauges see figure 8.1.

When comparing the results from the simulation with the experiment for the wet bed case several observations can be made. But, as the observations are almost the same for the comparison of the wet bed case as for the comparison of the results of the dry bed case, which are discussed in section 8.3.1, only the differences between the two cases will be discussed.
The major differences between the results of the two experiments are the differences in the undulations during the first few seconds and the difference in arrival times of the bend generated upstream travelling bore.

When comparing figures 8.2 c, d and e to 8.4 c, d and e, most obvious is the difference in undulation at the front of the wave. In the wet bed case the undulations are much stronger then in the dry bed case. This suggests a bore at the front of the flow in the wet bed case, because the oscillations are more intensive.

In the experiment the upstream moving bore, produced in the bend area, arrives earlier for the initial wet bed test case than for the dry bed case. In the simulation this is not reproduced at all. This means that either the upstream travelling wave propagates faster, or the downstream flow velocity is less. The arrival times of the front of the wave in the wet bed case are somewhat later than those of the dry bed case. This suggests a lower downstream flow velocity for the wet bed case.
9 Conclusions and recommendations

9.1 Conclusions

1. Only at the front of a flood wave dynamic terms dominate the momentum equations, yet they are very important when modelling flow.
   Dynamic terms of the momentum equation only have a small space-scale importance. However when modelling flow, it is very important to implement these terms correctly, small differences in implementation can cause large differences in front propagation velocities and front shapes.

2. The method used in DELFT-FLS gives qualitatively good overall results when simulating flow over both initially dry and wet bed of the CADAM test case.
   The results found with the DELFT-FLS method are probably a bit influenced by the inlet of the channel. This inlet is simulated only 10 cells wide, which might be too low a resolution. The results however, do follow the mean water level of the experimental measurements. A simulation with higher resolution might give a better solution. For polder scale computations the method will probably provide accurate enough solutions.

3. Problems with closed boundaries not parallel to structured rectangular grids can be effectively solved.
   For a boundary situated at an angle of 45° to the grid direction, a routine was found to correctly handle the boundary conditions. This routine is very specialised, but more generic solutions can be found by testing the shape of a boundary over more than one cell in stead of over just one cell as is done now. Then a more generic calculation of the resulting active flow area and the virtual velocities needed is possible, but still needs to be implemented. When simulating polders, closed boundaries that do not lay parallel to the grid have less influence. The effect is largest for narrow valleys and channels with large depth to width ratios, but small for polder flooding with very small depth to width ratios.

4. Sub-grid effects are important in simulations of polder flooding.
   When simulating polder flooding, with large areas of over-land flow and lots of (small) objects that influence the flow, friction and sub-grid effects are important. Therefore good representation rules will have to be found to incorporate these friction and sub-grid effects into a simulation.

5. The use of a structured rectangular grid of DELFT-FLS is suitable for the simulation of flooding.
   Even though a rectangular grid needs special treatment to overcome inaccuracies along boundaries that do not lie parallel to on of the grad axes, it gives results that are comparable to the real world. A structured rectangular grid makes it very easy to import maps of areas, because a rectangular grid is an often used format in Geographic Information Systems.
9.2 Recommendations

DELFt-FLS is currently being developed for use by local and central governments. It currently is not much more than a shallow layer around a calculation method. For successful use and operability DELFT-FLS should be further developed. To make it a more complete, user friendlier and more broadly applicable simulation tool currently several improvements are being implemented in DELFT-FLS:

- Coupling with the 1D model SOBEK,
- Design of a graphical user interface,
- Implementation of rainfall, run-off and ground water models,
- Damage assessment models.

This project has taken a look into the more fundamental, basic validation tests, with emphasis on the representation of dynamic flow. More detailed information should be obtained on the simulation of sub-grid effects. This can be done by comparing laboratory experiments with well chosen sub-grid effects to computer simulations. Some interesting sub-grid effects may come from flow around objects that are smaller than the main grid size in the computer simulation. Research into sub-grids effects should be done. The best way to do this is probably by the use of experiments.

Also some real world scale verification tests should be done. One possibility is a simulation of the Gelderse Vallei, an area around Utrecht in The Netherlands. For this area, some information on past floods is available. It would consist of making an inventory of available data, incorporating this data into a simulation model (like DELFT-FLS) and assessing on how past floods can be used for the prediction of future floods.

The 1D and CADAM test cases provide mainly validation on dynamic characteristics of the flow. For validation of characteristics of overland or sheet flow, other simulations should be done on larger scales. Then roughness, infiltration and the effect of large obstacles like cities or villages, can be used to validate DELFT-FLS. Possible places, where information can be found are the military archives. Some measurements on floods might be available on the Grebbe Line and the other water defences that made up fortress Holland in the last few centuries.
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Available digitally (MS Word 97 format):
http://www.hrwallingford.co.uk/projects/CADAM/proceedings/soares.doc


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The DELFT-FLS model

DELFT-FLS is a 2Dh simulation model especially designed for the simulation of floods. The numerical method is adapted from the DELFT-3D method to accurately calculate all different flow regimes, like river flow, overland flow and supercritical flow.

The numerical method used in DELFT-FLS is based on the continuity and shallow water equations in the following way:

\[
\frac{\partial \zeta}{\partial t} + \frac{\partial (uh)}{\partial x} + \frac{\partial (vh)}{\partial y} = 0
\]  
(A.1)

\[
\frac{\partial (uh)}{\partial t} + \frac{\partial (hu^2)}{\partial x} + \frac{\partial (vuh)}{\partial y} + gh \frac{\partial \zeta}{\partial x} + g \frac{u^2}{C^2} = 0
\]  
(A.2)

\[
\frac{\partial (vh)}{\partial t} + \frac{\partial (vhu)}{\partial x} + \frac{\partial (h v^2)}{\partial y} + gh \frac{\partial \zeta}{\partial y} + g \frac{v^2}{C^2} = 0
\]  
(A.3)

with

- \( \zeta \) water level above reference [m],
- \( d \) water level below reference [m],
- \( h \) Total water depth, \( h = \zeta + d \) [m],
- \( u \) velocity in x-direction [m/s],
- \( v \) velocity in y-direction [m/s],
- \( C \) Chézy coefficient \([m^{1/2}/s]\) and
- \( g \) gravitational acceleration term \([m/s^2]\).

The continuity and shallow water equations are discretised to a fully staggered grid, a so called C grid. In such a grid, the water levels are approximated at locations \((i, j)\), the velocities in x-direction \((u)\) at \((i+1/2, j)\) and the velocities in y-direction \((v)\) at \((i, j+1/2)\). Such a grid can be seen in the next figure:

```
  j +1/2  v
  j       u  \zeta, d, h  u
  j-1/2   v
  i-1/2  i  i+1/2
```

Figure A.1: 2D staggered grid
The DELFT-FLS scheme incorporates the following characteristics. First, the continuity equation is approximated with both global and local mass conservation. Secondly, the water depths are guaranteed to be positive, so no ‘flooding and drying’ procedures are needed. These two characteristics ensure a stable solution. This can be shown as follows.

When the bottom is considered not to vary in time, the continuity equation can be rewritten to:

$$\frac{\partial h}{\partial t} + \frac{\partial (uh)}{\partial x} + \frac{\partial (vh)}{\partial y} = 0$$  \hspace{1cm} (A.4)

A semi-discrete discretisation of (4) is given by:

$$\frac{dh}{dt} + \frac{u_{i+1/2}^n - u_{i-1/2}^n}{\Delta x} h + \frac{v_{i+1/2}^n - v_{i-1/2}^n}{\Delta y} h = 0 \text{ at } (i,j)$$  \hspace{1cm} (A.5)

where $u$ at $(i,j)$ means $u_{i,j}$.

The time integration is based on the well known $\theta$-method. The continuity equation then becomes:

$$\frac{h_{i+1,j}^n - h_{i,j}^n}{\Delta t} + \frac{u_{i+1/2}^{n+\theta} - u_{i-1/2}^{n+\theta}}{\Delta x} h_{i,j}^n + \frac{v_{i+1/2}^{n+\theta} - v_{i-1/2}^{n+\theta}}{\Delta y} h_{i,j}^n = 0 \text{ at } (i,j)$$  \hspace{1cm} (A.6)

with $u^{n+\theta} = \theta u^{n+1} + (1 - \theta) u^{n}$, $v^{n+\theta} = \theta v^{n+1} + (1 - \theta) v^{n}$.

This can then be rewritten, assuming positive flow, to:

$$h_{i+1,j}^{n+1} = h_{i,j}^{n} - \frac{\Delta t \cdot u_{i+1/2}^{n+\theta}}{\Delta x} h_{i,j}^{n} + \frac{\Delta t \cdot u_{i-1/2}^{n+\theta}}{\Delta x} h_{i-1,j}^{n} - \frac{\Delta t \cdot v_{i+1/2}^{n+\theta}}{\Delta y} h_{i,j}^{n} + \frac{\Delta t \cdot v_{i-1/2}^{n+\theta}}{\Delta y} h_{i-1,j}^{n} \text{ at } (i,j)$$  \hspace{1cm} (A.7)

with $u_{i+1/2}^{n+\theta} > 0$, $u_{i-1/2}^{n+\theta} > 0$, $v_{i+1/2}^{n+\theta} > 0$ and $v_{i-1/2}^{n+\theta} > 0$.

It can now be seen, that strict positivity follows when:

$$\frac{\Delta t \cdot u_{i+1/2,j}^{n+\theta}}{\Delta x} + \frac{\Delta t \cdot v_{i,j+1/2}^{n+\theta}}{\Delta y} < 1$$  \hspace{1cm} (A.8)

Finally, the momentum equation is approximated with a proper momentum balance near large gradients. This ensures that the solution converges to a stable solution.

The momentum conservative case for flow in $u$- and $v$-directions can be written as:
\[
\begin{align*}
\frac{du}{dt} + \frac{s_q - 1 + \frac{q}{h_{1/2} + h_{3/2}}}{h_{1/2} + h_{3/2}} \left( \frac{u - u_{-1}}{\Delta x} \right) + \frac{s_q - 1 + \frac{q}{h_{1/2} + h_{3/2}}}{h_{1/2} + h_{3/2}} \left( \frac{u - u_{-1}}{\Delta x} \right) + g \zeta_{0r} + g \frac{u|u|}{C^2 h(u)} = 0, \quad \text{at } (i + 1/2, j) \\
\frac{dv}{dt} + \frac{s_q - 1 + \frac{q}{h_{1/2} + h_{3/2}}}{h_{1/2} + h_{3/2}} \left( \frac{v - v_{-1}}{\Delta y} \right) + \frac{s_q - 1 + \frac{q}{h_{1/2} + h_{3/2}}}{h_{1/2} + h_{3/2}} \left( \frac{v - v_{-1}}{\Delta y} \right) + g \zeta_{0r} + g \frac{v|v|}{C^2 h(v)} = 0, \quad \text{at } (i, j + 1/2)
\end{align*}
\]

with
\[
\begin{align*}
\frac{s_q}{q} = uh(u), \\
\frac{q}{q} = vh(v), \\
\left( \zeta_{i+1/2,j} \right)_x = \frac{\zeta_{i+1,j} - \zeta_{i,j}}{\Delta x}, \quad \text{and} \\
\left( \zeta_{i,j+1/2} \right)_y = \frac{\zeta_{i,j+1} - \zeta_{i,j}}{\Delta y}.
\end{align*}
\]

This approximation applies for positive flows only, but similar approximations can be constructed for other flow directions.
B Analytical 1-dimensional dam-break solution

In this appendix the equations, used to describe 1-dimensional flow after a dam break, are presented. Two different situations can be discerned. The first is the dry bed case, where the upstream side of the dam contains water with a constant depth and the downstream side consists of a dry horizontal bed. The second is the wet bed case, where both sides of the dam contain bodies of water. The derivation of the equations will be done for both the dry and the wet bed case.

The solution of both cases is based on integration of the differential equations by the method of characteristics (Stoker, 1957). The following differential equations can be used in the frictionless 1Dh case:

\[
\frac{\partial \zeta}{\partial t} + \frac{\partial (uh)}{\partial x} = 0
\]  
(B.1)

\[
\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + g \frac{\partial \zeta}{\partial x} = 0
\]  
(B.2)

For this set of differential equations the following characteristics can be given to describe the solution curves in the x, t-plane:

\[
\begin{align*}
C_1: & \quad \frac{dx}{dt} = u + c \\
C_2: & \quad \frac{dx}{dt} = u - c
\end{align*}
\]  
(B.3)

with \( c = \sqrt{gh} \) the speed, relative to the flowing stream, at which a disturbance propagates.

A centred simple wave can be constructed by exchanging the dam with a piston which is moved with velocity \( w \) in the direction of the positive x-axis. The characteristics of the wave that follows directly behind the piston (still in contact with that piston) give the following figure in the x, t-plane:
Three areas can be discerned. In zone I the water in still at rest. The line dividing zone I and II is the characteristic belonging to the first instants of the flow. As the water level drops, $c$ will drop as well, which is represented by a steeper line in the $x$, $t$-plane. In zone II the slopes of the characteristics are given by $\frac{x}{t} = \frac{3}{2}u + c_0$. This zone is terminated on the upper side by the line $x = \left(\frac{3}{2}w + c_0\right)t$. In zone III again a constant state exists. This zone is limited on the upper side by the characteristic of the ‘piston curve’. The piston is supposed to accelerate in infinite small time from 0 to a certain velocity $w > 0$. The exact direction of the line dividing zone II and III depends on the velocity $w$ of the piston. When $w$ becomes equal to $-2c_0$ zone III disappears, because the dividing line between zones II and III falls onto the piston curve. If the piston is moved at even higher velocities, the flow will separate from the piston and become free flow. This is what happens in a dam-break scenario with an initially dry downstream bed. The surface profile of the following flow looks as follows:

With some algebra values for $c$ and $u$ in zone II can be found. They are:

$$
\begin{align*}
    c_2 &= \frac{1}{3} \left( \frac{2c_0 - \frac{x}{t}}{2} \right) \\
    u_2 &= \frac{2}{3} \left( c_0 + \frac{x}{t} \right) \\
\end{align*}
$$

$$\forall x \in [-c_0t, 2c_0t]$$

On the downstream side of the dam the water level rises. As the water level rises the velocity $c$ will increase as well. The characteristics will cut each other. From there on the method of characteristics is not valid anymore and no solution can be found. A shock will
form. To overcome the difficulty of shocks the laws of conservation of mass and momentum can be applied over a column of water that encloses the shock.

The actual calculations were done using the following FORTRAN 77 code:

```fortran
program dambrk
  implicit double precision (a-h, o-z)
  dimension s(0:100), s0(0:100), u(0:100), x(0:100)
  c
  c Copyright G.S. Stelling
  c
  c Initial values
  c
  g = 9.81
  t = 0.0
  dt = 1.0
  xd = 0.0
  x0 = 0.0
  xu = 0.0
  dx = 5.0
  h1 = 1.0
  h0 = 0.1d-12
  mmax = 100
  itmax = 50
  eps = 1.0d-8
  do l = 0,100
    x(i) = (i-50)*dx
  enddo
  do l = 0,100
    if (x(i),lt,0.0) then
      s(i) = h1
    else
      s(i) = h0
    endif
    s0(i) = s(i)
    u(i) = 0.0
  enddo
  c
  c Initialise for iteration
  c
  c1 = sqrt(g*h1)
  c0 = sqrt(g*h0)
  u20 = c0
  z0 = c0
  c20 = c0
  iter = 0
  10 continue
    iter = iter+1
  c
  c Newton iteration for correct coefficients of exact solution
  c
  aa = 2*z0 - u20
```

---

wl | delft hydraulics
\[ ab = -c_{20} \]
\[ ac = -z_0 \]
\[ ad = 0.5c_0^2 + u_{20}z_0 - z_0^2 + 0.5(c_{20})^2 \]
\[ ba = -c_{20}^2 + c_0^2 \]
\[ bb = 2c_{20}u_{20} - 2c_0z_0 \]
\[ bc = c_{20}^2 \]
\[ bd = -c_{20}^2u_{20} + c_0^2z_0 - c_0^2z_0 \]
\[ ca = 0.0 \]
\[ cb = 2.0 \]
\[ cc = 1.0 \]
\[ cd = 2c_1 - u_{20} - 2c_{20} \]
\[ dd = aa*(bb^2cc - bc^2cb) - ab^*(ba^2cc - bc^2ca) + ac^*(ba^2cb - bb^2ca) \]
\[ d1 = ad^*(bc^2cc - bc^2cb) - ab^*(bd^2cc - bc^2cd) + ac^*(bd^2cb - bb^2cd) \]
\[ d2 = aa^*(bd^2cc - bc^2cd) - ad^*(ba^2cc - bc^2ca) + ac^*(ba^2cd - bd^2ca) \]
\[ d3 = aa^*(bb^2cd - bd^2cb) - ab^*(ba^2cd - bd^2ca) + ad^*(ba^2cb - bb^2ca) \]
\[ dz = d1/dd \]
\[ dc2 = d2/dd \]
\[ du2 = d3/dd \]
\[ z0 = z0 + dz \]
\[ c20 = c20 + dc2 \]
\[ u20 = u20 + du2 \]
\[ \text{if (abs(dz).gt.eps) goto 10} \]
\[ \text{if (abs(dc2).gt.eps) goto 10} \]
\[ \text{if (abs(du2).gt.eps) goto 10} \]

\[ c \]
\[ \text{Correct shock speeds (z, c2 and u2) are found} \]
\[ c \]
\[ z = z0 \]
\[ c2 = c20 \]
\[ h2 = c2^2/g \]
\[ u2 = u20 \]
\[ \text{do itime = 1,imax} \]
\[ t = t + dt \]

\[ c \]
\[ \text{Determination of various zones with different solutions} \]
\[ c \]
\[ xd = -c1^2t \]
\[ xm = (u2 - c2)^2t \]
\[ xu = z^2t \]
\[ \text{do l = 0.mmmax} \]

\[ c \]
\[ \text{Undisturbed left area} \]
\[ c \]
\[ \text{if (x(i).lt.xd) then} \]
\[ u(i) = 0.0 \]
\[ s(i) = h1 \]
\[ c \]
\[ \text{Rarefaction area} \]
\[ c \]
\[ \text{else if (x(i).ge.xd.and.x(i).lt.xm) then} \]
\[ c = (2c1 - x(i)/t)/3.0 \]
\[ s(i) = c^2/g \]
\[ u(i) = 2.0/3.0*(c1 + x(i)/t) \]
\[ c \]
\[ \text{Shock area} \]
else if (x(i).ge.xm.and.x(i).lt.xu) then
  u(i) = 2.0/3.0*(c1 + xm/h)
  s(i) = h2
end

/* Undisturbed right area */

else if (x(i).ge.xu) then
  u(i) = 0.0
  s(i) = h0
endif
enddo
enddo

/* Write the output to a file */

open(unit = 33, file = 'exact.out')
do m = 0,mmax
  write (33, '(i4,f10.2,3d15.4)') m, x(m) + 250, s0(m), s(m), u(m)
endo
C The 1Dh dry bed case

In this section the results of the computations for the test cases with the initial dry downstream bed are shown.
1Dh test case dry bed, $t = 30$ s.

- Exact solution
- FLS Parallel
- FLS Diagonal
The 1Dh wet bed case

In this section the results of the computations for the test cases with the initial wet downstream bed are shown.
1Dh test case wet bed, t = 30 s.

- Exact solution
- FLS Parallel
- FLS Diagonal

Water level [m] vs. X location [m]
The CADAM dry bed case

The graphs comparing the CADAM test case with initially dry bed will be inserted in this section. The data have been collected over a period of 40 seconds with a sampling rate of 0.1 seconds for nine measuring points.
The graphs comparing the CADAM test case with initially wet bed will be inserted in this section. The data have been collected over a period of 40 seconds with a sampling rate of 0.1 seconds for nine measuring points.
G Water level contour plot at $t = 2.5 \text{ s}$