Emergency closure of dike breaches

The effect and applicability of emergency measures

Master Thesis

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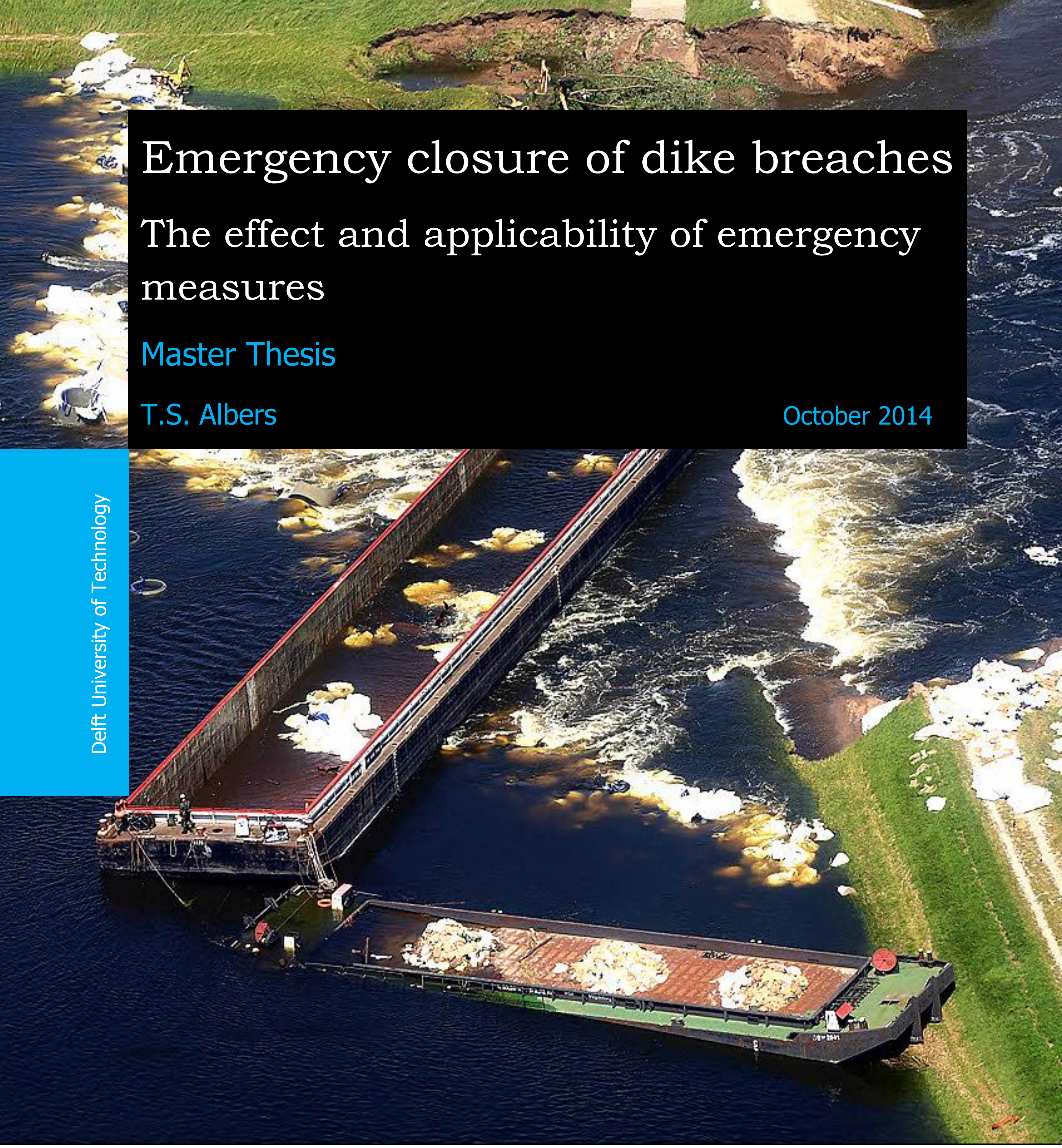

TU Delft Delft University of Technology

Deltares Enabling Delta Life

Cover photo: Breach in Fischbeck Germany, attempted to close by means of vessels and big bags, June 2013.
Source: Deutsche Presse-Agentur (DPA).
Retrieved September 2014 from: http://www.badische-zeitung.de/panorama/das-loch-im-deich-bei-fischbeck-ist-erfolgreich-gestopft--72821409.html

Emergency closure of dike breaches

The effect and applicability of emergency measures

By

Tijmen Sebastiaan Albers

This master thesis has been written in partial fulfilment of the requirements for the Master of Science degree in Civil Engineering, Track Hydraulic Engineering at the faculty of Civil Engineering & Geosciences of Delft University of Technology, Delft, the Netherlands.

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"De hele moderne maatschappij berust nu eenmaal op het vertrouwen van gewone mensen in de tovenaars. En iedereen, die in Nederland beneden de waterspiegel woont, laat in blind vertrouwen de tovenaars-waterbouwkundigen aan de dijken ploeteren."

"Modern society is simply based on the faith of laymen in the wizards. And everyone living below sea level in the Netherlands, blindly entrusts the slogging at the dikes to these wizards-hydraulic engineers."

- A. den Doolaard (1947)

(Freely translated from: A. den Doolaard (2001), Het Verjaagde Water, VSSD, Delft, The Netherlands)

Preface

This thesis is submitted in partial fulfilment of the requirements for the degree of MSc. in Civil Engineering at Delft University of Technology. The research was carried out in cooperation with Deltares and was supervised by both Deltares and Delft University of Technology. This report elaborates on emergency closures of dike breaches and is aimed at scientists, engineers and other people that are interested in the field of flood defences.

Acknowledgements
First of all I would like to thank my graduation committee for their guidance and support. During my research their knowledge and feedback was very valuable and they increased my enthusiasm for the subject. I would like to thank Prof.dr.ir. Bas Jonkman for his help to find this fascinating graduation topic and his constructive criticism during the research. I would like to thank ir. Henk Jan Verhagen for his support and interesting suggestions on my report and I would like to thank ir. Joop Weijers for sharing his enthusiasm, experiences and knowledge about real closures.

Furthermore, I would like to thank Deltares in general for giving me the opportunity to accomplish my thesis in a stimulating environment. Special thanks to ir. Eric Huijskes for his ideas, enthusiasm and creative discussions which helped me a lot during this research and for learning me a lot more than only research related items. Also, special thanks to ir. Ulrich Förster for always making time for a discussion and sharing his knowledge and useful literature. I would like to thank ir. Joost den Bieman and ir. Bas Hoonhout for helping me out with the XBeach software. Also, I would like to thank ir. Kasper Lendering from Delft University of Technology and Hans Knotter from Water Board Rivierenland for their support.

Finally I would like to thank my parents for their stimulation and support in every imaginable way during my study and the years before. Of course, I would like to thank my friends and especially Inge for her scientific suggestions regarding my thesis and for always making me smile.

Tijmen Sebastiaan Albers
Delft, October 2014

Executive summary

Introduction
Floods cause, even in modern countries, a lot of victims and damage every year. Generally, countries invest in prevention and protection, yet, also emergency response measures could be a (last) option. Regarding the latter, a distinction is made between preventive emergency measures (e.g. sand bags to raise the crest level of a flood defence) and curative emergency measures (e.g. measures to combat a breach in a flood defence). It can be stated that the knowledge about the closure of a dike breach and the implementation of emergency measures is not at the desired level. Besides, closing a breach is very difficult and it is rarely performed successfully. There is a need for research regarding emergency closure of dike breaches indicated by among others Dutch Water Boards. The problems of a dike breach can broadly be categorized in three fields: organizational problems, logistical problems and technical problems. This thesis focusses mainly on the technical problems, distinguishing breach characteristics and type of emergency measures. Therefore, in this thesis the research question is: *What is the effectiveness and applicability of curative emergency measures, applied in developing breaches in a Dutch river or lake dike?*

Cases
There are very few cases of dike failure in which emergency measures were applied and measurements of the breach development were documented. Common aspects of previously failed breach closures are a lack of material, equipment and/or manpower and the absence of a solid closure strategy. In order to make a breach closure successful, these aspects should be taken care of. Furthermore for a successful closure, improvisation and quick action are important factors as well.

Simulations
To get a better understanding of the effect of emergency measures, simulations of developing breaches including emergency measures can be made with a numerical model. Simulations have several advantages. It allows identification of breach characteristics like duration and breach stages during closure attempts. Furthermore, it can be used to optimize closure strategies and measures and it can help flood managers to prepare for emergency situations. However, such a model is non-existent. Yet, XBeach is capable of simulating emergency measures as so-called non-erodible layers in developing breaches. With some modifications XBeach seems promising for simulating breach development with emergency measures.

The modifications are based on XBeach's limitations. First, emergency measures are implemented as non-erodible layers, these are however not adjustable in time. Running multiple simulations after each other representing the different closure phases, solves this problem. Second, the non-erodible layer is always stable. This means that the emergency measure can not move due to high flow velocities or scour during the simulation. This is the main limitation of XBeach for this research. To still be able to check the stability, separate calculations are done. Third, XBeach is not a 3D model. Therefore, emergency

measures where a 3D effect (piping) plays an important role are not modelled. Fourth, XBeach is only capable of simulations with non-cohesive sand. To deal with this, a scaling from non-cohesive to cohesive timespans was made.

Emergency measures
For a breach closure in a sand dike, there is (likely too) little time. To estimate the time available in cohesive dikes, the timing is scaled to realistic proportions. Various curative measures exist. Simulated measures in this study in XBeach are a scaffold, Big Bags, a vessel and an emergency dike. The emergency dike seems to be the most promising measure. It makes use of the smaller flow velocities upstream of the breach. The breach dimensions stay smaller and the polder water level is lower. Point of attention is the static stability after the closure, with piping as the most important threat. Logistically seen, a complete closure with an emergency dike for a breach in a clay dike is plausible, using trucks to bring in Big Bags and helicopters to place them at the desired location.

Since the simulations are made for one type of dike, some characteristics are varied to determine their influence. A berm added to the geometry seems to have little influence. However, a drop in outer water level creates a more favourable condition, just as an increased polder area. This is also true for an earlier implementation of the emergency measure. A later implementation will make things worse.

Field test
As part of this thesis experiments were carried out to provide general findings regarding field tests. The experiments have been conducted in a dry and in a saturated state of a sand dike. As expected, the erosion in the dry condition took place faster than in the saturated condition. Hence, saturation has been shown to play an important role. Recommendations for future experiment are to include cohesive soil and test the most promising emergency measures. Experiments are worthwhile and can be used to complement investigations via simulations. Furthermore, they can be used to increase confidence in the findings.

Decision support
It should be noted that Dutch Water Boards currently do not have a protocol in place for implementation of curative measures. For the decision support in this thesis, recommendations are provided which have the aim of collecting basic information needed for implementation and defining basic plans and instructions for implementation of curative measures.

Conclusion
One of the most important conclusions is that the XBeach model provides very valuable results for the simulation of emergency measures in developing dike breaches. The effects of several emergency measures are modelled, explained and checked for stability. Characteristics that have large influences are a drop in outer water level, a larger polder area and an earlier implementation in time. An emergency dike is the most promising measure. It makes use of the more favourable conditions somewhat upstream of the breach. This measure reduces the breach dimensions and the discharge through the breach compared to the other measures. Furthermore, it is logistically plausible to conduct a complete closure using an emergency dike.

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List of Symbols

Basic units

kg	kilogram
m	meter
s	second
°	degree

Other units

N	(=kg · m/s²)	Newton
Pa	(=N/m²)	Pascal

Symbol	**Unit**	**Definition**
A	[m²]	Area of the flow area in the breach
A_D	[m²]	Area where the drag force exerts
A_L	[m²]	Area where the lift force exerts
A_p	[m²]	Area of the polder
A_S	[m²]	Area where the shear force exerts
b	[m]	Breach width at the bottom
B	[m]	Average breach width over the water depth
B_t	[m]	Breach width at the crest
B_w	[m]	Breach width at the water surface
B_a	[m]	Average breach width over the breach depth
C	[$m^{1/2}$/s]	Chézy coefficient
C_D	[-]	Drag coefficient of proportionality
C_L	[-]	Lift coefficient of proportionality
C_S	[-]	Shear coefficient of proportionality
d	[m]	Stone diameter
d	[m]	Water depth in the breach
D	[m]	Pole diameter for scour
d_c	[m]	Critical depth
E_{bo}	[m/s]	Erosion rate at the bottom of the breach
E_{sl}	[m/s]	Erosion rate at the toe of the breach side slope
F_D	[kN]	Drag force on an element
F_h	[kN]	Hydrostatic force of the water
F_L	[kN]	Lift force on an element
F_S	[kN]	Shear force on an element
F_v	[kN]	Vertical uplift force of the water
Fr	[-]	Froude number
g	[m/s²]	Acceleration of gravity
h	[m]	Breach depth
h_o	[m]	Water depth
H_p	[m]	Inner water level
h_s	[m]	Equilibrium scour depth

H_w	[m]	Outer water level
L	[m]	Length of the inner slope of a dike
l_n	[m]	Adaptation length of the flow over the inner slope of a dike
m	[-]	Discharge coefficient
M_e	[sm²/kg]	Material dependent factor describing the erodibility of soil
M_{ef}	[sm²/kg]	Material dependent factor describing the erodibility of soil of the dike foundation
Q_{br}	[m³/s]	Discharge through the breach
R	[m]	Hydraulic radius
t	[s]	Time
t_i	[s]	i-th moment in time
T	[kN]	Friction force
U	[m/s]	Averaged flow velocity
u_c	[m/s]	Critical velocity
U_{peak}	[m/s]	Peak flow velocity
W	[kN]	Own weight
x	[m]	Coordinate along the inner slope of a dike
Z_{br}	[m]	Height of the breach bottom
β	[°]	Slope angle
γ	[°]	Inclination angle of side slopes
ϕ	[°]	Internal angle of friction
ρ_w	[kg/m³]	Density of water
τ	[N/m²]	Shear stress of the dike soil
τ_b	[N/m²]	Bed shear stress
τ_c	[N/m²]	Critical shear stress for erosion of soil

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1. Introduction

1.1 Current state of emergency closures

The Netherlands is known for its extensive knowledge about hydraulic engineering and their successful battle against floods. Today, the result of this long battle can be seen in different regulations, organisations and tools to control the water. Large investments have been and still are made to protect the valuable low-lying land. Water defences are designed and maintained according to the law. Much effort is put into the prevention of floods. Water defences have been and are designed for low probability of flooding and large hydraulic structures have been built to cut off vulnerable flood prone areas from the sea.

Since a couple of years the focus has widened. Next to prevention (mainly protection), spatial planning and disaster management are taken into the scope of flood fighting. With this philosophy, attention is given to other aspects than prevention recently. However, the main focus is (and should be) still on the prevention of floods. In the Netherlands abundant attention is paid to the preventive part. The Netherlands is reasonably well prepared to implement preventive emergency measures. These measures aim to prevent a breach. Until the moment of a dike breach people know what to do. If, however, a dike unfortunately breaches, there is no protocol to follow to close the breach again. This should be done by curative measures; they 'cure' the breach. Furthermore, the effect and application ranges of these curative emergency measures are unknown. Almost no investigations are done for an emergency measure to close or control a dike breach. This could be a missed opportunity since dike breaches have large consequences and it is important to undertake action as fast as possible. The longer it takes to close the breach, the more damage occurs to both the hinterland and the dike itself.

The reason for the low amount of attention paid by the Netherlands to curative emergency measures is because the focus is mainly on the prevention of floods. The probability of a breach is low and curative emergency measures are rarely used. Another reason for the low amount of attention is the limited feasibility of emergency measures. The probability of a successful closure is low, when looking at the history.

Recent floods, like the one in Central Europe in the summer of 2013 showed that even modern countries like Germany are not able to effectively counter a dike breach at this moment. Implementation of emergency measures to close dike breaches failed. Floods caused major damage and an enormous amount of money is spent on recovering from floods. When curative emergency measures were applied, it was not sure whether they would be effective.

Because of the large consequences and the relative frequent occurrence of floods around the world it can be stated that the knowledge about the closure of a dike breach and the actual implementation of emergency measures are not at the desired level. Research is needed to get a better understanding of the physical processes and effects and application ranges of emergency measures. Also, Dutch Waterboards (e.g. Rivierenland) indicate the need for more knowledge on breach closures with curative emergency measures.

The main goals of this thesis are to gain insight in the physical processes that are crucial in the closure of a dike breach and to understand what the effect of an emergency measure will be. Moreover, the application range of the emergency measures regarding the stability is an item to investigate.

Chapter 2 discusses the problem in more detail. The scope, research question, objectives and methodology are presented too. Chapter 3 gives an overview of the literature on the processes taking place during a dike breach and the emergency measures available to implement in this breach. To learn lessons from occurred dike breaches and emergency closures, several cases are investigated in Chapter 4. In Chapter 5, simulations and calculations of emergency measures in developing dike breaches are presented. The effect and stability limits of several emergency measures are explained. Field experiments have been done in this thesis and they are described in Chapter 6. The knowledge gained in this thesis is shaped in recommendations for a decision support system. A plan for a set-up of a decision support system is presented in Chapter 7. This thesis ends with conclusions and recommendations in Chapter 8.

1.2 Collaboration with Deltares

This thesis is written in collaboration with Deltares. Deltares is an independent institute for applied research in the field of water, subsurface and infrastructure. Their main focus is on deltas, coastal regions and river basins. Managing these often densely populated and vulnerable areas is complex, which is why Deltares works closely with governments, businesses, other research institutes and universities at home and abroad[1].

'Flood Risk Management' is one of the work fields of Deltares and focusses on the safety aspect in relation to flooding. A central question is: how safe is safe enough? Addressing this question is high on the agenda of Deltares.

Deltares is also involved in projects for Rijkswaterstaat regarding emergency measures for dikes in response to flood (threat). In 2012 and 2013 a project was carried out, which resulted in an overview of the process and the steps that are needed to go from observed damage on a dike to the implementation of a preventive emergency measure (Deltares, 2013). In this report the knowledge gaps are mapped. The intention for 2014 is, to jointly with Dutch Water Boards and Rijkswaterstaat, build a "wiki" containing relevant information, best practices and possibilities for knowledge exchange. In this way the knowledge is put in a central place and it is possible to learn from the experiences from others. This wiki is a tool for the 'cold phase', when there is no emergency. The next step is to set up a decision support system for the 'warm phase', i.e. for during an emergency. The 'warm phase' is defined as the situation in which the dike watch is on the dike, it actually

[1] *http://www.deltares.nl/en/about-deltares*

may go or even goes wrong and a preventive measure is needed. The decision support system should help come up with the right decision.

At the same time Deltares was carrying out their investigation for preventive measures, Delft University of Technology and STOWA (Centre for Applied Water Management Research) carried out a research (Lendering, et al., 2014) about the reliability and effectiveness of preventive emergency measures. This program made the effectiveness visible with failure probabilities of the preventive measures, including the failure of human action. There was a fruitful interaction between the programs of Deltares and Delft University of Technology. The scope of both projects was limited to the moment before a breach occurred.

This thesis connects to both projects. The focus of this thesis is the moment from a dike breach onwards. It is rather difficult to make a sharp cut in this process since the results from for example the preventive measures are of influence on the curative measures. For this reason the part of the process before a breach is not disregarded. Attempts have been made to connect to these programs in a useful way.

Furthermore it should be pointed out that two research programs of Deltares are supporting this thesis. These are:
- Flood Risk Management Strategies;
- Real-time Information for Flood Event Management.

2. Problem description

2.1 Problem analysis

Process

The breaching of a dike can be seen as a process with several underlying relations. In Figure 2.1, the decomposed core process is visible, together with the scope which is discussed in Chapter 2.2. A lot of connections are left out on purpose since the aim is to reveal the core process. The figure is now discussed. River water levels and discharges are constantly monitored and forecasts are made on regular basis. If these pose no threat, no further actions are taken. If there is a possible threat for the dikes, a warning is broadcasted. This warning starts a response which can have different actions. Evacuation is one of them but is not considered. A logical response is the placement of preventive emergency measures. Examples are sandbags to raise the crest height, to construct a berm to increase stability or for the containment of sand boils. Emergency measures can be divided in preventive and curative emergency measures. Preventive measures are defined as measures that can be implemented before a breach has formed; curative measures are applied after the formation of a breach.

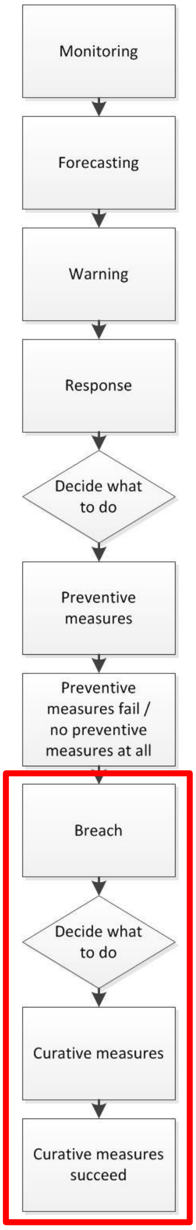

Figure 2.1: Simplified process of a dike breach and the scope

However if a placed preventive measure fails, a breach will occur. This thesis focusses on the curative measures to address such a breach. A decision about a curative measure needs to be made. This decision should be supported by information (about the type of dike, type of load, type of failure mechanism, the place, scale and availability of equipment) gained by the preventive measures, if deployed. This information gives an advantage and maybe even a time saving in the decision for a curative measure. With this basis, authorities can review about the technical, logistical and organizational aspects of a potential curative measure.

Framework

If there is zoomed in on the diamond shapes of Figure 2.1, which are the decision moments where emergency measures are implemented, three critical aspects are important. These three aspects are the technical, the organizational and the logistical (Deltares, 2013), (Lendering, et al., 2014). Literature focuses mainly on the technical and logistical side (Deltares, 2011), (Gerven, 2004), (Resio & Boc, 2011). The *technical* problem consists of the type of dike, the characteristics of the dike and the failure mechanism. These aspects are important for the breach development. If a breach is formed, the development and the flow pattern inside the breach will affect the effectiveness and stability of the emergency measures. Interaction between the breach processes and the emergency measure is the core of the technical aspect. *Logistics* have to do with the time and place of the breach. Time for an effective solution can be very short and the access to the site is often hampered (Laska, 2009). The availability of material and equipment and the implementation of the emergency measure are important aspects of logistics too. Captured in the *organizational* part are the responsibility and tasks of people, manpower, procedures and communication. Another organizational aspect that has not directly to do with the closure of a dike breach is training of the staff. For the closure of a breach the logistical and organizational aspects are at least as important as the technical one. If one aspect is not carried out properly the breach cannot be closed and the closing operation fails.

Problem focus
All the three aspects are essential to get a successful breach closure, however the core of this thesis is the technical part of a breach closure. The logistical and organizational parts are essential; however they will get less priority in this thesis.

The technical side focusses on the physical processes that are crucial in a dike breach situation. The technical part starts with, the type of dike, the characteristics of the dike and the failure mechanism. These aspects have a large influence in the breach development. The flow pattern inside the breach affects the effect and stability of the emergency measures. The core of the technical aspect is the interaction between the breach processes and the emergency measure. The emergency measures are combined with the breach processes and simulations and calculations are done to understand the effect of an emergency measure. Besides the effect, emergency measures depend on their application range. The velocity of the flow in the breach or the dimensions can limit the implementation of the measures. Research into these limits is an important part of this thesis.

2.2 Scope

Time scope
This thesis will especially focus on the time between the moment of the start of a dike breach and the moment that there is no need for an emergency closure anymore. The latter is reached when the water level at the inner side has risen and has become equal to the water level at the outer side of the dike. Only curative emergency measures are investigated, evacuation is not considered. Between the described moments, a breach develops and the necessary actions are attempted to close this breach again. However, it is almost impossible to draw such a firm line in the breaching process and emergency measures to close a breach should be treated as part of this process. This process was already sketched in Chapter 2.1. The part of the process which is within the scope of this thesis is displayed with a red square in Figure 2.1.

Physical scope
The process of a breach is not the only part that should be delimited. The physical part needs a scope too. Breaches can occur in several types of water defences. In this thesis Dutch water defences are considered. Not only primary water defences will be in the scope. Regional water defences are in the scope too. No hydraulic structures are considered. Dunes are neither part of this thesis. Emergency measures for the closure of breaches in lake and river dikes in the Dutch system will be the core of this thesis. Sea defences are not taken into account since the options to apply an emergency measure during a storm in a sea defence do not have much perspective.

However the scope of this thesis is defined within the Dutch borders, also dike breaches from abroad will be investigated. The problems and solutions of the breaches abroad may give inspiration and insight in the processes and may provide valuable lessons. Disregarding the knowledge and experience gained by closing breaches abroad would be a loss of information.

The logistical scope is limited to the materials and equipment needed for the closure and the way to get them to the breach in time. Purchase and accommodation of the materials and equipment is not included in this thesis. As scope for the organizational part, the aspects for the recommendations for the Decision Support System are also taken into account.

2.3 Research questions

The research question should in the first place contain the technical aspect. Effective emergency measures and their applicability should be sounded in the question. Also, the established scope must be included in the question. The basis is understanding the processes in the breach. The research question of this thesis is:

> *"What is the effectiveness and applicability of curative emergency measures, applied in developing breaches in a Dutch river or lake dike?"*

The following sub questions will be answered in this report:

- *"What lessons can be learned from past attempts of the emergency closure of dike breaches?"*
- *"What are the effects and application ranges of curative emergency measures on a developing breach?"*
- *"Is it useful to perform field experiments regarding emergency closures?"*
- *"What recommendations for protocols or procedures can be done to arrive at effective implementation of curative measures?"*

2.4 Objectives

The main objective is to find a robust framework for the design, management and operation of an emergency closure of a dike breach. It is understood why some emergency measures are effective and others are not. In case of a dike breach this framework is able to support a decision to choose an emergency measure. The main objective can be divided in specific key elements, which support the main objective:

1. Distinguish the different critical aspects of a dike breach into separate tangible parts that can be understood.

If the breach process can be split into smaller parts, the critical aspects can be picked out and prioritised. There can be separate investigations for the critical parts.

2. Collect lessons learned from dike breaches and the applied emergency measures.

In the Netherlands in the past several dike breaches occurred. Sometimes emergency measures were implemented. Abroad, even more floods took place and there were attempts to save the dike with emergency measures. The lessons from these floods and the deployment of emergency measures are valuable and will be collected.

3. Understand the effect of an emergency measure and know its application range.

Using simulations and calculations the effect of emergency measures in developing breaches can be investigated. However, these emergency measures can be limited by the high flow velocity or the dimensions of the breach. The measures must fulfil the requirements for stability.

4. Find out if field testing is useful.

Several experiments are done in *Flood Proof Holland*[2]. With these experiments a better insight in the physical and practical aspects can be obtained. After the experiment a statement can be made about the usefulness of experiments for emergency measures in general.

5. Connect the knowledge gained in this thesis to the current practice at Dutch Water Boards.

The last objective is formulated to use the gained knowledge in a real life setting. Connection with the existing flood safety programs at Dutch Water Boards is the goal. Recommendations to make better decisions with the new findings are done.

2.5 Methodology

To fulfil the objectives and answer the research questions, a methodology framework is set up. Figure 2.2 displays this in a sort of time line from left to right. The methodology starts with the research proposal. Thereafter the analysis of the breaches and their processes and characteristics is started, together with the analysis of the available emergency closure measures. If the most critical processes and characteristics are found, case studies are investigated. A reconstruction and hindsight calculations are performed on successful closures. In this way an insight in the essential technical, logistical and organizational set up around these closures are obtained. If the closure was not successful the analysis will be about why it did not succeed.

With the gained knowledge of before mentioned literature aspects of breach processes and emergency measures, simulations and calculations are done. The learned lessons from the case studies are taken into account too. Several promising emergency measures are simulated and their effects are monitored. Stability calculations need to point out if the measures are stable in the high flow velocities.

After the effects stability limits are investigated, field tests are carried out in an experiment in the *'Flood Proof Holland'*. In this outdoor laboratory, innovative flood control measures

[2] *For more information see Chapter 2.5 or the website: http://floodproofholland.nl/*

can be tested and demonstrated. The measures can finally be designed for the logistical and organizational part. When all the results are available the integration of the emergency measure in the current practice at Dutch Water Boards can be carried out. Finally, conclusion and recommendations are drawn.

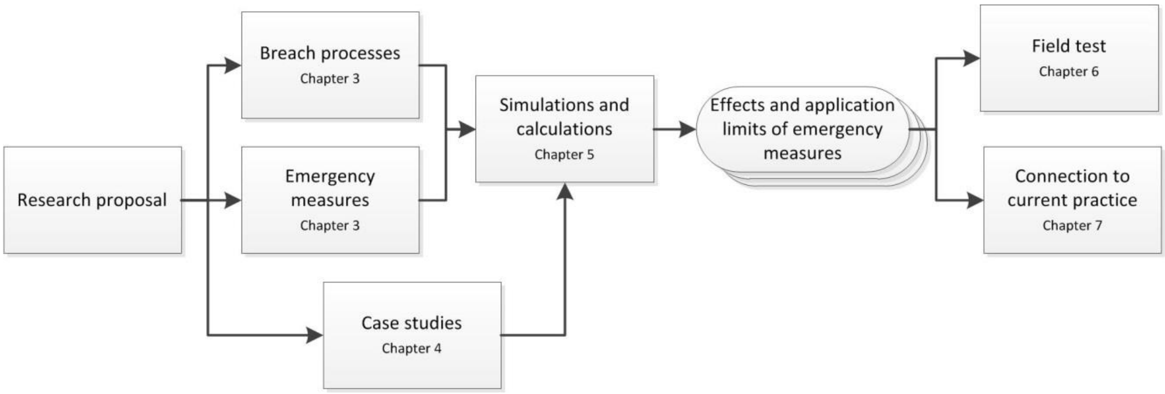

Figure 2.2: Methodology

In this framework the established objectives are covered and the main question of this thesis is comprised. The different critical aspects of a dike breach are distinguished into separate tangible parts that can be understood. There is found why an emergency measure is effective and why some emergency measures are not effective. Applicability limits of the measures are established. With field tests is demonstrated if it is worthwhile to do so regarding emergency closures experiments. The obtained results can subsequently be integrated in recommendations for a decision support system, connecting to the current practice at Dutch Water Boards.

3. Breach processes and emergency measures

3.1 Introduction

In this chapter the breach process and emergency measures are discussed. Despite this thesis focusses on the interaction between the breach process and emergency measures, they will be treated separately in this chapter to make sure both aspects are understood. To gain insight in the effect of the critical aspects of emergency closures, the breach process will be decomposed in characteristics of the dike and characteristics of the breach. The characteristics of the dike have large influences on the development of the breach and thus on the suitable closure method. The emergency measures are selected and checked for their aimed effect. In relation to the possible preparation advantages of preventive measures, they will be investigated shortly and with respect to the curative measures. The goal of this chapter is to distinguish and understand the aspects of the breach process that have influence on the deployment of emergency measures and to exam curative measures for their potential to implement in a dike breach. Also, closure techniques from planned closures are discussed.

3.2 Breach process

The breach process is influenced by several characteristics of the dike. The most important characteristics are treated in this paragraph. The dike type is of influence for the breaching process. Dependent on the consistency of the core, the breach development will speed up or slow down. The geometry of the dike is different for various expected loads. Subsoil beneath the dike body is of importance too. Primary flood defences and regional flood defences are distinguished. Their impact is discussed below in Chapter 3.2.1. Breach development is discussed in Chapter 3.2.2.

3.2.1 Characteristics of the dike

Dike type

In the Netherlands a lot of different dike types are present. For the Netherlands four characteristic dike cores can be summarized from all these types (GeoDelft, 2002). They are displayed in Figure 3.1.

The first type is a dike which consists completely of clay. This is the 'original dike', as it was constructed in the past. Dike number two is a with sand on the inner slope strengthened clay dike. A clay cover is on top of the sand. Dike type three is a sand dike. The

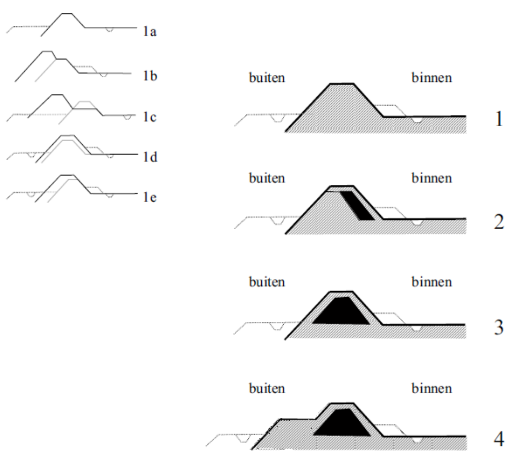

Figure 3.1: Characteristic dike bodies (GeoDelft, 2002)

body of this type consists of sand and is covered with a clay layer. The last dike type is a large reinforcement where the old clay dike is used as an outer berm for the new dike. The new dike consists of a sand core with a clay cover. The first type of dike is usual an old lake dike. New polder dikes are sand dikes like type three. Also repaired dikes after breaches can be grouped under type three. Sea dikes are often sand dikes with an old clay core, like type two and four. All the types can have geometries as displayed on the left side of Figure 3.1, labelled a to e.

Geometry

For the geometry of dikes, three main forms are distinguished; geometry of sea, river and lake dikes. Here, geometry of river and lake dikes will be discussed since sea dikes are not in the scope of this thesis. The dikes have different geometry since the loading is different per situation. Loading on a lake dike is triggered by a storm. This loading situation is of relatively short duration and to a high degree characterized by wave attack. Here, extra attention is given to the outer slope, crest height and revetment. A river dike is exposed to the high water load for a longer time. For this reason attention is primary paid to the inner slope and a berm to counter piping. The geometry of a lake dike is comparable with a sea dike (Weijers & Tonneijck, 2009). In Figure 3.2 schematic profiles of sea, lake and river dikes are displayed. The thick line indicates revetment.

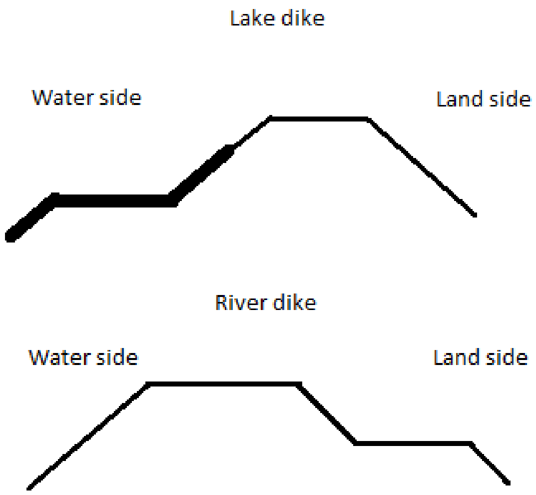

Figure 3.2: Schematic view of lake and river dike

Revetment

Revetment plays an important role in breach development. A good revetment is able to slow down the breach process. Most common revetments used in dikes are grass, rock, placed blocks and asphalt. The type of revetment depends on the dike type and on the geometry. Combinations that are often used are placed blocks with filter on clay, grass on clay and asphalt on sand (GeoDelft, 2002).

Subsoil

In the Netherlands the subsoil consists of sand deposits sometimes covered with weak clay and/or peat layers. The geological situation of the Netherlands is variable throughout the country. In the western and north-western part of the Netherlands Holocene clay and peat layers of considerable depth are present. In the central, eastern and southern part of the Netherlands the Pleistocene sand layers are close under the surface. In the lower river area the dikes are founded on a thick layer of clay or peat. Failure mechanisms that occur the most in this in the lower river area are macro instability of the inner slope, flotation and overtopping. In the upper river area the sand layers are closer to the surface. Failure mechanisms that occur the most in the upper river area are piping, macro instability of the inner and outer slope, micro instability and overtopping (GeoDelft, 2002).

Category

In the Dutch safety system a distinction is made between primary and regional flood defences. The definition of primary water defences is, a water defence which provides protection against flooding and is part of a dike system, which may be connected to natural high grounds or it is situated in front of a dike system (Ministerie van Verkeer en Waterstaat, 2007b). All not primary water defences are regional defences. A difference between the mentioned defences is the safety level. Primary flood defences have a way stricter exceedence frequency, since a breach would cause more damage. The probability of breaching of a dike is larger for regional flood defences since they have a smaller exceedence frequency. Another difference is in the loads. Loads on primary flood defences are due to storms at sea or high water waves at rivers. For regional flood defences the load is caused by an excess of rain. Primary water defences have an 'unlimited' amount of water that could discharge through the breach. Regional flood defences have a limited amount of water that could discharge through the breach. The water level will noticeably drop if a regional flood defence breached.

Flood wave

High water on a river is caused by a flood wave. This wave can last up to a couple of weeks. As the high water wave passes, the water level drops. The drop in water level can trigger instability of the outer slope, because the dike body is saturated. Probably, this is not immediately a problem since the water did already drop and does not overtop the dike. A problem occurs if the dike is still damaged and a second high water wave takes place. The failure probability is in that case increased. Damage could occur to the already damaged outer slope. Other failure mechanisms could occur easier because of the initial damage and a higher phreatic line in the dike body at the moment of the start of the second high water wave.

Failure mechanism

The type of failure mechanism determines the start of the breach. For the three most common failure mechanism an analysis is done about the formation of the initial breach. Overtopping, piping and macro instability of the inner slope are selected to be investigated. These were the three most occurred failure mechanisms following from an historical analysis of dike breaches in several countries (Vorogushyn, et al., 2009). From the analysis was found that for every failure mechanism the initial breach is formed at the top of the dike and the initial damage must be large enough for soil particles to be carried away by the currents. The shape of an initial breach is assumed as a trapezoidal cross section with side slope angles γ about equal to the angle of internal friction φ (Visser, 1998), (Zhu, 2006), see also Figure 3.3. However, different failure mechanisms do result in small differences in the initial breaches. For overtopping the failure takes place as described above, with a single gully. Macro instability results in the failure of large lumps of soil, just as with the piping mechanism. Due to the (large amount of) soil that is slipped off the dike body is weakened. For this reason the erosion in stage III can take place quickly for these mechanisms. For the complete analysis see Appendix I.

3.2.2 Characteristics of the breach

Breach development in non-cohesive dikes (Visser, 1998)

For breach development in sand dikes the model BRES is developed. A few assumptions are done in this model. Wave influences are not taken into account in the model. In case of a cover of cohesive material, there is assumed that the initial damage did uncover the sand core of the dike. It is assumed that the remaining cohesive parts will not slow down the breach erosion process. The initial breach in which the water starts to flow is assumed at the top of the dike. The shape of the initial breach is assumed trapezoidal, with side angles γ assumed equal to the internal angle of friction φ. It is assumed that the area of the initial breach is large enough for the water to flow through and start the breach process.

The breaching process in sand dikes is distinguished in five stages. In stage I, erosion of the inner slope by the overflowing water causes the initial slope angle β_0 to increase. Stage I is defined from the start $t=t_0$ up to the moment when β reaches a critical value β_1 at $t=t_1$. Retrograde erosion of the inner slope at the constant critical slope angle β_1 happens in stage II. The width of the crest of the dike body decreases. This stage ends at $t=t_2$ when the crest vanishes and the breach inflow starts to increase. In the third stage the top of the dike lowers into the breach. The breach is widened by regressing side slopes with critical slope γ_1. The dimensions of the breach increase and this stage ends at $t=t_3$, when the dike is completely washed out down to the base of the dike at polder level. In stages I, II and III the initial breach cuts itself into the dike. The initial breach is displayed in Figure 3.3. The development during the stages I, II and III is displayed in Figure 3.4.

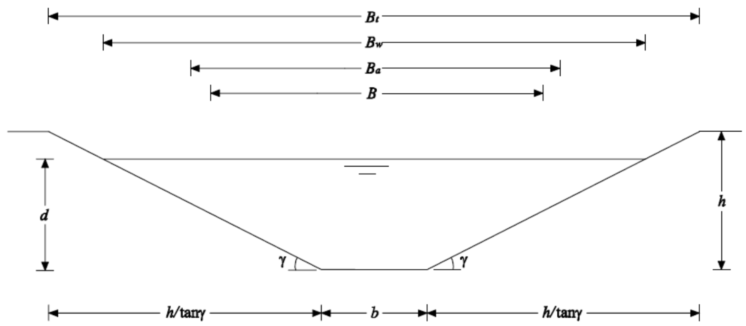

Figure 3.3: Cross section of initial breach in the crest (Zhu, 2006)

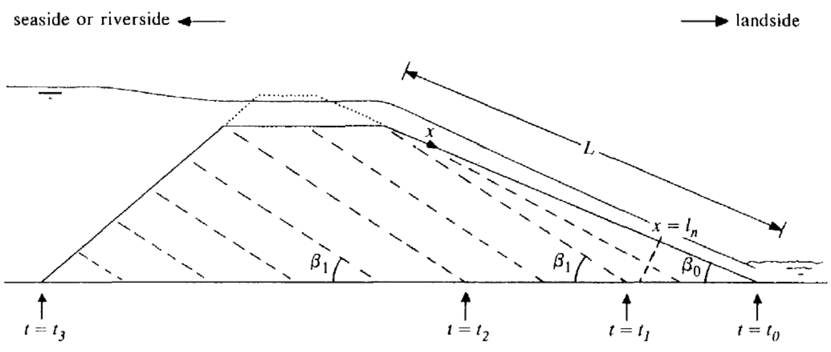

Figure 3.4: Development of the breach in stages I, II and III (Visser, 1998)

Critical flow is reached in stage IV. The water flowing through the breach is virtually critical and the breach grows mainly laterally where the side slopes still have the critical angle of γ_1. Dependent on the erodibility of the dike base, the breach grows with a certain speed in the vertical direction. The flow changes from critical ($Fr = 1$) to subcritical ($Fr < 1$) at the end of stage IV at $t=t_4$. The flow is subcritical in stage V. The breach develops in the same way as in stage IV, with the difference that the growth is influenced by the backwater curve. This means that the flow velocities become smaller and the growth rate decreases. At $t=t_5$ the flow velocities are so small that the breach erosion stops. At $t=t_6$ the water level in the polder has equalled the outside water level and the flow through the breach stops. Most of the discharge through the breach takes place in stages IV and V. The five stages and their characteristic appearance are visible in Figure 3.5.

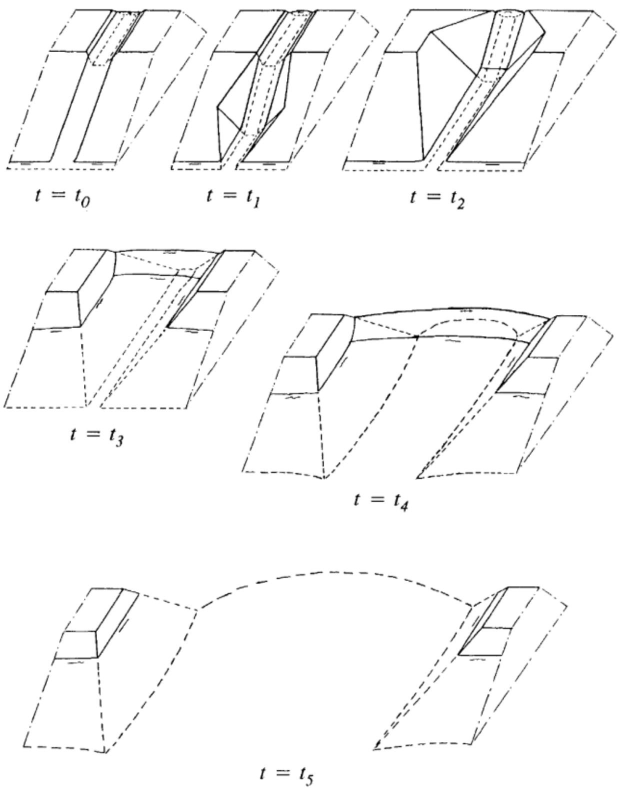

Figure 3.5: Schematic illustration of breach growth in sand-dikes (Visser, 1998)

After stage III the continuation of the breach erosion process depends on three aspects. The resistance of the dike base against the erosion can hamper the breach growth. Two other influences are the presence of a toe protection and the presence of a high foreland. Three types of breaches can be distinguished after stage III, dependent on the mentioned conditions.

Type A is a breach that has a dike base which consists of a solid clay-layer and a toe construction at the outer slope of the dike, see Figure 3.6. If a breach occurs in this type of dike, the vertical erosion is hampered by the clay base and the toe protection at the outer slope. This is the most 'favourable' situation for a dike breach. The formation of a scour pit occurs at the downstream side of the dike. Reconstruction of the dike can take place at the same place as the original dike.

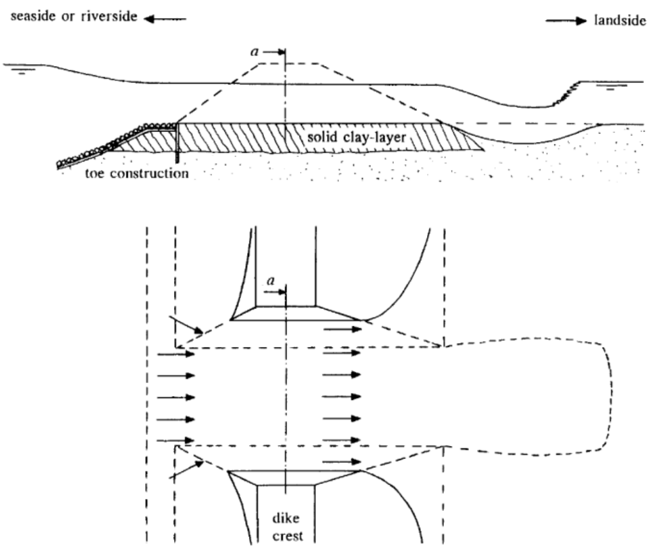

Figure 3.6: Type A breach (Visser, 1998)

A type B breach grows in the lateral and vertical direction. Because there is no clay layer or toe protection present, the erosion can precede without hampering. In a type B breach, there is a high foreland present. However, this foreland can erode away in contrast to the solid clay layer of type A. The scour hole will be formed at both the upstream and the downstream side. In Figure 3.7 a type B breach is displayed.

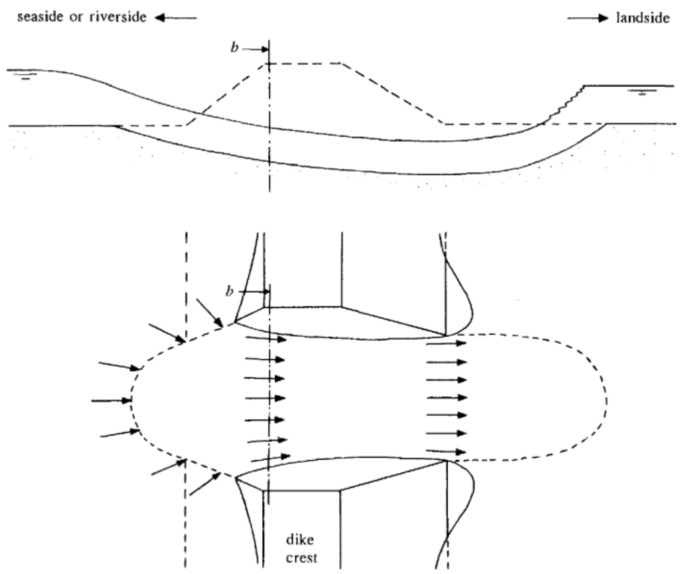

Figure 3.7: Type B breach (Visser, 1998)

A type C breach grows just as a type B breach in the lateral and vertical direction due to the lack of resistance against erosion. The difference between type B and type C is that a type C breach does not have a high foreland. A Type C breach is displayed in Figure 3.8.

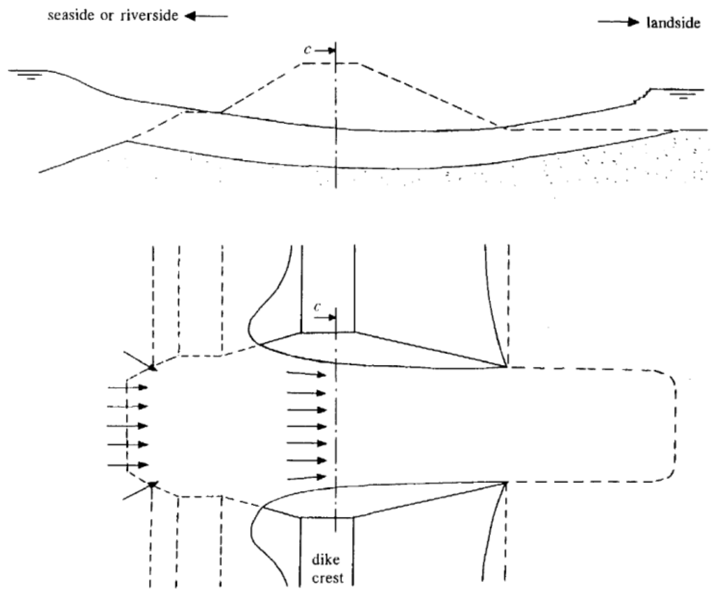

Figure 3.8: Type C breach (Visser, 1998)

Breach development in cohesive dikes (Zhu, 2006)

For the breach process of cohesive dikes a mathematical model is developed. The model assumes homogeneous cohesive dikes. Similarity can be seen with the breach process of non-cohesive dikes. Five stages are distinguished for cohesive dike breaching too. Assumptions about the start of the breach, the shape of the initial breach and the neglecting of waves are the same as for the non-cohesive dikes.

The first stage starts at $t=t_0$. Erosion starts, induced by the flowing water, at the inner slope and the crest. More erosion takes place at the toe of the inner slope than at the upper part of the inner slope. Because the lower part of the inner slope erodes more than the upper part, the inner slope steepens. Stage I ends at $t=t_1$, at that time the gradient of the inner slope has increased from the initial value β_0 to the critical value β_1. The critical angle is a value dependent on the properties of the dike material but will be assumed at 80° - 90°.

Stage II starts at $t=t_1$ with the critical slope angle β_1. This angle stays the same throughout stage II. The erosion also lowers the dike crest and the discharge through the breach increases. Different mechanisms combined cause the breach to erode further: flow shear erosion, fluidization of the slope surface, scour of the dike foundation and slope mass failure. Stage II ends when the head cut retreats to the outer slope at $t=t_2$.

In stage III the breach develops faster, but in the same way as in stage II. The dike body is thinner and weaker than in stage II. By the faster development of the breach, the discharge increases too. This will make the breach develop faster which, in turn, will increase the discharge, and so on. The angle of the slope remains the critical angle of β_1. At the end of stage III, $t=t_3$, the dike body is eroded away completely.

Erosion in stage IV takes place mainly in the lateral direction. The flow shear erosion along the side slopes results in erosion and thus side slope instability. This is the main failure mechanism which causes the breach to develop in lateral direction. Vertical erosion depends on the properties of the dike. The same three types, A, B and C as in the non-cohesive dikes are distinguished. The inner water level at the $t=t_4$ starts to influence the breach flow. At this point stage IV ends.

The development of the breach in stage V is the same as in stage IV. The rate of the erosion and the discharges decrease, since the water at the inner side decreases the breach flow. At the end of stage V, at $t=t_5$, the flow velocity decreased so much that no erosion takes place anymore. The erosion process has stopped, however, water still flows through the breach. At $t=t_6$ the water level at the outer side reached the level of the water at the inner side and the flow through the breach stops. The five stages are displayed in Figure 3.9.

<u>Mathematical model</u>

Stage I, II and III

The breach can be assumed as a broad-crested weir during the erosion process. Figure 3.3 displays an initial breach. B is the average breach width over the water depth d, B_a is the average breach width over the breach depth h, B_w is the breach width at the water surface, γ is the side slope angle, b is the breach width at the bottom of the breach and B_t is the breach width at the crest.

$$B = b + d/\tan\gamma \qquad \text{[m]} \qquad (3\text{-}1)$$

$$B_t = b + 2h/\tan\gamma \qquad [\text{m}] \qquad (3\text{-}2)$$
$$B_w = b + 2d/\tan\gamma \qquad [\text{m}] \qquad (3\text{-}3)$$
$$B_a = b + \text{h}/\tan\gamma \qquad [\text{m}] \qquad (3\text{-}4)$$

The cross sectional area A of the breach flow and the hydraulic radius R are:

$$A = Bd \qquad [\text{m}^2] \qquad (3\text{-}5)$$

$$R = A/(b + 2d/\sin\gamma) \qquad [\text{m}] \qquad (3\text{-}6)$$

The discharge Q_{br} through the breach is calculated by:

$$Q_{br} = m\left(\frac{2}{3}\right)^{3/2}\sqrt{g}B(H_w - Z_{br})^{3/2} \qquad [\text{m}^3/\text{s}] \qquad (3\text{-}7)$$

In which m is the discharge coefficient ($m \approx 1$ for stage I, II and III), g the gravity acceleration, H_w the outer water level and Z_{br} the height of the breach bottom. H_w and Z_{br} are measured above a reference level of $Z = 0$. Equation 3-7 is also valid for the stages II, III and IV.

In the first three stages of the breach flow the breach at the crest is enlarged by the erosion of the flow. The gradient of the breach side slopes increases. The formula for the angle increase dγ in one time step dt is expressed as:

$$\cot(\text{d}\gamma) = \tan\gamma + \frac{2h}{E_{bo}\cdot\sin 2\gamma\cdot \text{d}t} \qquad [-] \qquad (3\text{-}8)$$

E_{bo} is the rate of flow shear erosion in vertical direction at the breach bottom. M_e is a material dependent factor describing the erodibility of the soil. τ_b is the actual bottom shear stress and τ_c is the critical bottom shear stress.

$$E_{bo} = M_e(\tau_b - \tau_c) \qquad [\text{m/s}] \qquad (3\text{-}9)$$

The breach width at the dike crest (B_t, see Figure 3.3) does not increase due to erosion until the breach side slopes reach the critical value of β_1. ΔZ_{br}' is the extra depth needed for the bottom of the breach, to arrive at the critical angle for the side slopes of β_1 (see Figure 3.10).

$$\Delta Z_{br}' = \frac{h\sin(\beta_1 - \gamma)}{\sin\gamma\cos\beta_1} \qquad [\text{m}] \qquad (3\text{-}10)$$

Any erosion deeper than this level induces too steep breach side slopes and slope instability will occur. The breach width at the crest will increase by this failure mechanism. This instability occurs very quickly and the failed soil material is swept away by the currents immediately. The fallen material is assumed to have no influence on the breach process. Side slope instability has nothing to do with the breach growth at the bottom. The breach growth is fully determined by the flow shear erosion at the breach bottom. The breach width increase per time step is then represented by a continuous curve modelled in the mathematical model as:

$$\frac{\text{d}B_t}{\text{d}t} = \frac{2E_{bo}}{\tan\beta_1} \qquad [\text{m/s}] \qquad (3\text{-}11)$$

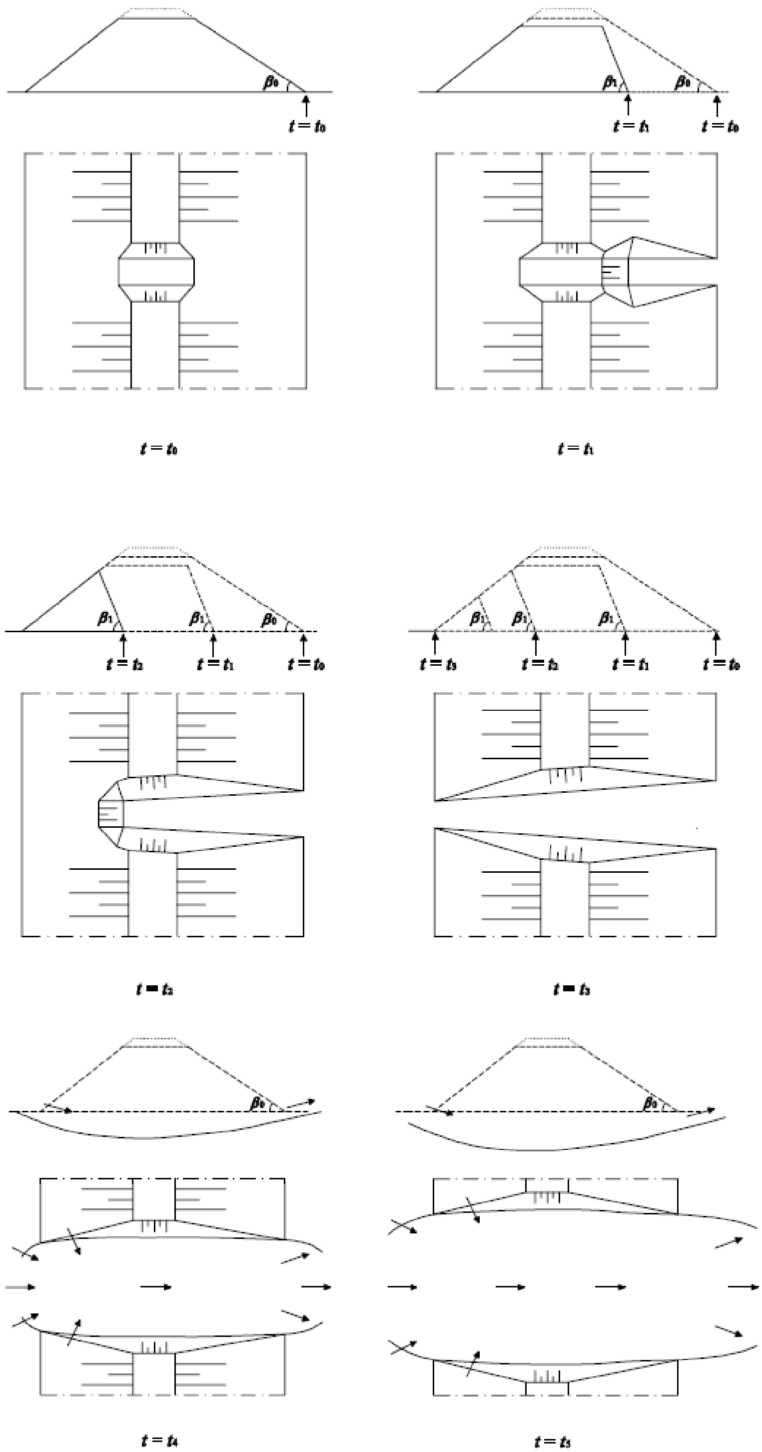

Figure 3.9: Breach development in cohesive dikes (Zhu, 2006)

Figure 3.10 displays the breach development of the widening of the breach in stage I, II and III. In (a) the steepening of the side slopes is displayed. By the erosion of the bottom, the angle of γ increases. The critical angle is reached in (b). Vertical erosion over a distance of ΔZ_{br}' was needed to reach this angle. In (c) breach enlargement after the arrival at the critical slope is shown. Any erosion after the angle β_1 is reached causes a too steep slope. This is compensated by slope failure due to instability.

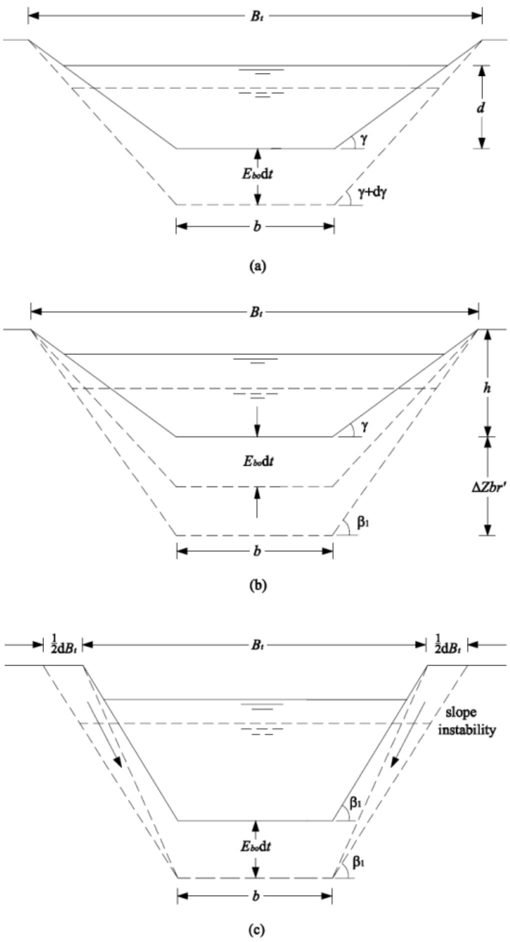

Figure 3.10: Breach width development during stages I, II and III (Zhu, 2006)

Stage IV and V

After the complete washout of the dike body, the subsequent breach development is dependent on the geotechnical conditions and characteristics of the dike. Distinction is made in types A, B and C, as discussed above. In Figure 3.11, the breach development in the stage IV and V is displayed.

In a type A breach the vertical erosion, E_{bo}, can be neglected since a solid clay layer is present. Only erosion at the side slopes erode in a type A breach. The breach development in case of a type A breach can, after some mathematical computations be written as:

$$\frac{\mathrm{d}B_t}{\mathrm{d}t} = \frac{2dE_{sl}}{\sin\beta_1(d+E_{sl}\mathrm{d}t\cdot\cos\beta_1)} \quad [\mathrm{m/s}] \quad (3\text{-}12)$$

In which E_{sl} is the rate of flow shear erosion at the toe of the breach side slope, perpendicular to the slope.

$$E_{sl} = M_e(\tau_b - \tau_c) \quad [\mathrm{m/s}] \quad (3\text{-}13)$$

Equation 3-12 can be simplified if time step dt is chosen small enough. Then $E_{sl}\mathrm{d}t \cdot \cos\beta_1 \ll d$ and Equation 3-6 becomes:

$$\frac{\mathrm{d}B_t}{\mathrm{dt}} = \frac{2E_{sl}}{\sin\beta_1} \qquad [\mathrm{m/s}] \qquad (3\text{-}14)$$

The discharge in a type B breach is approached with the same equation as for the stages I, II and III. The difference between the discharge formula for the stages I till III and stage IV is the discharge coefficient m. The shape of the spillway determines the amount of inflow. For a type B breach the backwards erosion forms a curved spillway, with a length larger than b. The discharge coefficient which corresponds with a curved elliptical spillway is in the order of π/2.

$$Q_{br} = m\left(\frac{2}{3}\right)^{3/2}\sqrt{g}B(H_w - Z_{br})^{3/2} \qquad [\mathrm{m^3/s}] \qquad (3\text{-}15)$$

For a type B breach the vertical erosion can be neglected too. This is true because the vertical breach growth is way smaller than the lateral growth.

As the vertical erosion is neglected, the time step is chosen small enough and some mathematical operations, the formula for erosion in type B can be simplified to:

$$\frac{\mathrm{d}B_t}{\mathrm{dt}} = \frac{2E_{sl}}{\sin\beta_1} \qquad [\mathrm{m/s}] \qquad (3\text{-}16)$$

For a type C breach, a same procedure can be followed. The erosion in lateral direction can be described by the same formula as a type B breach, see Equation 3-16. However, in a type C breach vertical erosion has a not neglectable influence. The vertical erosion can be described with:

$$\frac{\mathrm{d}Z_{br}}{\mathrm{d}t} = E_{bo} \qquad [\mathrm{m/s}] \qquad (3\text{-}17)$$

With:

$$E_{bo} = M_{ef}(\tau_b - \tau_c) \qquad [\mathrm{m/s}] \qquad (3\text{-}18)$$

M_{ef} is a material dependent factor describing the erodibility of the dike foundation.

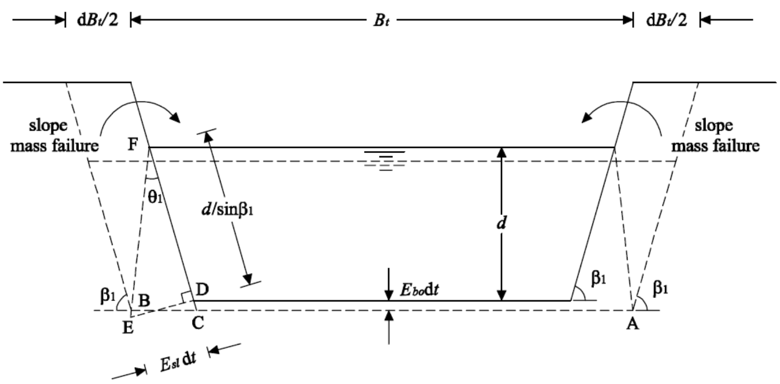

Figure 3.11: Breach development during stages IV and V (Zhu, 2006)

At some point the water that flowed into the hinterland reaches a level where it will affect the breach flow. The inner water level will reduce the amount of flow through the breach. The breach flow is than changed from free to submerged. This happens if the inner water level H_p has risen to a level that:

$$H_p - Z_{br} = \frac{2}{3}(H_w - Z_{br}) \qquad [\mathrm{m}] \qquad (3\text{-}19)$$

Increase of the inner water level is described by:

$$\mathrm{d}H_p = \frac{Q_{br}\mathrm{d}t}{A_p} \qquad [\mathrm{m}] \qquad (3\text{-}20)$$

In which A_p is the storage area of the polder.

In Stage V the discharge is influenced by the inner water level, in stage V is described by:

$$Q_{br} = m\sqrt{2g}B\left(H_w - H_p\right)^{1/2}\left(H_p - Z_{br}\right) \qquad [\mathrm{m^3/s}] \quad (3\text{-}21)$$

The velocity of the breach flow in stages I, II, III and IV is described by:

$$U = \frac{Q_{br}}{Bd} \qquad [\mathrm{m/s}] \qquad (3\text{-}22)$$

The velocity of the breach flow in stage V is described by:

$$U = \sqrt{2g(H_w - H_p)} \qquad [\mathrm{m/s}] \qquad (3\text{-}23)$$

Development of the breach for the types A, B and C are described by the same formulas as in stage IV.

Currents in a breach

The currents in a breach are complex and irregular. Flow velocities differ as well in the flow direction of the breach as in the transverse and vertical direction. Since stability calculations are done with velocities, the flow profile in a breach has to be investigated so a distinction can be made between the averaged velocity and peak velocity.

Figure 3.12 displays the flow velocity of a breach in flow direction. The observed velocity profile has its peak velocity near the slopes and the velocity decreases somewhat in the direction of the centre. The measured velocities are in meters per second. There is assumed that the shape of the velocity profile is the same for other velocities. It can be noted that the velocity in the centre is approximately 90% of the peak velocity. This is the case for the initial and the final stage.

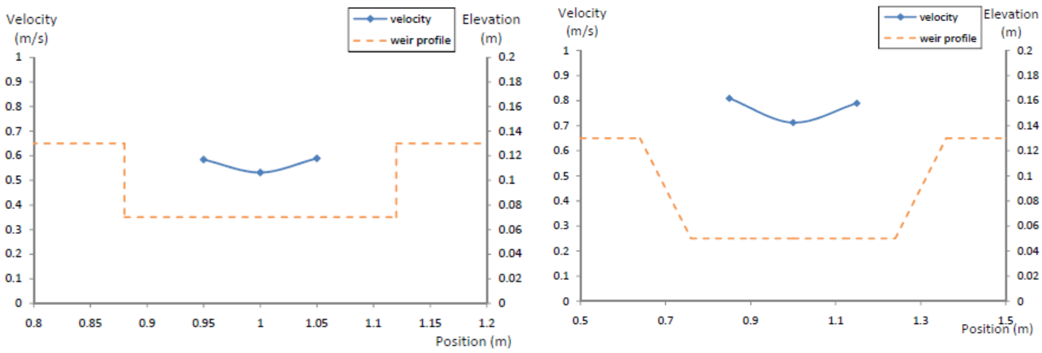

Figure 3.12: Velocity profile in the flow direction at the first stage of the breach (left) and the final stage of the breach (right), (Ren, 2012)

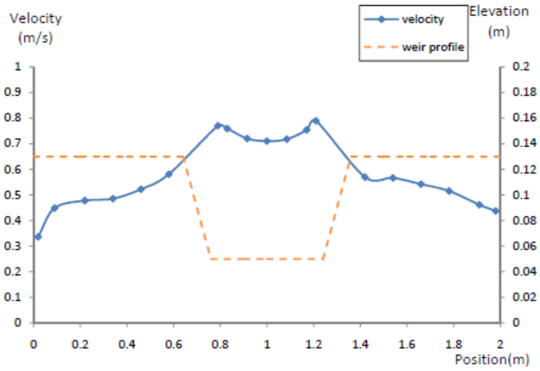

Figure 3.13: Velocity profile in the flow direction with an overtopped weir, (Ren, 2012)

Figure 3.13 shows the velocity profile in the flow direction with an overtopped weir. If this Figure is compared to Figure 3.12, the same shape of the velocity profile can be observed. The smallest velocity in the breach is close to the slopes. Towards the centre the velocity increases where after the velocity drops in the centre.

The average velocity over the breach is about 90% of the peak velocity. The velocity in the centre of the breach is equal to the averaged velocity. The measured velocity in the centre can be used as average velocity over the breach.

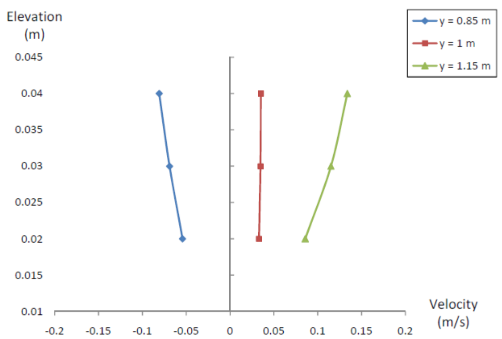

Figure 3.14: Velocity profile in transverse direction, (Ren, 2012)

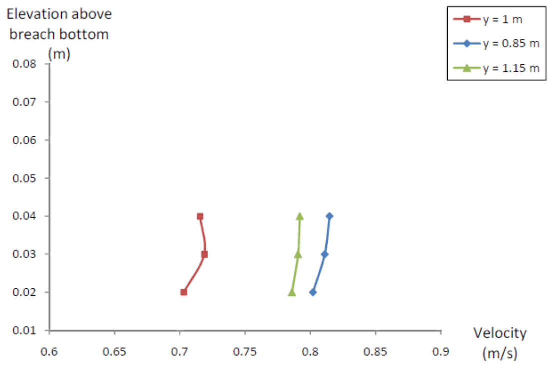

Figure 3.15: Vertical velocity profile in the flow direction, (Ren, 2012)

Figure 3.14 displays the velocity profile in transverse direction. The magnitude of these velocities is much smaller. The figure shows that the transverse flow velocity is towards the slopes.

Figure 3.15 displays the vertical velocity profile in the breach in the flow direction. The velocity at the surface of the water is larger than in the direction of the bottom. The differences are however small.

3.3 Emergency measures

There is no uniform definition for emergency measures. In this thesis a distinction is made between control measures, preventive emergency measures and curative emergency measures. All are called emergency measures interchangeably throughout the literature. In this thesis, control measures are defined as a measure, prepared beforehand, for a specific known situation when there is no case of an emergency. The locations where the control measures need to be placed are known due to inspections or assessments and the placement can be prepared in advance. Emergency measures are unprepared and site specific. These are applied after an in situ inspection of the dike and are unknown beforehand and thus unprepared (Lendering, et al., 2014). Control measures and preventive emergency measures are applied in the phase before the formation of a breach; the preventive phase. Preventive emergency measures are put in place to avoid the formation of a breach and curative measures are used to limit, reduce or counter the breach growth. When in this thesis is spoken about an emergency measure, a curative measure is meant. Figure 3.16 displays the measures in the process.

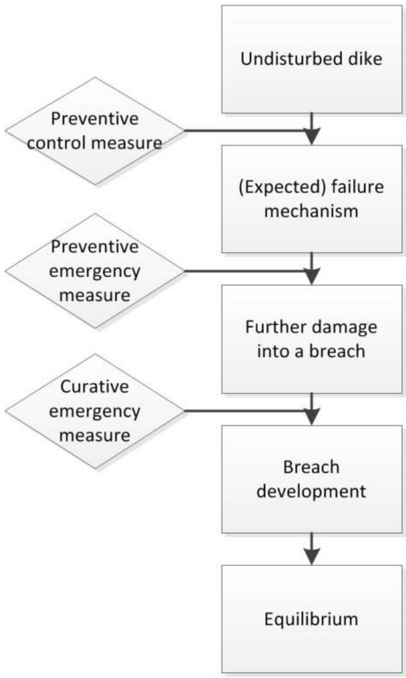

Figure 3.16: Definitions preventive and curative emergency measures

3.3.1 Preventive emergency measures

In this paragraph the extension of the preventive measures with respect to the curative measures are discussed. For a more complete overview is referred to Appendix I.

Overtopping: extension to curative measure

None of the technical measures to prevent overtopping are suitable to limit, reduce or counter breach development. Material and equipment used for the measures to raise the retaining height are transported via the land. The transport modes and routes to weak places at the dike can be established and prepared. Also, trucks are available to drive to the emergency location.

Piping: extension to curative measure

Again the technical measures of sand bags or sand as piping berm are not able to function as a curative emergency measure. For the piping berm however large amounts of sand are needed. It is therefore possible to establish and prepare the transport routes to the weak places. Trucks that are able to carry sand are available to use for transportation of an emergency measure.

Macro instability inner slope: extension to curative measure

The used material for the berm is insufficient to counter the breach development. For the berm however large amounts of sand are needed. It is therefore possible to establish and prepare the transport routes to the weak places. Trucks that are able to carry sand are available to use for transportation of an emergency measure.

3.3.2 Curative emergency measures

If the breach is initiated, preventive measures are not effective to stop the breach. Other measures are needed to close the breach again. These are curative measures, thus applied to 'cure' the breach. Below, six measures are discussed on their technical, logistical and organizational advantages and disadvantages. Also the experiences with the measures are written down, if there are any at all. From literature the following measures are discussed:

Ships and barges

Technical

The technical idea of using a ship or barge to close a breach is to shut off the breach at once. The ship closes the gap fully. For this reason the ship or barge has to have a larger length than the breach and must be higher than the depth of the breach. A way to implement a ship in the breach is to sail the prow of the ship the dike next to the breach. After this action, the currents sweep the stern across the breach into the dike body on the other side of the breach, like the gate of a lock (Deltares, 2011). Next, the vessel is sunk down with explosives. A requirement, to let a ship become effective as an emergency measure, is a flat, stable bottom. A protected berm and solid clay layer are examples of a stable bottom. If this is not the case, the water will flow underneath the ship and erodes away the soil resulting in a larger breach over time. Often it is due to the currents not possible to place any erosion protection in the breach or to perform any re-profiling of the gap, to fit the vessel's shape. If a ship is placed successfully, the danger is not stopped yet. Since a large hydraulic head over a small horizontal distance is formed, piping becomes a problem. Piping channels underneath the ship can form and threaten the stability. The implementation of a ship is still seen as a lucky shot, since a lot of attempts failed (Joore, 2004).

Figure 3.17: Ship as an emergency measure, source: Nationaal Archief/Spaarnestad Photeo/ANP©

It is also possible to use sunk down vessels or barges to reduce the discharge through the breach. Then another emergency measure is able to close off the breach. This is not an emergency measure on itself, however it supports one.

Logistics and organization

First of all, there need to be a ship close enough to the breach to get there in time. If no ship is available it is not possible to implement this measure. This is also true if the ship is not suitable for the ad hoc measure because of the wrong dimensions. Next to the availability of the ship there are people needed to sail the ship into the breach with its high flow velocities. This requires manoeuvre skills and a crew willing to do so. The ship is property of someone, who might not be willing to give his ship away (Deltares, 2011).

Experience

Several attempts have taken place to close a breach with a ship. Most of them failed. In Chapter 4.2.1 the case of Nieuwerkerk aan den IJssel, the Netherlands, 1953 is described, for the details reference is made to this chapter. This is one of the few attempts that were successful, which were considered as lucky shots. In Chapter 4.2.4 the case of Fischbeck, Germany, 2013 is described, where the implementation of ships did not lead to successful closure. For more details see this chapter.

In Jiujiang, China, 1998, use is made of sunken down ships to reduce the discharge. In Chapter 4.2.2 this is discussed.

Potential for a ship as emergency measure
Ships could be used as an emergency measure, however there are some essential requirements to let a ship be effective as emergency measure. *Technically,* the biggest issue is a solid and levelled breach bed. Besides, the dimensions of the ship must fit the breach dimensions. The availability of a ship, within a range small enough, with a crew able and willing to sail the vessel into the breach are the *logistical* and *organizational* critical aspects. Also, the material to close off the gaps between the ship and the dike need to be present.

Caissons
Technical
Caissons are large, floating, boxes made of concrete or steel. They need to be fabricated before the implementation and are moved with help of tugs to put them in place, since they are not self-propelled (Deltares, 2011). Caissons have technically seen the same effect as ships and barges, however caissons can be linked together to form a larger chain. They are placed parallel to the dike, often with more caissons in a row. The last caisson that completes the closure is difficult to implement since the flow velocities are high. For this reason sluice caissons are used. These are caissons with open parts where the water can flow through. When all caissons are placed they are closed at once. The same requirement, as for the ships and barges, a flat, stable bottom holds (Joore, 2004). If this is not present, seepage flow will induce piping and the caisson will fail. There is no time to place bottom protection.

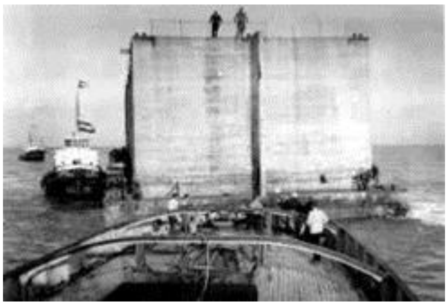
Figure 3.18: Example of a caisson

Logistics and organization
Usually, caissons are used to perform planned closures of estuaries from the sea. These closures can be prepared beforehand. Closure with caissons needs a large preparation. Bottom protection and a flat stable bed need to be prepared. The caissons have to be constructed before the emergency. The construction of a caisson has to be done before and they must be available immediately. Equipment in the form of tugs that bring the caissons to the breach is another logistical aspect that must be taken care of. As emergency measure it can be stated that a lot of logistical and organizational difficulties occur that have to do with the preparation, transport and placement of the caissons (Huis in 't Veld, 1987), (Verhagen, et al., 2012).

Experience
Several planned closures were performed with caissons in the province of Zeeland, the Netherlands. The gaps that caused the inundation of the island Walcheren in 1944 were closed with caissons. Also, the gaps that remained after the Big Flood in the Netherlands in 1953 were mainly closed with caissons (Rijkswaterstaat & KNMI, 1961). Large preparations proceeded before the implementation. This is even more the case for the caisson closures from the Delta works. An important preparation task is to make sure there are enough

caissons to use. After the flood of 1953 use was made of a 'unity-caisson'. This is a caisson with set dimensions, so it could be implemented in every breach. With the order of the 'unity-caisson' a lot of time was saved.

Potential for caissons as emergency measure
Caissons have potential as emergency measures for situations where the breach is larger than a single element (ship, caisson). As an emergency measure that should be applied immediately the *logistical* and *organizational* aspects require too much time. It takes a long time to bring the caissons in place and sink them down. Then the fabrication is not even mentioned. The *technical* aspects are comparable with the ones of ships and barges; a solid and levelled breach bed is needed.

Big bags
Technical
Big Bags are bags made of geotextile and have a volume of 0.3 to 2 m^3. They are filled with sand or other fill material with high density. The bags have loops on top so they are easy to carry by equipment. When the Big Bags are filled they are sewed together to prevent the fill material to be washed out (Deltares, 2011). Often Big Bags are placed in the breach from the dike heads towards the centre. In this way a horizontal closure is carried out. The way a Big Bag can reshape after placed on the bottom is favourable for a water tight closure. For a complete closure a lot of Big Bags are needed. A closure with Big Bags will therefore take a lot of time.

Figure 3.19: Big Bag

Logistics and organization
The placement of Big Bags is done by helicopters. Since the weight that a helicopter is able to carry and the limited amount of helicopters the placement of sandbags is a time consuming job. It is examined that the use of Big Bags placed by helicopters to close the breach is only efficient in the first stages of breach development (Gerven, 2004). The weather might be causing a problem too. If there is a storm, helicopters are not able to be in the air or carry Big Bags. For the placement of Big Bags a specialistic team is needed.

Experience
In Chapter 4.2.3 the closure with Big Bags in New Orleans is discussed. This closure failed due to Big Bags that had not enough weight and were placed in a horizontal closure method. During the emergency closure in Fischbeck, Germany, Big Bags were (unsuccessfully) used to fill up the gaps between the barges, see Chapter 4.2.4.

Potential for Big Bags as emergency measure
Big Bags have some good characteristics as emergency measure. *Technically* they are able to reshape after placement on the bottom, this is favourable for a water tight closure. The problems are in the *logistics* and *organization*. There is a limited amount of helicopters and they have limited carrying capacity. This makes the implementation of Big Bags applicable for probably the first stage of a breach only.

Scaffold

Technical

A scaffold is made up out of piles. These piles are made of wood or steel. The scaffold is constructed from both dike heads towards the centre. This framework has as function to prevent elements from washing away. These elements could be rocks, sandbags or other material available, thrown in after the completion of the scaffold. In this way a vertical closure is performed. The material closes off the breach from the bottom upwards. This closure method has as advantage that during the heightening of the sill, the flow pattern will change from an imperfect weir to a perfect weir. During this transition the flow will become critical and after this stage the flow velocity will decrease (Verhagen, et al., 2012). This is favourable for a closure.

Figure 3.20: Construction of a scaffold, (Rage of the River Gods, 2001)

Logistics and organization

The construction of a scaffold is a labour intensive task. If the scaffolding needs to be constructed in situ, it can take a lot of time. The construction of the scaffolding in the breach itself is difficult because of the high currents. This solution requires sufficient manpower willing to operate in dangerous situation. If the scaffolding was constructed as prefab element, it could be transported by helicopter. The weight would not be a problem if the scaffolding is constructed from light material, however, the weather might be. Next to the scaffolding itself there must be material present to fill up the breach.

Experience

This method is common in China. In Jiujiang, see Chapter 4.2.2, a scaffold was implemented successfully to close a breach.

Potential for a scaffold as emergency measure

The method of closure with a scaffold is *technically* seen an effective one. Because of the vertical closure, the flow velocity will decrease at a certain point in time. A withdraw is the time to construct the scaffold in situ. This is a dangerous and labour intensive job. With a prefab scaffold, this problem about the *logistical and organizational* aspects is decreased. There must be made sure that sufficient material is available to perform the closure, after the placement of the scaffold.

Emergency dike

Technical

In the breach itself, the flow velocities are large. It is difficult to implement a measure in these currents. Another option is to construct a half-circle shaped emergency dike on the upstream side around the scour hole. For this measure a solid high foreland needs to be present. Because the flow velocities are smaller at the upstream side of the breach, it is easier to place a bottom protection. After the bottom protection the emergency dike can be constructed of i.e. containers or granular material like rocks.

Logistics and organization

The in situ filling of the containers is not possible due to the large currents. Therefore the containers need to be filled before placement. The weight of the containers is too much to be carried by helicopter (Gerven, 2004). The other option is to construct the dam with rocks. However, the placement capacity is not sufficient. The first layer can be done with a wide stone dumping vessel. However, the upper layers must be placed by helicopters or cranes on pontoons. This equipment does not have sufficient capacity to close a breach (Gerven, 2004). Another interesting option to construct the dam is Big Bags. Helicopters can construct such a dike.

Experience

In the past river breaches were repaired in this way. Along the large Dutch rivers the shapes of these dikes can be recognized (Voorde, 2004). These emergency dikes were however constructed after quite some time, so there was no real case of an emergency closure. Also, the closure of a breach near Schelphoek, the Netherlands, 1953 was closed in this way (Rijkswaterstaat & KNMI, 1961). This was a very large breach in a sea defence.

Potential for an emergency dike as emergency measure

There is potential for an emergency dike as measure during an emergency. The *technical* aspects can be covered. There might be problems in the *logistical* and *organizational* aspects. This emergency measure could be efficient to be implemented.

PLUG

Technical

PLUG stands for Portable Lightweight Ubiquitous Gasket. The PLUG is developed by the Department of Homeland Security of the United States. It is, according to Resio & Boc (2011): "a tube made of high strength fabrics designed to be partially filled with water and then floated into a levee breach, which they plug and thus stop or greatly reduce water flow through the breach." The PLUG is designed for the closure of dike breaches in the first four to six hours after the breach. The system can be applied in relative deep and narrow breaches. Question is if after four to six hours a deep and narrow breach has been formed, or that the breach is already widened.

Figure 3.21: PLUG in a physical scale test (Resio & Boc, 2011)

Logistics and organization

Helicopters or boats are able to bring the PLUG to the emergency location. For this operation a team of people is needed. The currents bring the PLUG to the breach and this will stop or reduce the water flow through the breach. Poor weather conditions can impede the implementation of the PLUG.

Experience

The PLUG is tested in the laboratory and in full scale. Both tests were successful. During the full scale test within 90 seconds after the implementation the breach was effectively closed (Resio & Boc, 2011). The breach was 12 meter wide and 2.4 meter deep with a discharge of almost 30 m^3/s. The PLUG was 30 meters wide. Notion must be made that the PLUG was used on concrete walls, so the effect of failing dike heads was excluded in the test. No real implementations of the PLUG in emergency measures are reported.

<u>Potential for PLUG as emergency measure</u>
The potential for a PLUG is high. During a full scale test it is proved that the PLUG is *technically* feasible. The question remains if the PLUG is still feasible if the dike heads are composed out of soil. The *logistical* and *organizational* aspects are covered too, since the PLUG is lightweight and applicable with limited resources.

3.4 Planned closures and closure techniques

In the past, closure operations were performed, for example the closure of tidal basins or reservoir dams. These closures can't be compared with the emergency closures performed during a dike breach, since the planned closures are prepared. A strategy can be determined with less time pressure beforehand. The logistical and organizational aspects are worked out in detail before the closure, so during the closure itself, the focus is on the technical part. Another difference is the fact that the future closed of area belongs at the moment of closure to the sea, so no damage occurs to the existing land if the closure operation fails. If a tidal closure is considered, a moment of tidal slack is often used for the moment of closure. Such a moment of small currents is not present in case of an emergency closure of a dike breach. This moment is reached when the water levels at the outer and inner side of the dike are equal. No explanation is needed that when that happens there is no case of an emergency closure anymore. Despite the differences, similarity can be seen between an emergency closure and a planned closure. In both, an emergency closure and the closure of a tidal basin, large current velocities are present in a gap which needs to be closed. The aim is to find lessons learned from planned closures which are applicable for emergency closures.

3.4.1 Strategies

For the planned closures, four main distinctions can be made regarding closure strategy. The different construction methods are displayed in Figure 3.22. After the application of bed protection, one could continue with a vertical closure, option A in the Figure. In a vertical closure, consecutive horizontal layers are used to close the gap. Option B is the horizontal closure. Hereby the gap is narrowed sideways. A combination of both before mentioned options holds first a vertical closure, creating a sill, followed by a horizontal closure. This can be seen in the Figure as option C. Instead of a horizontal closure after the vertical start, also box-type caissons can be used. This is the last option, D, in the Figure. These are all gradual closures; another option not displayed in the figure is a sudden closure. In this method the gap is blocked in one single operation by using a sluice caisson, vessel or pre-installed sliding- or flap gates.

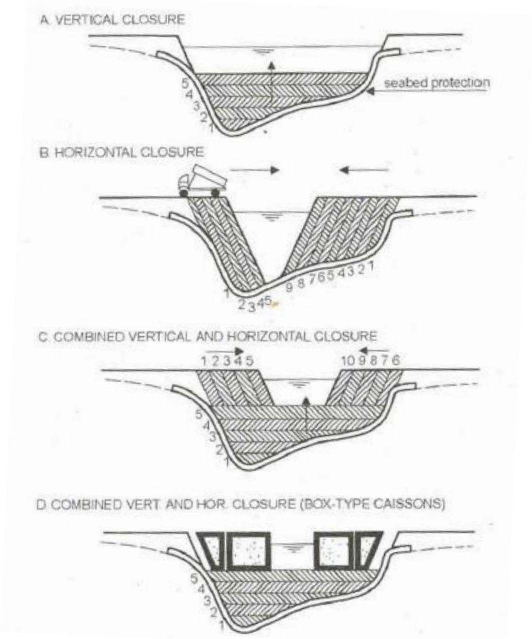

Figure 3.22: Basic methods of closure (Verhagen, et al., 2012)

The different closure methods require different equipment. Land-based, water-bone and in very special cases even air-borne equipment is used. Methods of closure may be based on the topography of the gap to be closed. Distinction can be made between tidal gullies, tidal flats, reservoir dam, tidal basins and river closures. The last one is a non-tidal closure. Planned closures can be performed with several materials like sand, clay, stones or caissons (Verhagen, et al., 2012).

3.4.2 Remarks

Closure operations with the horizontal closure method only are rarely applied in the Netherlands. If a horizontal closure method is performed, a sill is constructed along the gap first. If a gradual horizontal closure without, or with a too low sill, is performed, the area of the gap decreases and the current velocity will increase. This will make the closure of the last part of the gap extremely difficult. Heavy equipment will be needed even as a bottom protection that can resist these currents.

Sudden closures are often performed with sluice caissons; see Chapter 3.3.2 for caissons as emergency measures. Closures with caissons have a risk of failure due to piping. Lessons from occurred failures were to better perform the sinking of caissons, dumping of stones and the surveying of the sill. During the closure of the breaches of the Flood of 1953, it was impossible to measure all the dimensions of the occurred breaches in a short time. For this situation a unity-caisson was constructed. This caisson had a fixed length, width and height. These could be built relatively quickly and be applied in every breach.

If the vertical closure method is performed, the height of the sill is increased gradually. The sill is heightened till a level where a perfect weir is formed. This means, that when the weir reaches this level the flow velocity is independent of the head difference over the sill and is only dependent on the height of the sill (Konter, et al., 1992).

The closure method is determined by the situation, the stage and development of the breach and the available equipment (Rijkswaterstaat & KNMI, 1961).

3.5 Conclusions

Breach development already starts with the characteristics of the dike. The influence of dike core, geometry, revetment and subsoil on the breach development is enormous. There are four typical Dutch dike cores summarized. The geometry is divided in river and lake dikes. Revetment can consist of grass, rock, placed blocks and asphalt. For the subsoil upper and lower river areas are distinguished. Primary and regional flood defences are designed for different exceedence frequencies. The probability of a breach is thus larger for regional flood defences. A flood wave has several appearances which cause different failure mechanisms.

The three most common failure mechanisms are overtopping, piping and macro instability of the inner slope. Overtopping starts with a single gully while piping and macro instability of the inner slope affects large soil lumps. In the end the water overtops the dike for every failure mechanism and the breaching process starts the same way. However, for the piping and macro instability of the inner slope, the dike base is weakened already and the breach development is faster.

Breach development itself is categorized in five stages. In stage I the water overtops the dike and at the end of the last stage the breach is in equilibrium. Non-cohesive and cohesive dikes both have the same five stages. However, for non-cohesive dikes the breaching process goes faster. The breach development is dependent on all before mentioned aspects. There are three main types of breaches, dependent on the resistance against erosion.

Aspects of the preventive measures that could be advantageous for the implementation of curative measures should not be searched for in the used materials. For the preventive measures, granular material or composite structures are of little to no use to counter a breach. Only sandbags could perhaps function in a role as additional plug-ups for the phase after the implementation of the curative emergency measure. The real gain can be found in the transport modes and routes to weak places at the dike that can be established and prepared. Also, equipment like trucks are already available to drive to the emergency location and a team of dike watches is present.

As curative measures with the highest potential Big Bags, a scaffold, the PLUG and an emergency dike can be mentioned. They have technically, logistically and organizationally favourable aspects. A ship as an emergency measure is effective in very specific situations. The emergency measure with the least potential is a caisson since this measure needs a lot of preparation time.

By vertical planned closures the height of the sill is increased gradually. The sill is heightened till a level where a perfect weir is formed. The flow velocity will first increase and later decrease again. The flow velocity is than only dependent on the height of the sill. The closure method is determined by the situation, the stage and development of the breach and the available equipment (Rijkswaterstaat & KNMI, 1961).

4. Case studies

4.1 Introduction

Because of the low exceedence frequencies of the Dutch water defences and high maintenance standards in the Netherlands, (fortunately) no large dike breaches have occurred recently. If we go somewhat further back in time, in the Netherlands a large flood occurred in 1953. Despite this happened more than 60 years ago, quite some facts were documented. For other, more recent floods it is necessary to look abroad. After all, hindsight analysis of occurred breaches can provide valuable lessons and insight in important mechanisms during a dike breach. This information is useful regarding the implementation of an emergency measure. In this chapter cases are examined with the goal to get a feeling for the weight of the different aspects involved in an emergency closure. In the framework of technique, logistics and organization, the main direction for the critical aspects is formed. Elements or areas where progress can be made for better closure measures have been selected. These elements or areas will be stated in the conclusions and will be filled in in more detail in the next chapters.

As Visser (1998) already noted, data of dike failures are very limited. Collecting data does not have priority when a dike fails. Logically, the lives of people (and animals) will be taken care of in the first place. However, the importance of collecting data was not emphasized until recently. In, say, the last decade, information about dike breaches and the implemented preventive and curative emergency measures are documented in some way.

In Chapter 4.2 occurred breaches and the implementation of emergency measures in the Netherlands, China, United States and Germany are analysed in detail. For breaches and the implementation of emergency measures at several other locations, an overview with the most important characteristics is given. Conclusions and lessons learned are summarized in Chapter 4.3.

4.2 Analysis occurred breaches

For the analysis of occurred dike breaches, four events are selected. The selection is mostly based on the availability of data. However, there is aimed for different locations around the world. By this approach various techniques and situations are captured. The implementation of the emergency measures has various results. There is aimed for the collection of important aspects regarding the successful implementation of these emergency measures. Why were they successful or, why were they not successful? It is chosen to investigate breaches and the implementation of emergency measures in:

- Nieuwerkerk aan den IJssel, the Netherlands, Flood of 1953;
- Jiujiang, China, Flood of 1998;
- New Orleans, United States, Hurricane Katrina 2005;
- Fischbeck, Germany, Flood of 2013.

These cases are elaborated below, in chronological order. In Chapter 4.2.5 characteristics and lessons learned from other breaches are summarized. Breaches and the implementation of emergency measures in the countries Canada, Hungary, France, Japan, Thailand, Germany and the Netherlands are summed up in a table. Despite the fact that the information regarding the cases is derived from reliable articles, some parts are based on eyewitness reports from people without a hydraulic engineering background. Carefulness and a critical attitude regarding these parts of the information are required.

4.2.1 Nieuwerkerk aan den IJssel, The Netherlands, Flood of 1953

Location

Nieuwerkerk aan den IJssel is a town in the province of South Holland, the Netherlands. It is located, as the name suggests, on the IJssel River. This river is a fresh water river with tidal influence. The breach was situated about 30 kilometres landward from the mouth. Currently, there are several hydraulic structures present to prevent large floods to penetrate into the river. Such structures were not present back in 1953 and the tide could flow via the rivers deeply into the hinterland, as happened on February 1st 1953. Nieuwerkerk aan den IJssel is situated in a polder. This low-lying area is and was densely populated, with in that time about 3 million inhabitants (Gerritsen, 2005). Large cities like Rotterdam, Delft, The Hague and Leiden are in the same low lying polder, see Figure 4.1. Nieuwerkerk aan den IJssel is the lowest part of this polder and besides that, of all of the Netherlands, with a level of 6.76 m below NAP.

Figure 4.1: Inundated area if the dike breached[3]

Conditions

In the night of January 31 – February 1st 1953, a combination of spring tide and a high set up due to a severe north-westerly storm, caused water levels on the North Sea to reach a level of +3.85 m NAP at Hoek van Holland, this corresponds with an exceedence frequency of 1/250 years (Rijkswaterstaat & KNMI, 1961). Many water defences were overtopped and this caused extensive flooding. In the Netherlands only, 1,836 fatalities were reported.

The following conditions are specific for the IJssel dike in Nieuwerkerk aan den IJssel. At 0:00 hour the water level reached +2.60 m NAP. This was at low water, however this was already higher than the highest known water level at that time. The north-westerly storm increased to hurricane strength. Continuous rain, sleet and hail saturated the dike body. When the water level reached +3.00 m NAP, an evacuation was prepared. The water level kept rising and sandbags were placed as much as available on the dike. Next to the dike watchers, about 100 people from the army assisted with the placing of sandbags. Water started to overtop the crest and flowed over grass cover of the 'Schielandse Hoge Zeedijk, the Groendijk'. The inner slope eroded away and the dike had a width of only 1 meter left at some places[3]. At 4:00 o'clock a water level of +3.84 m NAP was reached.

[3] *Information on monument 'Een dubbeltje op zijn kant' (property of Hoogheemraadschap Schieland en Krimpenerwaard), IJsseldike, Nieuwerkerk aan den IJssel, the Netherlands*

At 5:30 a.m., a breach of 15 meter wide was formed. Mayor of Nieuwerkerk aan den IJssel, J.C. Vogelaar, requisitioned the ship 'Twee Gebroeders' of A. Evegroen. This ship just happened to be around. The skipper placed (against his own will) the prow of his ship in the dike next to the breach. After this action, the currents swept the stern across the breach into the dike body on the other side of the breach, like the gate of a lock. Remaining gaps were closed by sandbags and additional material. The dike at the other side of the IJssel River breached near Ouderkerk aan den IJssel at 6:30 a.m. This caused a drop in the water level of about 20 cm. About the same moment low water started, which both had favourable effects on the closure of the breach in Nieuwerkerk aan den IJssel (Boer, 2007), (Rijkswaterstaat & KNMI, 1961).

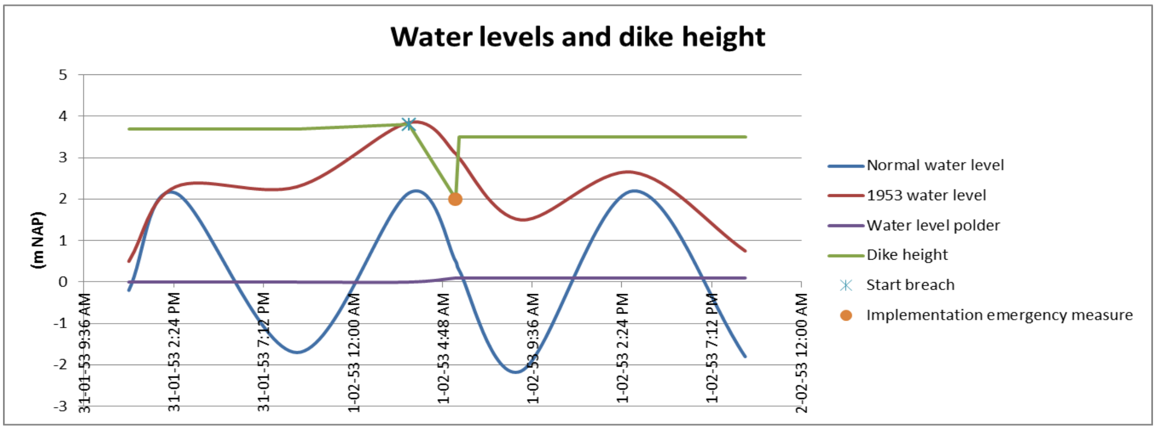

Figure 4.2: Water levels and dike height Nieuwerkerk aan den IJssel

In Figure 4.2, a sketch of the normal water level, the water level of 1953 and the water level in the polder are displayed, together with the dike height and the start and end of the breach. As can be seen the breach started almost at the highest water level. After implementation, the vessel was able to withstand the second high water wave. The water level was already dropping, however the emergency measure was very important since the normal high water level was already above the breach bottom, what would result in continuation of the breach development. See Appendix II for the composition of the figure.

Characteristics dike

The geometry of the dike is presented in Figure 4.3. This is the cross section that is made at the location of the emergency closure. The dashed line represents the 'old' profile, the dotted, line shows the situation caused by the flood of 1953. The solid lines show the dike reinforcements. The crest level of the 'old' dike was +3.70 m NAP (Rijkswaterstaat, 2012b). The inner and outer slopes were 1:2 and an outer berm was present around +1.5 m NAP. A toe construction was present at the outer slope around NAP level. The subsoil consisted of a sand body with a clay cover.

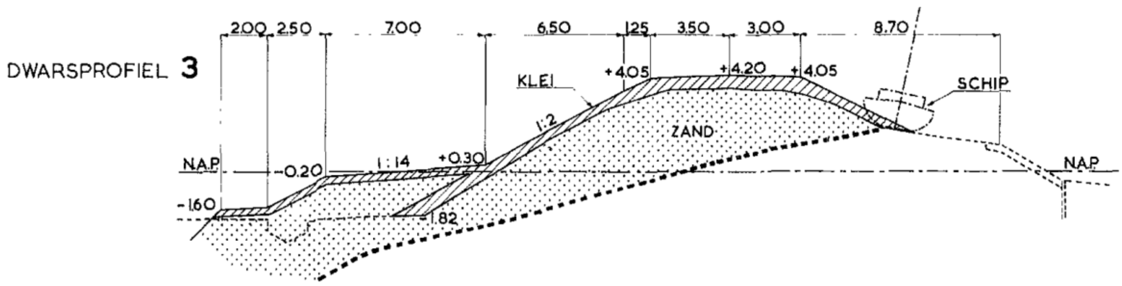

Figure 4.3: Cross section of the 'old' and reinforced IJssel dike (Rijkswaterstaat & KNMI, 1961)

Characteristics breach
At 3:00 a.m. a breach was started which developed to a length of 6.75 m and a depth of 2.60 m below the crest of the dike at 4:30 a.m. This breach grew quickly to a width of 15 m (Boer, 2007). At 5:30 a.m. a vessel of 120 tons was implemented on the outer slope. The force of the inflowing water was slowed down (Rijkswaterstaat & KNMI, 1961). On the outer slope a toe protection was present. The breach development was limited by the existing toe protection on the outer slope. The berm could resist the erosion and later proved to be a foundation for the emergency measure. Behind the breach a scour hole with a depth of 6 m below ground level was formed (Boer, 2007).

For this clay dike the breach development of Zhu (2006) is considered. The breach had 1.5 hour to develop before the emergency measure was implemented. In this 1.5 hour the breach did grow in the horizontal direction. The breach growth in the vertical direction was hampered by the toe protection. This makes the breach a type A breach (Visser, 1998). Chapter 3.2.2 described the breach processes in more detail. Figure 4.4 shows a cross section of a type A breach.

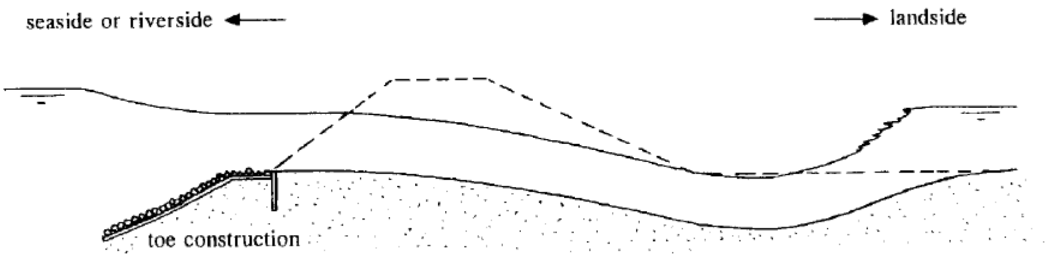

Figure 4.4: Breach stage IV, type A (Visser, 1998)

The breach developed to a stage IV type. Stage V was not reached since the water in the polder did not have influence on the velocity of the water flowing through the breach. If the backwater in the polder decelerates the flow stage V is reached.

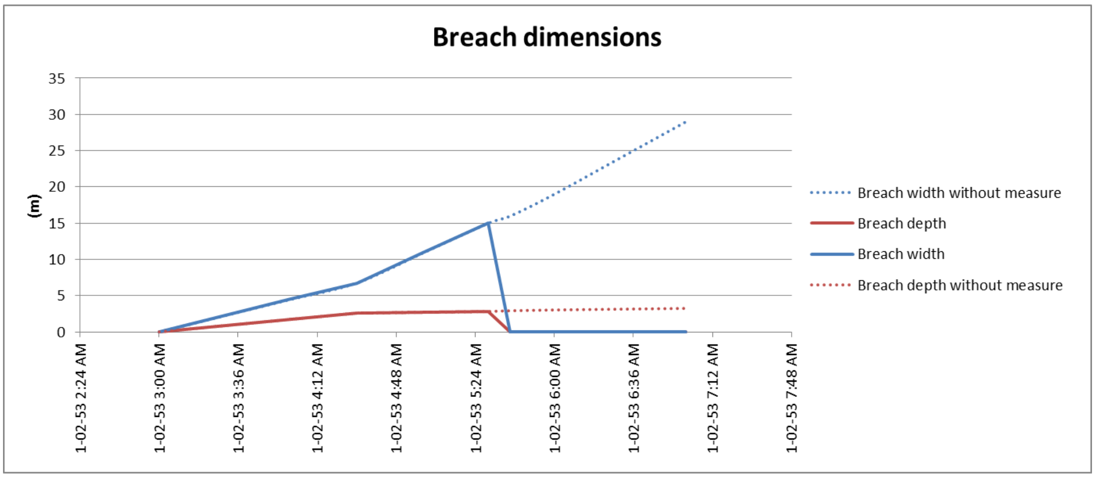

Figure 4.5: Development of breach dimensions Nieuwerkerk aan den IJssel

The development of the breach dimensions are displayed in Figure 4.5. Due to the toe construction the depth of the breach stays limited. The width of the breach develops rather quickly. If the emergency measure was not implemented the width would have doubled in somewhat more than an hour.

The breach dimensions without emergency measure displayed in Figure 4.5 are determined by the non-interfered development of the breach, determined by Formulas (3-9) and (3-11) in the mathematical model in Chapter 3.2.2. For the calculation see Appendix II.

Emergency measure

As mentioned above, the emergency measure used was a vessel. This vessel named "Twee Gebroeders" was first used to transport sand and gravel. It was built as an "IJsselaak" type ship. In 1925 the vessel was lengthened with 3 meter and had a total length of 18 m. Since that moment the vessel was motor driven (Boer, 2007).

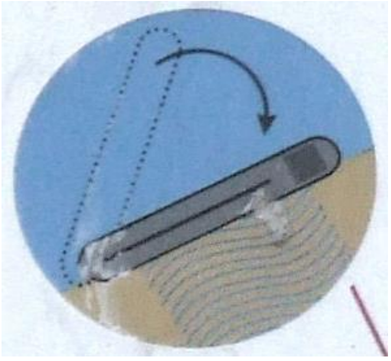

Figure 4.6: Closure method[3]

When the breach was 14 meter, the prow of the ship was placed in the dike next to the breach. After this action, the currents swept the stern across the breach into the dike body on the other side of the breach, like the gate of a lock. The closure procedure is displayed in Figure 4.6. Neither any erosion protection nor any re-profiling of the gap to fit the vessel's shape was performed. Piping was thus a serious threat. Once the ship was in the position where it closed off the breach, it was sunk down and filled up with sand. With sandbags and tarpaulins the remaining gaps between the outer slope and the vessel were closed (Verhagen, et al., 2012). The level of the sandbags behind the ship in the breach was heightened till crest height with sandbags before the next high water entered. To reach the location of the breach, a path along an unpaved dike, which was severely damaged by slipped inner slopes, impeded the equipment and material to reach the breach. About 50,000 sandbags were needed to finally close the breach (Rijkswaterstaat & KNMI, 1961). The scour hole formed at the inner side of the breach was closed with sand.

The breach was developing fast and due the lack of time, quick handling was the only way to gain success. If the aspects logistics and organisation are checked, a few notes can be made. A lot of manpower was available. Inhabitants, the police and probably the most important, the army were present. This was no simple task, seen the time at night. It remains unclear who was in charge, however the role and leadership of mayor Vogelaar was crucial. The presence of the ship was a coincidence. However the sandbags were specifically asked for by mayor Vogelaar at the Water Board of Schieland (Boer, 2007). To close the breach by means of a ship was improvisation, the presence of the sandbags were prepared.

Figure 4.7: Ship the 'Twee Gebroeders' after emergency closure, source: Nationaal Archief/Spaarnestad Photo/ANP ©

Effect

The emergency measure was successful and the breach was closed. If the breach was not closed, probably over 30,000 people would have drowned (Rijkswaterstaat, 2012b). The fact that the ship was larger than the breach and the presence of a berm with toe protection caused an almost immediate closure. Despite the high amount of improvisation, the desired result was obtained.

The performed emergency closure was a risky one. If the closure operation failed, the result could have been a much more severe situation. A possible failure could have been caused by the insufficient strength of the dike head to carry the horizontal load of the ship caused by the force of the water. The dike head was weakened by the saturation of rain, sleet, hail and the hammering water on it. Since the ship was 18 meter long and the breach 15 meter, on both dike heads the pressure of the ship is spread over 1.5 meter contact surface. If the dike head was pushed away, a larger breach was created and the closure would have been even more difficult.

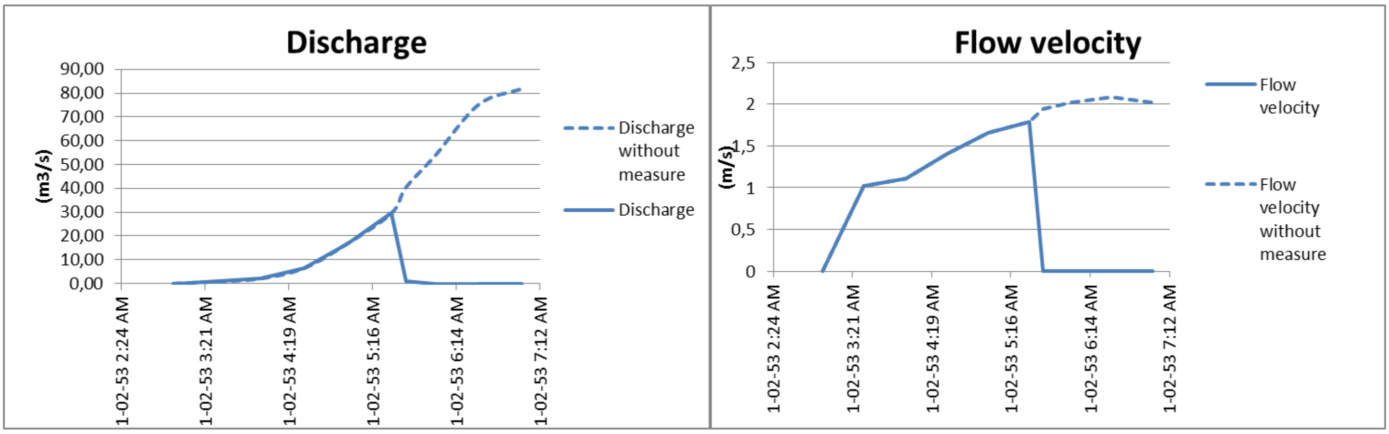

Figure 4.8: Development of discharge and flow velocity Nieuwerkerk aan den IJssel

Figure 4.8 displays the development of the discharge and flow velocity through the breach. Calculations can be found in Appendix II and are done with formulas (3-7) and (3-22) from Zhu (2006). Again the effectiveness of the measure can be seen by the increase in discharge if the measure would not have been applied. The flow velocity is an averaged one. Extreme velocities are larger, however, for the calculation of the forces on the ships the average velocity is a better representation.

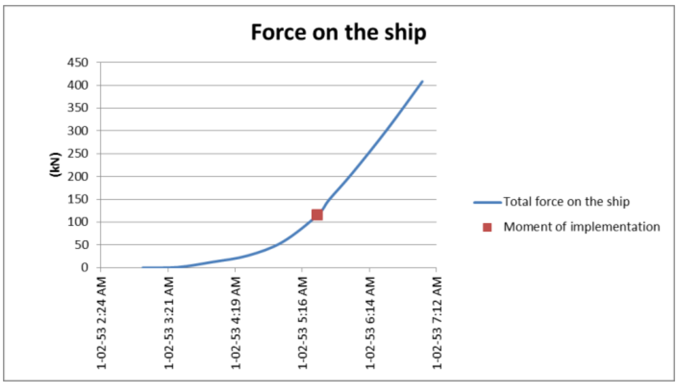

Figure 4.9: Force on the ship

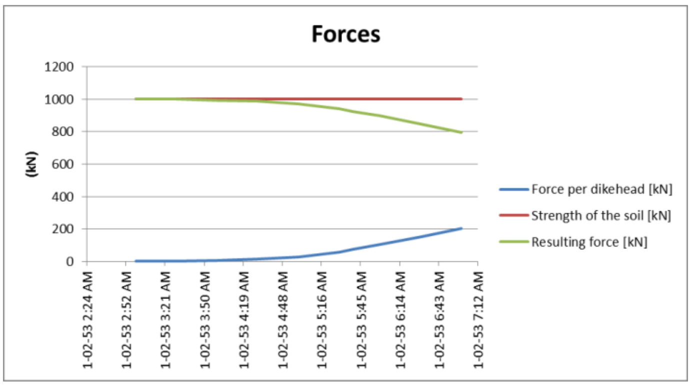

Figure 4.10: Forces per dike head

The force that acts on the ship during the closure is displayed in Figure 4.9. The dynamic and static water pressure are taken into account. See Appendix II for the calculations. The line in the figure continues in time after the moment of implementation. There is for this breach area assumed that there was a ship available that had the right dimensions to close the enlarged breach at every moment.

The forces on the ship are equally divided per dike head and displayed in Figure 4.10. There is assumed that the shear strength of the soil is constant, although this is questionable due to the saturation of the dike body. There is calculated that there was no danger of the water pushing the ship through the dike heads. Calculations are again in Appendix II.

Another failure mode could have been piping. Since the hydraulic head is spread over a short horizontal distance, the gradient of the water over the ship increased. This could have induced piping under the vessel and around the stern which could easily have eroded another gap under the ship. Then, the vessel could have been pushed away through the newly eroded gap, leaving an even larger gap (Verhagen, et al., 2012).

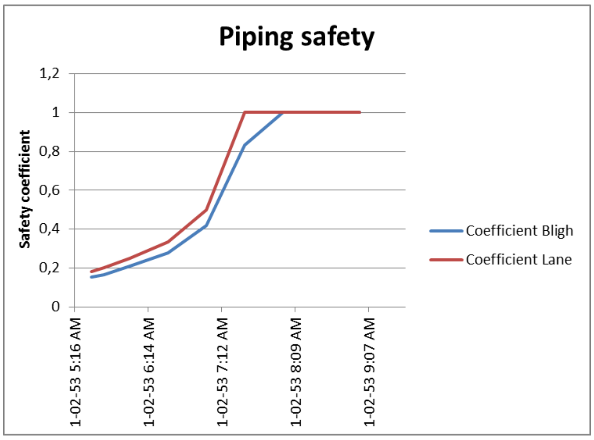

Figure 4.11: Piping safety

The danger for piping was especially present in the first phase after the implementation. This can be seen by the low safety coefficient in Figure 4.11. In this Figure a safety coefficient is calculated by dividing the actual piping length by the needed length for safety. See Appendix II for the calculations. However, the water dropped and the vessel eventually rose above the water level (see Figure 4.11), resulting in a safe situation. This can be seen in the graph, the safety coefficient is increasing since the water level dropped, resulting in a decrease in hydraulic head. During the danger of piping a dike of sandbags was constructed and other measures to fill up the gaps between the ship and the dike were taken. These are not taken into account in the graph. Piping channels should have formed according to the graph. This was not the case because of the short duration of the piping hazard and the additional measures to stop piping like sandbags and tarpaulin.

Conclusions and lessons learned

As first conclusion, it can be said that the closure was a lucky shot. There was almost no preparation or technical knowledge about what to do. In this case it worked out well, since a large part of the province of South Holland was kept dry.

The implementation of a ship as an emergency measure was effective due to the still present berm and that the dimensions of the ship were larger than the length and depth of the breach. Next to these aspects there was enough manpower and material present to close the remaining gaps. Due to the declining water level the danger of piping was decreased.

It is the question if this approach could be applied to other cases. The risk of failure of the method is high. The situation could have been worse if ship was pushed through the breach or if piping created a new breach. In this case piping did not occur because the emergency measure was immediately strengthened with a dike of sandbags.

The closure was *successful*, coupled to the framework, the technical aspect was fulfilled by the *ship which stuck on the berm*. Logistics were covered by the sufficient *availability of material and equipment*. The presence of enough manpower and especially the interference of the *army* made the organization a success.

4.2.2 Jiujiang, China, Flood of 1998

Location

The city of Jiujiang is situated in the north of the Jiangxi Province and lies on the southern bank of the Yangtze River. In 2010 the city of Jiujiang had almost 4.8 million inhabitants. Jiujiang is active in manufacturing of car, machinery, petrochemical, shipbuilding and textiles. The city is connected to other cities by a railway, a highway and via shipping on the Yangtze River. The Yangtze River is intensely used for shipping. The Yangtze River is the longest river in Asia and has a length of about 6,300 kilometres (Li, et al., 2003). The city of Jiujiang is about 500 kilometres from the mouth of the Yangtze River. The City Defence Dike, which protects the city of Jiujiang against the water of the Yangtze River, has a total length of 17.2 km of which 11.3 km of flood wall. The design height is 25.25 m, with 1.5-4 m soil filling behind the wall, whose height is 24 m. The river bed has a height of 20 m (Anonymous, 2008). The heights are above the Chinese reference level.

Figure 4.12: Location of Jiujiang, China (Yang, et al., 1998)

Conditions

China suffered from heavy rain in the months July and August 1998. The rainfall was the one of the most heavily recorded in China's history. This extensive rainfall caused extreme high water levels in the Yangtze River (Chen & Li, 2000). In the summer of 1998 a 3 month long battle against the high water levels is fought. The flood has impact in all of China and nationwide 5 million homes were demolished, 21 million hectare of farmland is devastated and a total of $19 billion damage is caused by the flood. The floods are the worst in at least 100 years.

Next to the extreme rainfall, the changing environment is suspected to have an impact on the scale of the flood. The exploding population occupies places which originally were flood storage places. Lakes that used to be storage basins in the past are drained and made into living area for new inhabitants. In 50 years half of the forest in the Yangtze River basin had disappeared. The Yangtze River is at a lot of places dammed and rerouted.

On August 7, 1998 1:50 p.m. a sudden breach on the river embankment just 4 km away from downtown Jiujiang caused flood water to pour into the city. The water level at the moment of the dike breach was 22.90 m above Chinese reference level. The breach was initiated by piping. The floodwater kept widening the breach. During the failure of the dam, it turned out that the dam was not built according to official specifications. No reinforcement was used during the construction of the flood wall. At 5:00 p.m. an 80 meter long freighter loaded with coal was planned to be sunk down. The large currents made it impossible to get the vessel close to the breach. With the help of tows, the vessel was held more under control and it was possible to bring it closer to the breach and sunk it down there. At August 8, 1:20 a.m. eight other ships were sunk in front of the breach. The aim was to slow the water down to give the flood fighters a chance to close the breach. At August 9, 11:30 a.m. more than 4,000 people were helping to close the breach, of which 220 of the People Liberation Army. The breach was finally closed on August 12, 6:00 p.m. After five days and nights of struggle, a 105 meter long dam was constructed of iron, wood, clay and rock (Yang, et al., 1998).

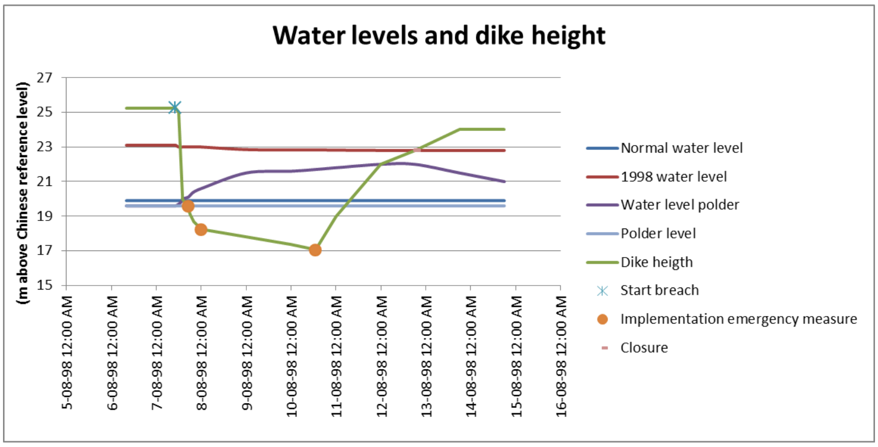

Figure 4.13: Water levels and dike height Jiujiang

In Figure 4.13, a sketch of the normal water level, the water level of 1998 and the water level in the polder are displayed, together with the dike height en the start and closure of the breach. The dike failed due to piping when the outer water level was below the crest. The piping process happened quickly, which explains the steep drop in dike height. The first yellow dot represents a vessel sunk down in front of the breach, this reduced the discharge. At the time of the second yellow dot, a team of experts arrived and started working on closing the breach by means of a scaffold. At the time of the third yellow dot, the scaffold was connected and the breach was closed with a vertical closure. The water level in the polder already rose to almost the outer level. However, the closure was important since the water was still flowing into the polder.

Characteristics dike

The dike was in this case a flood wall called the City Defence Dike. A picture of the cross section is visible in Figure 4.14. The geometry of the flood wall has a suspicious role in the occurrence of the breach. The City Defence dike was located in a city, and consisted of an embankment with a vertical wall at the riverside measuring approximately 3.5 m in height.

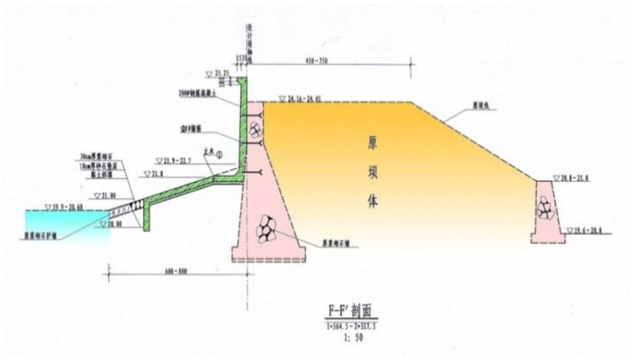

Figure 4.14: Cross section of the City Defence Dike (Anonymous, 2008)

The top level of this wall was between 0.8 m and 1.1 m higher than the crest level of the dike at the land side. The subsoil of the dike consists of clay with parts of gravel and silty loam (Deltares, 2009).

Characteristics breach

The breach in the City Defence Dike was initiated by piping. An 'early' stage of the piping phenomenon can be seen in Figure 4.15. A lot is said about the missing reinforcement in the flood wall. However, the flood wall seems not well designed for piping failure. A large hydraulic head due to the high water level in the river in combination with the geometry of the water defence and the lack of a seepage shut off screen started the piping mechanism.

Figure 4.15: Early stage of piping at the City Defence Dike (Rage of the River Gods, 2001)

The cause of dike failure was piping with subsequent sliding of the inner slope, resulting in a breach. Time dependent data of water levels in the Yangtze River are not available (Deltares, 2009). In Figure 4.16 the breach starts to develop. The soil beneath the flood wall is washed away, however the flood wall stayed in place for a while. People are desperately throwing in sandbags, which have no effect on the large currents in the gap.

Figure 4.16: Development of the breach under the flood wall (Rage of the River Gods, 2001)

The soil is continued to be washed away, resulting in a decreasing support of the concrete floodwall. In Figure 4.17 the flood wall is collapsed and the water has free entrance to flow into the lower laying city of Jiujiang. The breach keeps developing and a decision is made to sink down an 80 m long freighter in front of the breach. After this intervention, two sluice gates broke through leaving a gap of 64 meters wide. A scour pit with a maximum depth of 7 meter is formed. This is the result of a water head of 5 meter (Deltares, 2009).

Figure 4.17: Collapse of the City Defence Dike (Rage of the River Gods, 2001)

Figure 4.18 displays the development of the breach dimensions in Jiujiang. The breach develops quickly and has almost reached an equilibrium state. During closure, the depth of the breach is decreased gradually while the horizontal closure happens rather quickly. This is because of the closure method with a scaffold (see Chapter 3.3.2 for explanation of a scaffold closure). The breach dimensions without emergency measure are determined by the non-interfered development of the breach, determined by Formulas (3-9) and (3-11) in the mathematical model in Chapter 3.2.2 from Zhu (2006). For the calculation, see Appendix II.

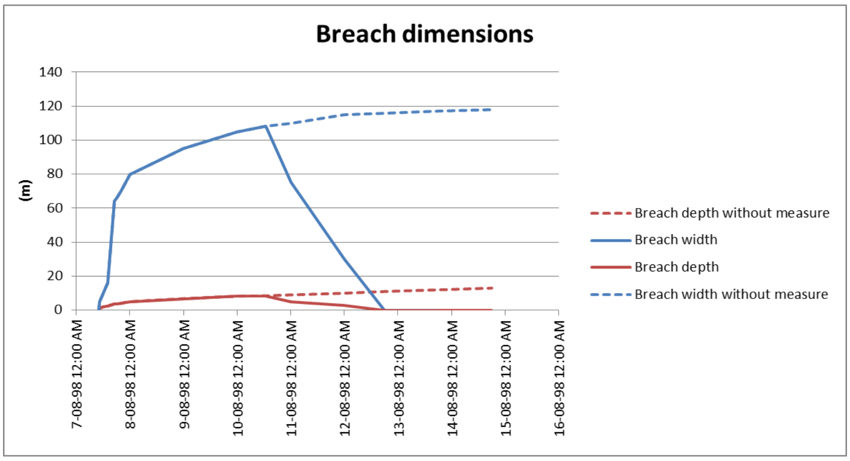

Figure 4.18: Development of breach dimensions Jiujiang

Emergency measure

When the breach was at an initial stage, the local authorities tried to close the breach by all means possible. They tried to avoid a disaster. Due to panic, the decision was made to throw whatever large object they could find in the breach. Precious assets like trucks and ships were sunk down into the gap. The current however was too strong and washed away the objects almost immediately. In Figure 4.19 can be seen how a vessel is pushed through the breach by the enormous currents, having absolutely no chance on closing the breach. This shows that there was no engineering view, just panic.

Figure 4.19: Failed emergency closure by means of a vessel (Rage of the River Gods, 2001)

Since the earlier attempts failed, larger measures were deployed. Figure 4.20 shows a 75 m large freighter in front of the breach. This could have been a step in the right direction to close the breach, however the dam was constructed in such a poor way, that it collapsed even more after the ship was sunk down. After this event the government saw the urge to take action.

A team of about 200 technics was send to the gap by nightfall on August 7, 1998. They came up with an emergency closure plan as is showed in Figure 4.21. The idea behind this plan is that the barges slow down the currents enough for the construction workers to build a scaffolding of wooden and steel piles at the exact location of the old flood wall. This scaffolding is constructed from both dike heads on and is anchored at the large sunk down freigther. In total 8 vessels were sunk down successfully. After the completion of the scaffolding, elements are tossed into the water in front of the scaffolding. The scaffolding prevents the elements to be washed away and the closure can be completed. The water-head at the upstream of the breach-blocking dyke is 3.1 m higher that that at the downstream (Anonymous, 2008).

Figure 4.22 shows the construction of the scaffold. A lot of people were working on the closure, sometimes resulting in a chaos. In the end, the closure was successful. Figure 4.23 shows the final closure.

Figure 4.20: Sinking down a freighter filled with coal (Rage of the River Gods, 2001)

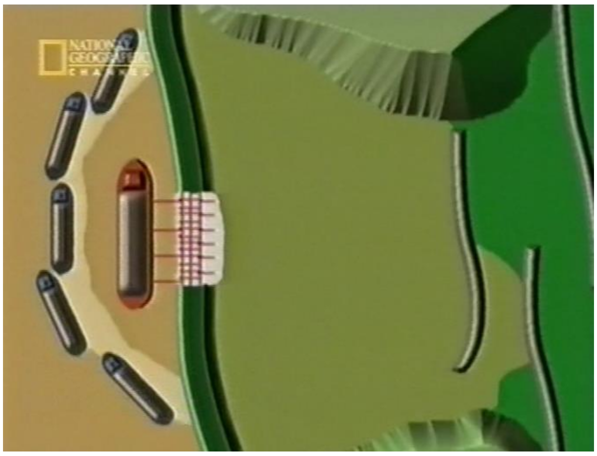

Figure 4.21: Emergency closure plan (Rage of the River Gods, 2001)

Figure 4.22: Construction of scaffolding to close the breach (Rage of the River Gods, 2001)

Figure 4.23: Final closure (Rage of the River Gods, 2001)

The cross section of the final closure can be seen in Figure 4.24. During the closure the available amount of materials and equipment was large. The logistical and organisational aspects are covered quite well. 263 ships, 200 vehicles and 25 large construction mechanical equipments are put into operation. 24,000 soldiers of the Liberation Army and the Armed Police Force are mobilized. There are also 5000 people participating in dealing directly with the flood emergency on the site of the breach, including the people for transportation, food charging and loading. In the peak time, the number of the personnel dealing with the flood emergency reaches 10,000 (Anonymous, 2008).

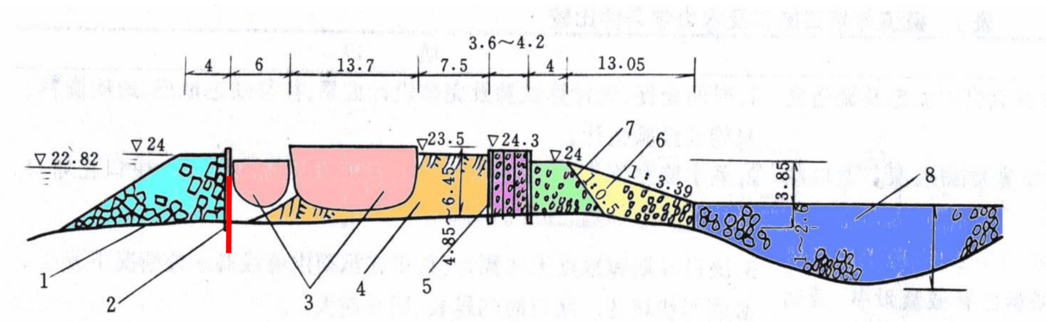

Figure 4.24: Cross section of the closure (Anonymous, 2008)

The numbers in Figure 4.24 are explained below.
1) Outer dike, 2) Stone retaining scaffolding, 3) Sunken boat, 4) Underwater clay blanket, 5) Steel and wood scaffolding, 6) Stone bags, 7) Temporary section line, 8) Filled scour hole and foundation reinforcement.

Effect
The early attempts to close the breach by throwing into the breach what was available failed miserably. Trucks and vessels were deployed to try to close the breach. In this way, technically seen a horizontal closure is tried (definitions of horizontal and vertical closures can be found in Chapter 3.3). The gap is narrowed and the flow velocity increases. Without bottom protection the element is pushed away by the currents or the flow will scour away the soil around the element. This results in an unstable element which will also be pushed away.

Decreasing the discharge through the gap is the main target of the vessels in front of the breach. Due to the effect of these vessels on the flow pattern, the discharge decreased from, approximately, 400 m^3/s to approximately 300 m^3/s (Anonymous, 2008). The vessels did have effect. However it remains the question if the benefit of less discharge outweighs the loss of eight vessels. Since without these vessels, the closure could probably also have been performed.

The decision to construct a scaffold is understandable. However, the location of the scaffolding is not that clever. At the location of the old flood wall, the currents are the strongest. To construct a scaffold at that location, one needs to deal with the largest forces and the deepest scour hole. Probably a better idea was to change the alignment of the dike. The reason why the scaffold was successful as a closure measure is because a vertical closure is performed in this way. During the heightening of the sill, the flow pattern will change from an imperfect weir to a perfect weir. During this transition the flow will become critical and after this stage the flow velocity will decrease (Battjes, 2002). This is favourable for a closure.

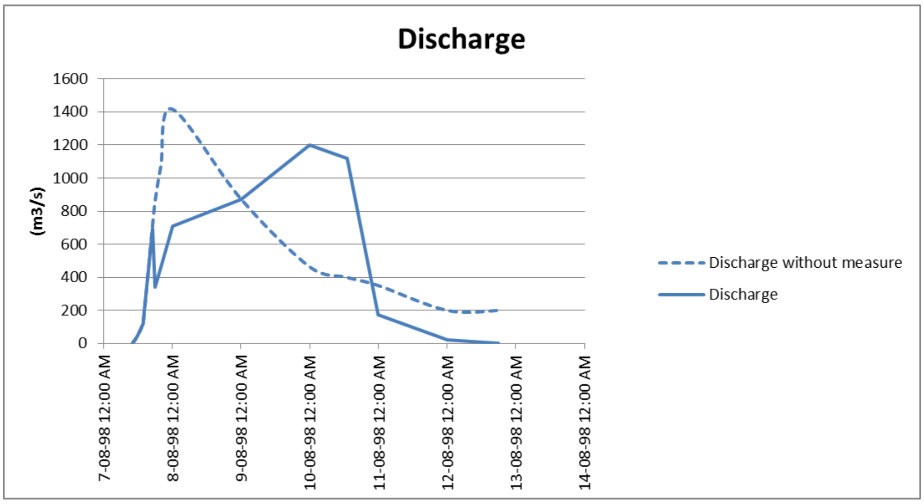

Figure 4.25: Discharge through the breach Jiujiang

Figure 4.25 displays the development of the discharge in the case with the emergency measures and without the emergency measures. The sharp drop in discharge is caused by the sinking down of a ship. However, the adjacent dike failed after the implementation explaining the increase in discharge right after. If this measure was not implemented the discharge would increase strongly. With the measures applied, the largest discharge is delayed. Calculations can be found in Appendix II and are done with formulas (3-7) and (3-21) from Zhu (2006). Again the effectiveness of the measure can be seen by the increase in discharge if the measure would not have been applied. The closure was still (somewhat) effective, since the discharge would continue if the breach was not closed.

Conclusions and learned lessons

As first conclusion can be stated that for every flood defence a good design is essential. Construction errors made the City Defence Dike to a poorly performed design. Every failure mechanism should be taken into account for the design.

In this case, the attempts which can be categorized as horizontal closures failed. Without the proper bed protection this closure method is unsuitable. Attempts with this method will probably fail.

Vertical closures are a better solution. A scaffold could support this closure method. The smaller elements are in this way not carried away by the currents. Vertical closures have the benefit of a decreasing current velocity at the transformation from an imperfect to a perfect weir. It must be mentioned that in this case the enormous manpower and available equipment was a big advantage to close the breach. Vessels in front of the breach are able to reduce the discharge.

The closure was *successful*, coupled to the scaffold, the technical aspect was fulfilled by the *vertical closure* supported by a scaffold of wood and steel. Logistics were covered by the sufficient *availability of material and equipment*. The presence of enough manpower and especially the interference of the *army* made the organization a success.

4.2.3 New Orleans, United States, Hurricane Katrina 2005

Location

New Orleans is situated in the southeast of Louisiana, United States, in the delta of the Mississippi River. Parts of New Orleans lay below mean sea level. The city covers an area of 468 km². Before hurricane Katrina the city had 500,000 inhabitants. The city is threatened by different possibilities of floods; Lake Pontchartrain on the north side, the Gulf of Mexico on the east and the Mississippi River itself. Since the establishment of the city of New Orleans, several floods hit the city, caused by both high river discharges and high water levels due to hurricanes (HKV lijn in water & Delft University of Technology, 2006). The analysis in this paragraph will be focussed on the failure of the 17th Street Drainage Canal's flood wall.

Figure 4.26: Location of New Orleans (HKV lijn in water & Delft University of Technology, 2006)

Conditions
Hurricane Katrina caused extreme high water levels in the Gulf of Mexico on August 29, 2005. Next, the water in Lake Pontchartrain rose. This lake has a direct connection with the drainage canals in the city centre of New Orleans, causing high water in the canals (HKV lijn in water & Delft University of Technology, 2006). Before Hurricane Katrina hit in August 2005, predictions about the risk of a hurricane and the concerns about the flood defence system, were spread. Breaches at approximately 20 locations were formed (Sattar, et al., 2008). The estimated damage in New Orleans only is at least $30 billion and about 1,100 fatalities were reported (HKV lijn in water & Delft University of Technology, 2006). In the greater New Orleans region 1,503 fatalities were reported (Seed, et al., 2008a).

The Central Region (Seed, et al., 2008b) and the East Bank (Seed, et al., 2008d) were flooded; however one of the most devastating breaches in New Orleans was the 17th Street Canal breach. Due to the surrounding topography, this breach had a huge impact on the surroundings. The 17th Street Canal had an I-shaped flood wall as flood defence with a height of about 12 ft above mean sea level. The earthen embankment had a height of about 6 ft above mean sea level. Storm surge levels in the 17th Street Canal were measured at 30 minute intervals during the storm. At the south end of the canal, the water level rose to 7.5 ft above mean sea level at about 5:00 a.m. Shortly after 9:00 a.m., it dropped rapidly back to less than 2 ft above mean sea level and never rose again to levels much higher than that. The breach occurred at about 9:00 a.m., producing the rapid and permanent lowering of storm surge water levels within the canal. The water level at the moment of the breach was about 8.5 ft above mean see level. Overtopping was therefore not the failure mechanism of the flood wall. Geotechnical failure caused the flood wall to fail (Seed, et al., 2008c).

The occurred breach had a total width of 137 m. The flood protection system in the 17th Street Canal was an I-section, concrete flood wall over a levee embankment of fill material built over a marsh layer. The storm surge moved the levee and flood wall horizontally for about 14 m and this breach accounted for much of the flooding of the city. The US Army Corps of Engineers responded immediately by placing Big Bags filled with sand in the breach with helicopters. These bags were completely washed away by the currents. Later the weight of the Big Bags was increased. It took several days to close the breach and the water level was equalled.

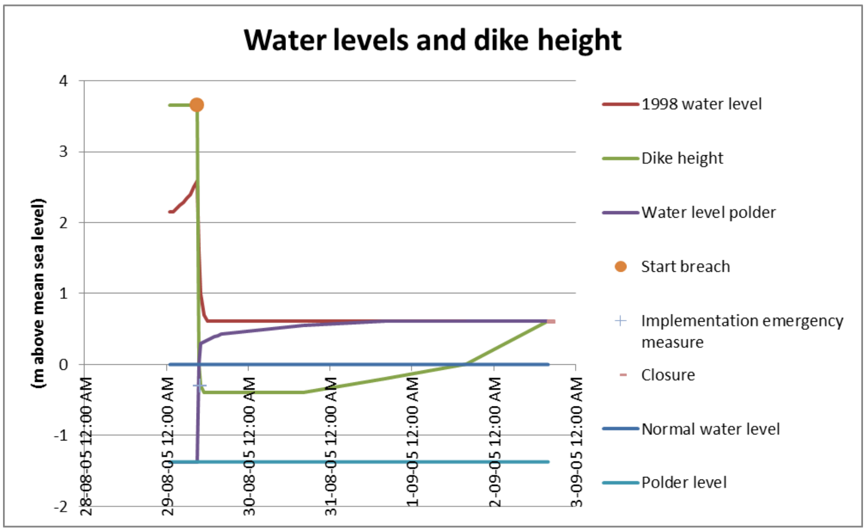

Figure 4.27: Water level and dike height New Orleans

In Figure 4.27, a sketch of the normal water level, the water level of 1953 and the water level in the polder are displayed, together with the dike height and the start and end of the breach. The dike failed due to piping when the outer water level was below the crest. The piping process took place quickly, which explains the steep drop in dike height. The yellow dot represents the start of the breach. At the time of the + sign, the USACE started placing Big Bags in the breach. This measure did not have effect since the Big Bags were washed away by the flow. In the end, the Big Bags were able to close the breach. The water level in the polder already rose to the outer level so the closure can be seen as unsuccessful.

Characteristics dike

The cross section of the flood defence including the subsoil of the 17th Street Canal is displayed in Figure 4.28. The flood defence consists of a concrete I-shaped flood wall on to of an earthen embankment and a sheet piling below the flood wall. The top of the floodwall has an elevation of 12 ft above mean sea level. The crest of the embankment has an elevation of about 6 ft above mean sea level. The hinterland, on the protected side, lies at a height of 4.5 ft below mean sea level. Since the water level reached a maximum level of 8.5 meter above mean sea level, the flood wall was high enough for this event to prevent overtopping. The subsoil of the flood defences had a wide spread erosion rate (Briaud, et al., 2008). The erosion rate of the soil has an enormous influence on the development of a breach in the time. During the design these different erosion rates were not taken into account.

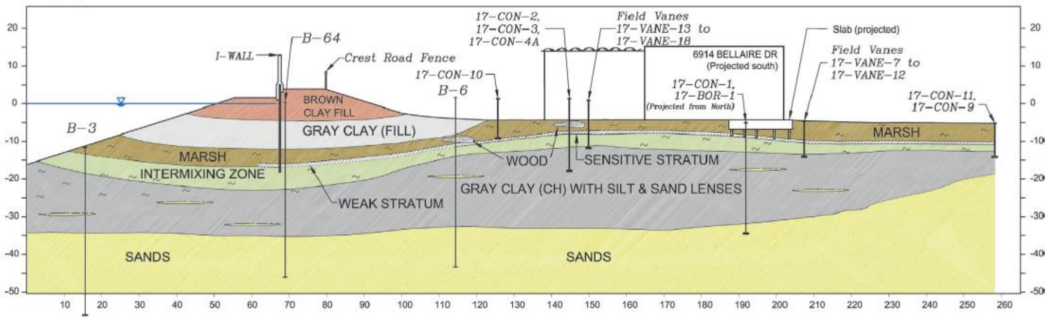

Figure 4.28: Cross section flood defence 17th Street Canal, before failure (Seed, et al., 2008c) measures in feet

Characteristics breach

Failure of the flood defence was caused by horizontal sliding of the inner slope. The layer over which the slope sheared was undetected during the design of the flood wall. This layer was a layer with deposits of a hurricane. It consists of organic silty clay. That layer became the critical shear surface for the lateral translational failure. Some other too optimistic assumptions and interpretations during the design had their effect on the failure. In Figure 4.29 the cross section of the dike after the failure is displayed. The shear plane is highlighted in black.

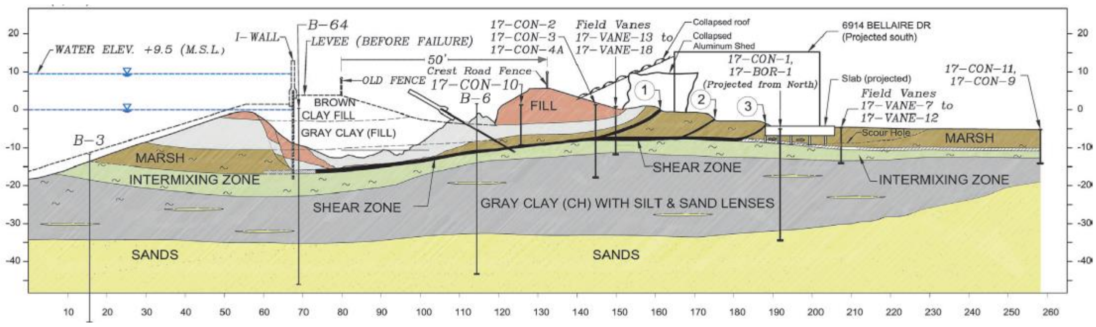

Figure 4.29: Cross section flood defence 17th Street Canal, after failure (Seed, et al., 2008c)

The development of the breach is determined according to eyewitness stories and a video made by the fire crew, directly after the start of the breach. The concrete flood defence wall moved laterally and opened a V-shaped gap between two concrete wall parts. This happened around 6:00 a.m. The main breach occurred around 9:00 a.m. and lowered the water levels in the canal permanently. The rotational resistance of the sheet piles was insufficient. The sheet piles were not deep enough put into the subsoil, creating insufficient passive resistance (Sasanakul, et al., 2008), (Sills, et al., 2008).

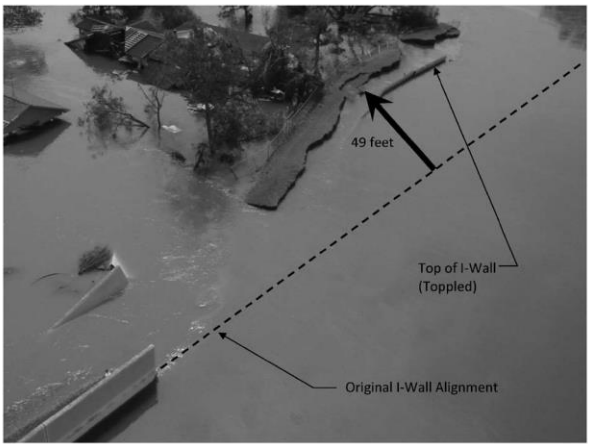

Figure 4.30: Top view of the breach (Seed, et al., 2008c)

The embankment and floodwall were pushed laterally about 14 m away of its original position. This can be seen in Figure 4.30. The total width of the breach was 137 m (Chaudhry, et al., 2010).

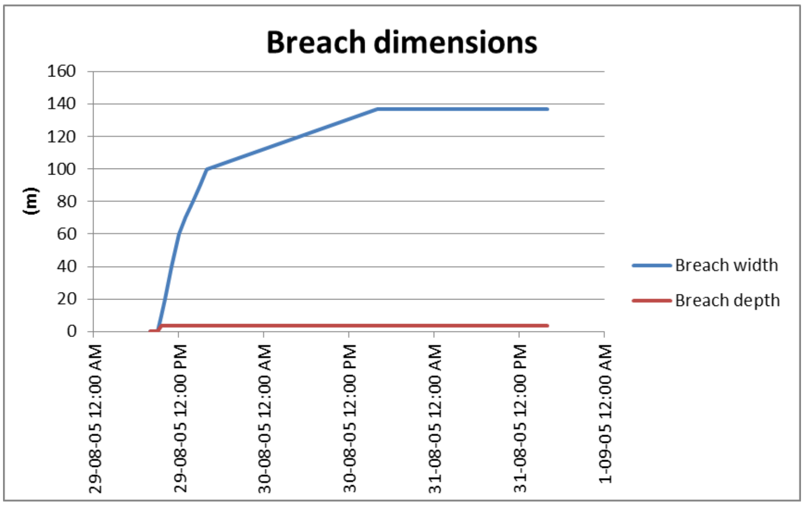

Figure 4.31: Development breach dimensions New Orleans

Figure 4.31 displays the development of the breach dimensions in New Orleans. In this graph the breach width and depth of the development with the emergency measures is displayed only. The breach width and depth without emergency measures is the same, since the emergency measure did not have effect on the breach development. The dimensions are based on observations during the flood (Seed, et al., 2008c), (Sills, et al., 2008), (Chaudhry, et al., 2010).

Emergency measure

As emergency measure Big Bags were deployed. The US Army Corps of Engineers responded immediately by placing Big Bags filled with sand in the breach. Using the National Guard Helicopter (see Figure 4.32) they started to place about 1,350 kg (≈ 1 m^3) heavy bags in the breach. These bags were completely washed away by the currents. The Big Bags were increased in weight to 2,700 (≈ 2 m^3) and 3,200 kg (≈ 2.5 m^3). The plans made to close the breach were changed in the field several times. This was due to the absence of systematic procedures that could be followed. Consequently, trial-and-error procedures were employed and it took several days to close the breach, most of the city was inundated and dewatering activities were substantially delayed (Chaudhry, et al., 2010), (Sattar, et al., 2008). The amount of manpower and equipment was sufficient.

Figure 4.32: Deployment of Big Bags (Seed, et al., 2008c)

Effect

The closure of the breach with Big Bags was not successful. Figure 4.32 shows already an (almost) equal water level at both sides of the flood defence. The bags were placed from the dike head towards the middle of the breach. In this way a horizontal closure is performed. The flow area is decreased and the velocity will therefore increase, resulting in larger water forces on the bags. This was shown by washing away of the bags. Closing the breach by means of Big Bags had effect when the water levels were equalled.

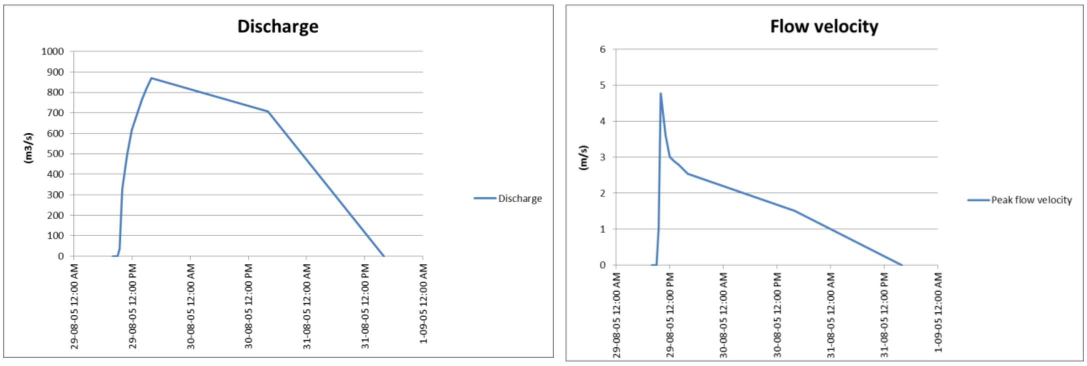

Figure 4.33: Development discharge and flow velocity New Orleans

Figure 4.33 shows the discharge and the flow velocity through the breach. After the breach the water level of the canal lowered, which explains the peaked velocity. The emergency measures had no effect on the discharge or flow velocity. For stability calculations, the peak flow velocity is needed. This is done by increasing the average flow velocity by a factor 1.1 (see Chapter 3.2.2). Calculations can be found in Appendix II and are done with formulas (3-7), (3-21), (3-22) and (3-23) from Zhu (2006).

The USACE started with Big Bags of 1,350 kg (≈ 1 m^3) which were washed away immediately. This behaviour can be observed in Figure 4.34. The USACE switched to Big Bags of 2,700 kg (≈ 2 m^3) and 3,200 kg (≈ 2.5 m^3). The stability of these Big Bags in the current can be seen in Figure 4.34. Calculations can be found in Appendix II and are done with the Izbash formula for individual stones.

Other closure methods, discussed and tested in hindsight, are two multi-barrier types and the closure of a bridge upstream of the breach. The first type of multi-barrier closure does not close the breach at the location of the breach itself. Buildings situated around the breach are used to create a barrier to raise the water level at the city side of the breach. By this operation the hydraulic head over the breach is decreased and the closure at the location of the breach can be fulfilled with smaller Big Bags. The second multi-barrier closure option reduces the flow through the breach by transverse dumped Big Bags. The breach itself can subsequently be closed with Big Bags. Another option is to close not the breach itself, but the Old Hammond Highway Bridge upstream. It is investigated that with these closure procedures the required weight of the Big Bags could be decreased (Sattar, et al., 2008).

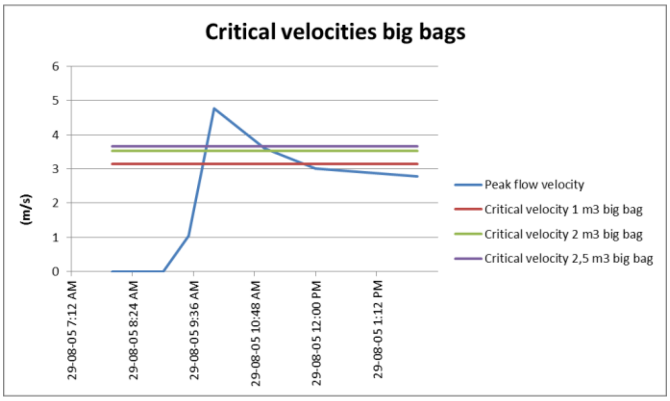

Figure 4.34: Critical velocities of Big Bags

Conclusions and learned lessons

The failure of the flood wall was induced by geotechnical failure. This emphasises the importance of geotechnical aspects in the design phase. Even if the design conditions are not reached yet, failure due to geotechnical causes can occur.

The closure with Big Bags at the location of the breach itself did not succeed. Closure procedures where the Big Bags were placed at other location were investigated to be more efficient. Less heavy Big Bags would then be sufficient to close the breach.

Again, a horizontal closure method was performed in first instance. This proved to be inefficient. The velocity of the current in the breach washed away the Big Bags.

The closure *did not succeed*, coupled to the framework, the technical aspect failed due to the horizontal closure method with *too light Big Bags*. Logistics were covered by the sufficient *availability of material and equipment*. The plan to close the breach *changed several times in the field*, this was not in favour of the organizational perspicuity.

4.2.4 Fischbeck, Germany, Flood of 2013

Location

Fischbeck is situated in the province of Saxony-Anhalt, in the north-eastern part of Germany. Fischbeck lies on the Elbe River. The city has a mere 672 inhabitant spread over almost 21 km². The surrounding area is mainly rural. A lot of nature reserves are present around Fischbeck. The location of the breach is about 300 kilometres from the mouth of the Elbe River. The total length of the Elbe River is 1,094 kilometres.


Figure 4.35: Location Fischbeck

Conditions

Heavy rain in late May and early June of the year 2013 caused river levels in several Central-European countries to rise. The conditions were more severe than during the flood in Saxony-Anhalt of 2002 described in (Horlacher, et al., 2007). Together with the Elbe River basin, the Danube River basin and the Rhine River basin had to discharge large quantities of water. On May 30 and June 1 as much rain fell as normally would fall on average in two and a half months. The return period of the rain is about 50 years. This precipitation caused return period of floods between 50 and 500 years dependent on the location. The flood caused 25 fatalities. Eleven people drowned in Czech Republic, eight in Germany and six in Austria.

Figure 4.36: Dike breach in Fischbeck, source: AP

On June 9, just after midnight, at the eastern side of the Elbe failure of the dike body started. The water level was at that moment about 20 to 40 cm beneath the crest of the dike. Sandbags and Big Bags were already deployed on the dike body to prevent a breach. This did not succeed and the dike breached. The water was still below the crest level of the dike and the failure was a geotechnical failure of the inner slope. The town of Fischbeck lies at a distance of about 2 km of the breach location. The water reached it very soon. Luckily the town of Fischbeck had been evacuated previously. The flood covered a large area. Up to the River Havel an area of tens of square kilometre was flooded (Delft University of Technology & Technische Universitat Dresden, 2013).

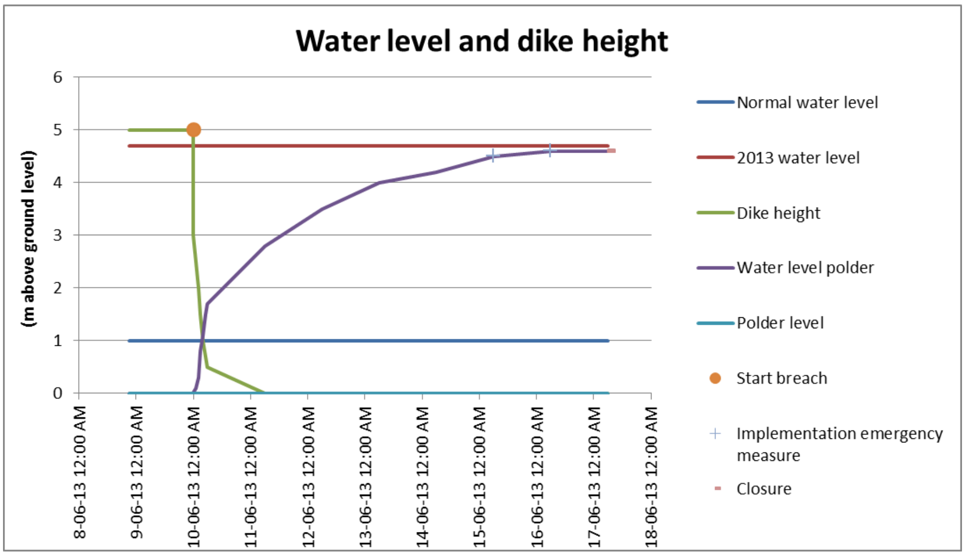

Figure 4.37: Water levels and dike height Fischbeck

In Figure 4.37, a sketch of the normal water level, the water level of 2013 and the water level in the polder are displayed, together with the dike height and the start and end of the breach. Assumptions about the polder water level have been made. The breach started due to macro instability of the inner slope. In a short time a wide breach developed. Emergency measures with Big Bags did not have effect. Vessels were sunk down and thereafter the breach was closed. The polder water level had almost reached the outer water level at the closure moment.

Characteristics dike

The dike had a crest height of about 5 meter above ground level. As can be seen in Figure 4.36 the slopes are moderate, about 1:3. No berm is present in this dike. The dike body consists of clayey sand. The subsoil beneath the dike body consists of sandy clay (Delft University of Technology & Technische Universitat Dresden, 2013).

Figure 4.38: Breach Fischbeck, source: Reuters/Thomas Peter

Characteristics breach

The location of the breach can be seen in Figure 4.38. The breach is situated at the corner of a dike system. According to eye witnesses the failure started with cracks in the crown. Hereafter a stepwise settlement of the inner slope occurred. This started at Sunday June 9 just after midnight. After some time, in the inner slope a slip circle developed. The sliding of the inner slope occurred fast. In 10-20 seconds the slope slipped off and the breach was initiated. The initial width of the breach was 50 m. After a couple of hours the breach width was developed to 100 meter. Behind the breach a scour hole of 2-3 meters deep was formed. The estimated discharge through the breach is up to 700-1,000 m^3/s (Delft University of Technology & Technische Universitat Dresden, 2013).

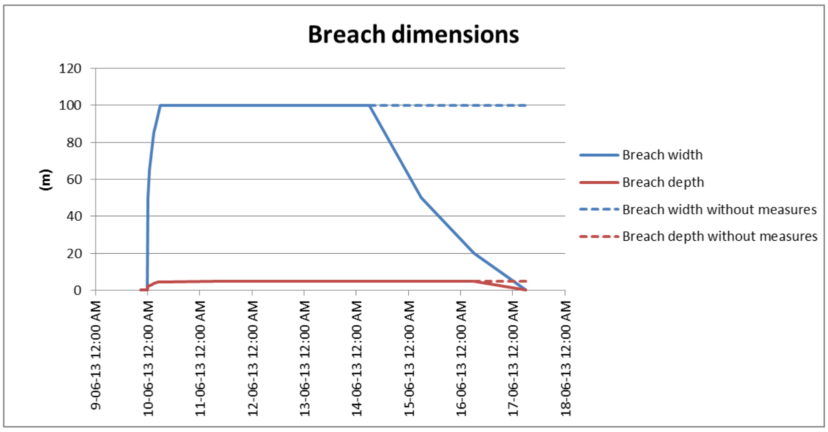

Figure 4.39: Development breach dimensions Fischbeck

Figure 4.39 displays the development of the breach dimensions in Fischbeck. The breach develops quickly and has reached an equilibrium state. The dimensions are based on observations during the flood (Delft University of Technology & Technische Universitat Dresden, 2013).

Emergency measure

Before the breach started, preventive measures were deployed. Sandbags and tarpaulin were put on the crest and slope of the dike. These measures were insufficient to avoid a breach. At the initial stage of the breach, there was tried to close the breach by means of sandbags, thrown into the breach. This measure had no effect. Next, barges were used as emergency measures. In total three barges were sunk down in front of the breach using explosives. In Figure 4.40 the closure operation is shown. The remaining gaps were closed with Big Bags. The barges reduced the inflow significantly. However the breach could be closed only when the water level at both sides of the dike was levelled out to equal water levels.

After the closure, a temporary flood defence was constructed. Sheet piling was used to make a core for the temporary dike body. An earthen embankment was raised in front of the sheet piles, between the barges. The new dike will follow a different alignment. The barges used to close the breach will be removed (Delft University of Technology & Technische Universitat Dresden, 2013).

Figure 4.40: Barges in the breach as emergency measure, source: dpa photo

Effect

Sandbags that were thrown into the breach were washed away immediately. It was realized that larger materials were needed. The barges that were sunk down in front of the breach did not have the intended effect. The area flooded, as can be seen in Figure 4.41. If with a hydraulic engineering perspective is looked at the decision to sink down barges, the failure of the emergency measure could have already been predicted. The ships had the effect of a horizontal closure. The flow velocity increased by sinking down the barges, making it even harder to close the remaining gaps. The breach could be closed when the water level was equal at both sides of the dike. Probably the German engineers did know that the probability of success to close the breach by sinking down barges was very small. However, if no attempts are undertaken, the inhabitants will blame the government for negligence.

Figure 4.41: Post flood situation Fischbeck, source: dpa photo

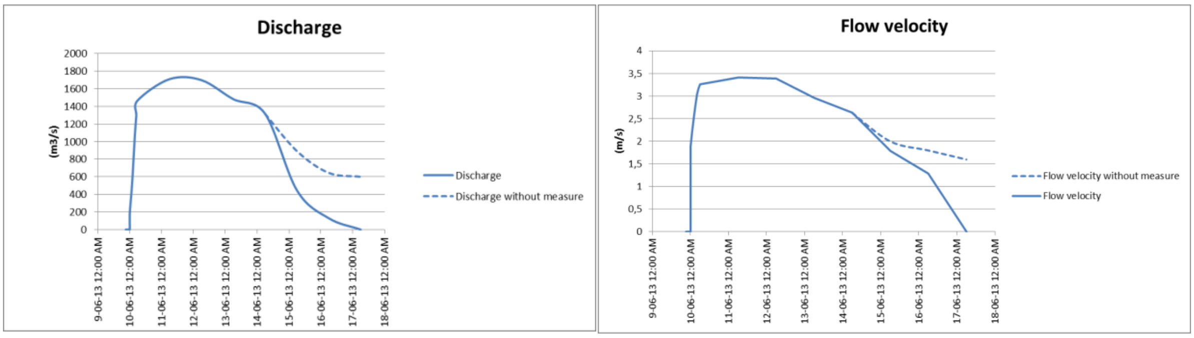

Figure 4.42: Development of discharge and flow velocity Fischbeck

Figure 4.42 displays the development of the discharge and flow velocity through the breach. Calculations can be found in Appendix II and are done with formulas (3-7), (3-21), (3-22) and (3-23) from Zhu (2006). Again the effectiveness of the measure can be seen by the increase in discharge if the measure would not have been applied. The flow velocity is again averaged. Since the area of a ship is taken for the calculation of the forces, the averaged velocity is allowed.

Figure 4.43 shows the critical velocity of the implemented ships. The ships were implemented when the polder water level was close to the outer water level. The forces on the ships were limited. However, the critical velocity for the ships was higher than the occurred velocity. Ships are floating objects, however they were sunk down and stabilized with big bags after. The moment the ships are just sunken down, they are not filled with big bags yet and are vulnerable for the large currents. Calculations show that the ships were stable during this situation. Calculations can be found in Appendix II and are done with the Izbash formula for individual objects.

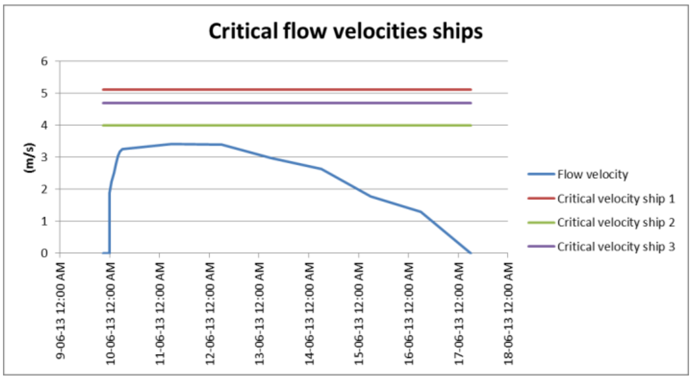

Figure 4.43: Critical velocity of the ships

Conclusions and learned lessons

The failure of the dike and the subsequent breach was induced by geotechnical failure. The inner slope of the dike slipped off. The water level was below the crest of the dike. Further investigation in soil mechanics is needed.

Political pressure can influence the decision to deploy emergency measures with a small probability of success. The inhabitants demand action from the government. If nothing is done they will blame the government for negligence.

Barges that are sunk down in front of the breach can cause a reduction in inflow discharge.

The closure *did not succeed*, coupled to the framework, the technical aspect failed due to the horizontal closure method with *insufficient vessels*. Logistics were covered by the sufficient *availability of material and equipment*. There was a *large pressure* from inhabitants to do something to close the breach, which forced a rushed decision.

4.2.5 Characteristics and lessons learned from other floods

The four cases discussed above are just a small fraction of the total amount of breaches that occurred around the world. In the literature more breaches are documented. Since discussing them all will be too abundant, a summarizing table of some events is included in this paragraph. In this way the lessons learned from these breaches will not be lost. In Table 4.1 breaches in the countries of The Netherlands, United States, Germany, China, Thailand, France, Japan, Hungary and Canada are summarized.

Table 4.1: Summarizing event table

Event	Characteristics	What went wrong/Emergency closures	Lessons learned
The Netherlands, Flood of 1916 Source: (Rijkswaterstaat, 1916)	• Breaches in water defences in several provinces • Mostly caused by overtopping	• No maintenance • Mixed results preventive measures • A few emergency closures by army (no details mentioned)	• Dikes need continuous maintenance • Too many breaches hamper effective closures

Zeeland, The Netherlands, Flood disaster of 1953 Sources: (Huis in 't Veld, 1987), (Gerritsen, 2005), (Rijkswaterstaat & KNMI, 1961)	• Breaches in water defences in the provinces Zeeland, Zuid Holland and Noord Brabant • Overtopping in combination with a (too) steep inner slope • Almost 48 km breaches in total • Almost 2000 fatalities • fl 1,5 billion damage	• Water 0,5 m higher than highest known • Mixed results preventive measures • Successful emergency closures in the cities Nieuwerkerk aan den IJssel and Ouderkerk aan den IJssel	• New approach for dike design needed: Delta Plan • Huge efforts are made by people during floods
Red River, Canada, 1997 Sources: (Rannie, 1998), (Simonovic, 1999)	• Flood wave in the river • $ 500 million damage	• Large scale (preventive) emergency measures were successful	• Set up flood management system is beneficial • Better management of emergency measures • More strict land-use control
Yangtze River, China, 1998 Sources: (Deltares, 2009), (Li, et al., 2003), (Deltares, 2011)	• Flood wave in the river • Failure due to overtopping and piping • Varying quality of the dikes	• Poorly designed water defences • Emergency measures successful	• Breach development of Chinese dikes • Importance of space for the river
Saxony, Germany, 2002 Source: (Horlacher, et al., 2007)	• Extensive rainfall • Over 100 breaches • Overtopping • € 1 billion damage	• Dikes designed for 1/100, high water was 1/125 flood • No mention of preventive measures • No successful emergency closures	• Important characteristics of failure mechanisms
New Orleans, USA, Hurricane Katrina 2005 Sources: (Briaud, et al., 2008), (Chaudhry, et al., 2010), (Sasanakul, et al., 2008), (Sattar, et al., 2008), (Seed, et al., 2008a,b,c,d)	• Set up by hurricane Katrina • Overtopping and piping under floodwalls • Over 1100 fatalities • $ 30 billion damage	• No mention of preventive measures • No successful emergency closures • No closed dike system • Political mismanagement	• Importance of soil mechanics, penetrations and transitions and decision making • It was possible to close breach with emergency measures
France, Xynthia, 2010 Source: (HKV lijn in water, 2010a)	• Sea storm • Dikes designed at 1/100 • Different failure mechanisms	• Bad maintenance • No clear information on threat • Violation of the	• Provide uniform information • Importance of spatial planning • Maintain dikes up

	• 47 fatalities • € 2.5 billion damage	spatial law • Emergency closures failed (no details)	to standards
Japan, Tsunami, 2011 Source: (HKV lijn in water, et al., 2012)	• Tsunami • Overtopping of walls • Over 19,000 fatalities • $ 210 billion damage	• No chance for preventive or emergency measure	• Spatial planning is important • Probability based design for water defences is needed
Chao Phraya, Thailand, 2011 Source: (Expertisenetwerk Waterveiligheid, 2012)	• Failure due to overtopping • Dike system of river dikes, irrigation canal dikes and a highway • Multiple breaches	• No real emergency measures; breach closure operations needed preparation	• Failure on transitions and connections • Need for research into emergency closures
East Germany, 2013 Source: (Delft University of Technology & Technische Universitat Dresden, 2013)	• Extensive rainfall • High water 1/50 • Water below crest so no overtopping • Piping and inner slope failure	• Preventive measures were taken • Emergency measures failed	• Need for research into emergency closures
Hungary, historical overview Source: (Nagy, 2006)	• Mainly overtopping • Also inner slope failure, piping and failure of structures	• No mention of preventive measures • Failed emergency closures (no details)	• Maintain dikes and especially structures • Get more insight in geotechnical failures

4.3 Conclusions

Successful closures are rare. Due to the complex situations which require quick actions in difficult circumstances the probability to effectively close a breach are small. The documentation of these closures is even more rare. However, it is useful to investigate cases because in this way common success or failure factors and lessons learned regarding an emergency closure can be obtained.

From the case studies a general lesson is that improvisation and quick action are essential to prevent large damages to both the dike and the hinterland. Important is that this quick action does not go at the expense of the hydraulic thinking. People want to close the breach as fast as possible and do generally not have the insight in the hydraulic processes and will, with a high probability, make wrong decisions. The consideration between quick action with the possibility of making matters worse or waiting some more time to make a more certain decision while the breach continues to develop is a hard one. This also proves that further investigation is needed in emergency closures.

From the case studies can be seen that every closure attempt using a horizontal closure method fails. In most cases random objects are thrown into the water next to the dike heads, resulting in a horizontal closure method. A more promising method is the vertical closure method, which could be supported by a framework to prevent smaller objects from being pushed away by the currents. However, this closure method is usually performed under pressure from inhabitants or the government.

The use of vessels in breach closing could be useful. Although, the probability of closing the gap in a single operation with one ship is a small one. High risks of making matters worse are present by a closure by means of a ship, piping could easily occur and. Ships, barges or vessels can be used to limit the discharge through the breach. This will make it easier to close the breach. The question remains if the benefit of less discharge outweighs the loss of the vessel(s), since the breach could probably be closed without a vessel too.

To place the general conclusions in the framework, the technical aspect is likely to succeed more with a *vertical closure* method. Vessels can reduce the discharge through the breach. *Improvisation and quick action* is essential for a breach closure, just as the *availability of material and equipment*.

5. Simulations of emergency measures

5.1 Introduction

In this chapter, the individually treated aspects from Chapter 3, breach processes and emergency measures, are combined. Together with the lessons learned from Chapter 4 about the case studies, the effect and applicability range of emergency measures is investigated. This is done by simulating the physical processes. A developing dike breach is simulated and within this developing breach, different emergency measures are implemented at different locations to study their effect. Simulation of such processes allows identification of breach characteristics like duration and breach stages during closure attempts. Furthermore, it can be used to optimize closure strategies and measures and it can help flood managers to prepare for emergency situations. Different scenarios can be simulated and 'off the shelf' strategies can be prepared for specific situations.

The first objective of this chapter is to identify a suitable tool for carrying out such simulations. XBeach is identified as such a tool and Chapter 5.2 will come up with improvements for XBeach to tailor it for simulation of emergency measures in dike breaches. The second objective is to understand the effect of an emergency measure and to set application limits for successful implementation based on stability requirements regarding flow velocity, breach dimensions and measure specific failure mechanisms. The third objective is applied for the first layer of emergency measures, a complete closure and for several dike characteristics. The fourth and final objective is to link the results to different (i.e. more realistic) timeframes for closure logistics. These topics are addressed in Chapter 5.3. This chapter ends with conclusions and recommendations in Chapter 5.4.

5.2 XBeach

Breach development models

There are only a very limited number of models to predict breach development. None of the models are capable of simulating emergency measures in a developing breach, i.e. those that do exist are for breach development prediction only. Examples are the models BREACH from Steetzel & de Vroeg and the dissertations of Visser (1998) for non-cohesive soil and Zhu (2006) for cohesive soil. Visser calibrated his model BRES (Breach Erosion in Sand-dikes) in a field experiment; Zwin '94. Another model which is not specifically designed for breach modelling, but is capable of it is XBeach. The section below explains how XBeach works. In Figure 5.1 and Figure 5.2 the comparison of BRES versus Zwin '94 and the comparison of XBeach and Zwin '94 respectively are made. Both figures show a good agreement of the calculated and the measured points.

The approach of Xbeach is in fact not that much different than that of Visser (1998). Because of the higher flow velocity at the bottom of the side slope (see Chapter 3.2.2), more erosion takes place at these spots, causing steeper slopes which leads to collapse of the side slopes. Despite the different formulas, the formulas have the same effect on the shape and development of the breach.

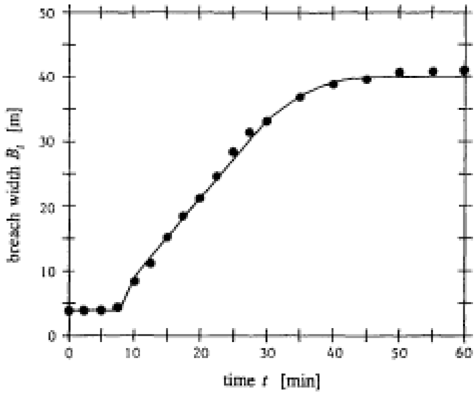

Figure 5.1: Comparison of BRES with Zwin '94 field experiment from: (Visser, 1998)

It is shown that XBeach is capable of simulating the non-interfered breach process quite well. The real advantage of XBeach over other software is the possibility to model non-erodible layers in a developing breach in XBeach. Other software such as BRES (Breach Erosion in Sand-dikes), developed by Visser (1998), are only capable of simulating the non-interfered breach process. The possibility to implement these non-erodible layers as simulated emergency measures is a key aspect for the obtained simulations. XBeach is thus capable of modelling developing breaches with simulated emergency measures which makes it suitable for the modelling done in this thesis. XBeach is a numerical model that was originally designed to assess natural coastal response during time varying storm and hurricane conditions (Roelvink, et al., 2009). With some adaptations and assumptions, it is made suitable for the simulation of emergency measures in dike breaches.

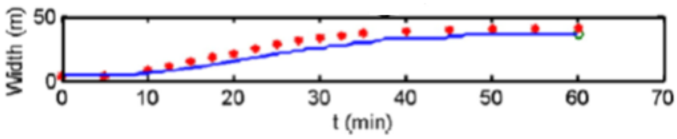

Figure 5.2: Comparison between XBeach (blue) and field experiment Visser (1998) (red), from: (Roelvink, et al., 2009)

For the simulations in this thesis, the tested XBeach Zwin model is used as starting point. The case is adapted in its shape, water levels and bathymetry. For the modelling of emergency measures non-erodible layers are added. In this way the model is formed into an independent case. In this model the parameters can be tuned to the preferences per situation. The next paragraph describes these preferences per emergency measure.

How does XBeach work?

XBeach is a nearshore numerical model to assess natural coastal response during time varying storm and hurricane conditions (Roelvink, et al., 2009). The model has proven to perform well in different scenarios, including overwash, dune erosions and breaching. XBeach is capable of the correct simulations of the extreme conditions that occur during the breach process. The model is specifically tested for this situation. XBeach is capable of simulations with non-cohesive sandy soil only. The breach process of the model is compared to the measurements of the breaching experiment at ‘t Zwin performed by Visser (1998). The model gives good results for this case (Roelvink, et al., 2009). The model makes calculations two-dimensional and integrated over the depth (Roelvink, et al., 2010). XBeach is capable of computations with non-cohesive sand only. The development of the breach is in this model is calculated for two main processes; sediment transport and avalanching. Sediment transport formulas have especially effect on the erosion at the bottom of the breach. These transport formulas are driven by the time varying flow. The second process is the avalanching mechanism. This mechanism makes the side slopes shear off if the gradient of the slopes becomes too steep.

Within XBeach software the emergency measures are modelled as non-erodible layers. These layers can be applied at any location and at size. As the name suggests, at locations where such a layer is applied, the surface is not able to erode and will stay in the original shape and height. This means that the emergency measure is always stable, no matter how large the flow velocity or the scour around the measure is. In the simulations non-erodible layers are not specifically visible; the surface of that layer will have the same colour as the adjacent ground. Simulated emergency measures are thus not visible. A non-erodible layer is unconditionally stable. The effects of scour around it of high flow velocities, which would make real emergency measures fail, do not have any effect on the non-erodible layers. This makes it necessary that the stability of the emergency measure needs to be calculated by hand afterwards, with the output of the model. The output of the model is formed of the flow velocity, the breach development in depth and width, the water levels and the bathymetry. It is not possible to replace or add a non-erodible layer during a simulation. Another limitation is that XBeach is not able to calculate 3 dimensional phenomena. Piping is not integrated in the model and is thus also part of the hand calculation afterwards.

Pros and cons
The advantages of using XBeach are discussed above and are the capability of simulating extreme conditions and the possibility to implement non-erodible layers as simulated emergency measures. Also, the non-interfered breach development in XBeach is tested by the Zwin case from Visser (1998).

XBeach has some limitations too. It is just as important to know the limitations as to know the capability of the software. Limitations of XBeach are:

1) implemented non-erodible layers are not adjustable in time;
2) the non-erodible layer is always stable;
3) XBeach is no 3D model;
4) XBeach is capable of simulations with non-cohesive sand only.

Ad. 1)
The implemented non-erodible layers are not adjustable in time. This means that the non-erodible layer needs to be placed somewhere before each run and stays at that position the whole simulation. In this way a complete stepwise closure can not be modelled at once. A solution for this problem is to take the output of a simulation without measures, adjust the bathymetry to simulate an emergency measure and make the new bathymetry, the water levels and a non-erodible layer the input for the next run. This requires however a lot of work.

Ad. 2)
The non-erodible layer is always stable. This means that no matter how large the flow velocity or the scour around it gets, the layer will stay in its place. Emergency measures can become unstable if large flow velocities or scour occur. Therefore, these failure mechanisms are checked by hand calculations.

Ad. 3)
Because XBeach is a 2DH model, it is not able to simulate 3D effects. The piping phenomenon can not be modelled with XBeach. Also, the secondary vertical flow effects can be captured only partly. It is therefore not possible to model immediate closures like the closure by a vessel, caisson or PLUG. If a successful implementation of these measures

is modelled the breach is closed at once and no water will flow through the breach anymore.

Ad. 4)
XBeach is capable of simulations with non-cohesive sand only. Since the model calculations are done with a non-cohesive material, the erosion will happen quite fast. In reality, dikes are composed of a cohesive cover over a non-cohesive core or are fully composed of cohesive material. If the works of Visser (1998) on breaches in non-cohesive soil and Zhu (2006) on breaches in cohesive soil are compared, one can conclude that the breach process is essentially the same. However, the way of erosion is somewhat different. Erosion of cohesive soil takes mostly place in lumps while non-cohesive material erodes gradually. The difference between cohesive and non-cohesive material in dike breaching is mainly the time it takes to erode the soil. This again can be seen in the laboratory experiments conducted by Visser (1998) and Zhu (2006). The results obtained by the model are thus correct with respect to the effect of the emergency measure. However, the amount of time, for the same amount of erosion to take place in reality, is underestimated in the model. In reality dikes with cohesive parts in it will take way more time to erode.

Point of attention when interpreting the model outcomes, is that the simulated emergency measure is successfully implemented in one piece at once. This is not realistic. Furthermore the time that it takes to erode away the dike is an underestimation of the worst case scenario of a real dike. The simulated dike consists of sand only and has already a depression below the water level in it. The depression accelerates the erosion process. Real dikes consist not only out of sand, so it takes a longer time to erode, this is discussed in Chapter 5.3.10.

5.3 Simulations

5.3.1 Model set-up

Emergency measures
The simulations that are done by the numerical model XBeach are based on the emergency measures discussed in Chapter 3.3.2, namely ships and barges, caissons, scaffolding, Big Bags, emergency dike and the PLUG. Modelled are the measures: Big Bags, scaffold, a vessel in front of the breach to reduce discharge and an emergency dike. These emergency measures are modelled. They are chosen since they represent the possible closure strategies: horizontal closure (Big Bags), vertical closure (scaffold), reducing the discharge (vessel) and a closure outside the breach (emergency dike). The direct closure by means of a ship is not possible to model since the piping effect can not be modelled.

Dike
Dike dimensions are based on an imaginary dike, but representative for a common dike in the Netherlands. The way the dike is modelled is described next. Slopes of the dike are made 1:3 and the crest width is 4 m. The dike is 2.75 m high and surrounding ground level is at 0.0 m. There is chosen for a constant outer water level of 2.15 m and no waves. This is representative for a primary water defence in a river system where the water level will be more or less constant during a breach caused by the large amount of water stored. To make sure the breach process starts at the desired spot, a dip in the crest is modelled. The dike crest is lowered to +2 m from ground level and the depression has a width of 1 m. The grid

is non-equidistant with grid sizes gradually varying from 0.5 m near the breach to approximately 50 m far away from it, see Appendix III for the grid and other model scripts.

Polder

The polder area is determined in such a way that all the breach stages are simulated within a reasonable calculation time of the model. As polder area ±2 km^2 is chosen. At the start of the simulation, the conditions of the dike are as described above. There will be made simulations with some different characteristics as a kind of sensitivity analysis. In this way the effect of a berm, varying outer water level, moment of implementation or a larger or smaller polder area can be investigated.

Measurement locations

As mentioned before the output of the model is formed of the flow velocity, the breach development in depth and width, the water levels and the bathymetry. To be more clear where these outputs are measured, a sketch is made to show the precise locations. Flow velocities and water levels are measured upstream, in the breach and downstream, all in the symmetry axis of the breach. Figure 5.3 shows the locations. White spots with numbers 1, 2 and 3 are measurement points. Point 2 is situated in the centre of the breach and the points 1 and 3 are 30 meter upstream and downstream of the centre. As was shown in Chapter 3.2.2, the velocity in the centre of the breach is the averaged velocity. At the line labelled with number 4, the transverse velocity profile is measured. In this way the maximum velocity is measured. Measurements are done at these points to get a complete view of the velocity in the breach itself as well as up and downstream of the breach. For the forces on the emergency measures, the locations with numbers 2 and 4 are important. To check the effect of a measure, also the downstream location, number 3 is interesting. The breach width is measured at the crest of the dike. The breach depth is measured from the dip downwards. The left side of this figure is the outer water side, the right side is the polder area.

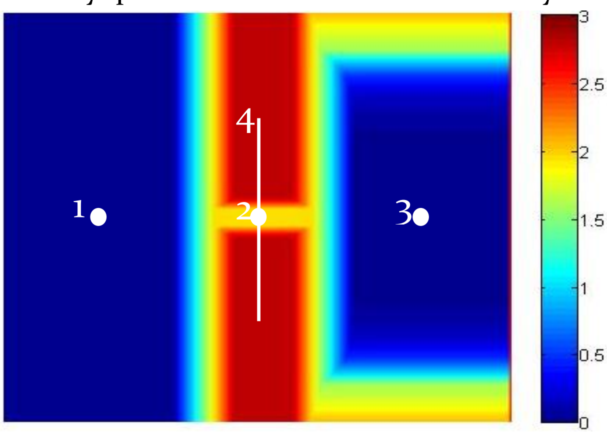

Figure 5.3: Top view of the breach simulation at t = 0 with the measurement locations, elevation height in meters

Discussion

With this simulation set up, the most unfavourable scenario for a dike is simulated. The dike consists of loosly packed sand instead of soil which has been in a dike for decades. Furthermore, no outer protection or other favourable dike aspects like revetments are modelled. Another issue is that the sediment transport formula of Van Rijn used in XBeach, is not capable to simulate the sediment pick up rate at high velocities correct. An overestimation is done since it takes hindered erosion not into account. This is a phenomenon occuring at high flow velocities where underpressures in the bed limit the erosion capacity. In XBeach an artificial limitation is set successfully to create the same effect as this physical erosion limitation. The breaching process in XBeach is tested for the Zwin dike. There is assumed that XBeach is still able to produce accurate results for the used dike used in this chapter since the dike can be considered as a scaled Zwin dike with a factor of about 1.2.

5.3.2 Start scenario; do nothing

Starting point

As first scenario, the 'do nothing' scenario is simulated. In this case no emergency measure is implemented. The model set up is as described in the paragraph above.

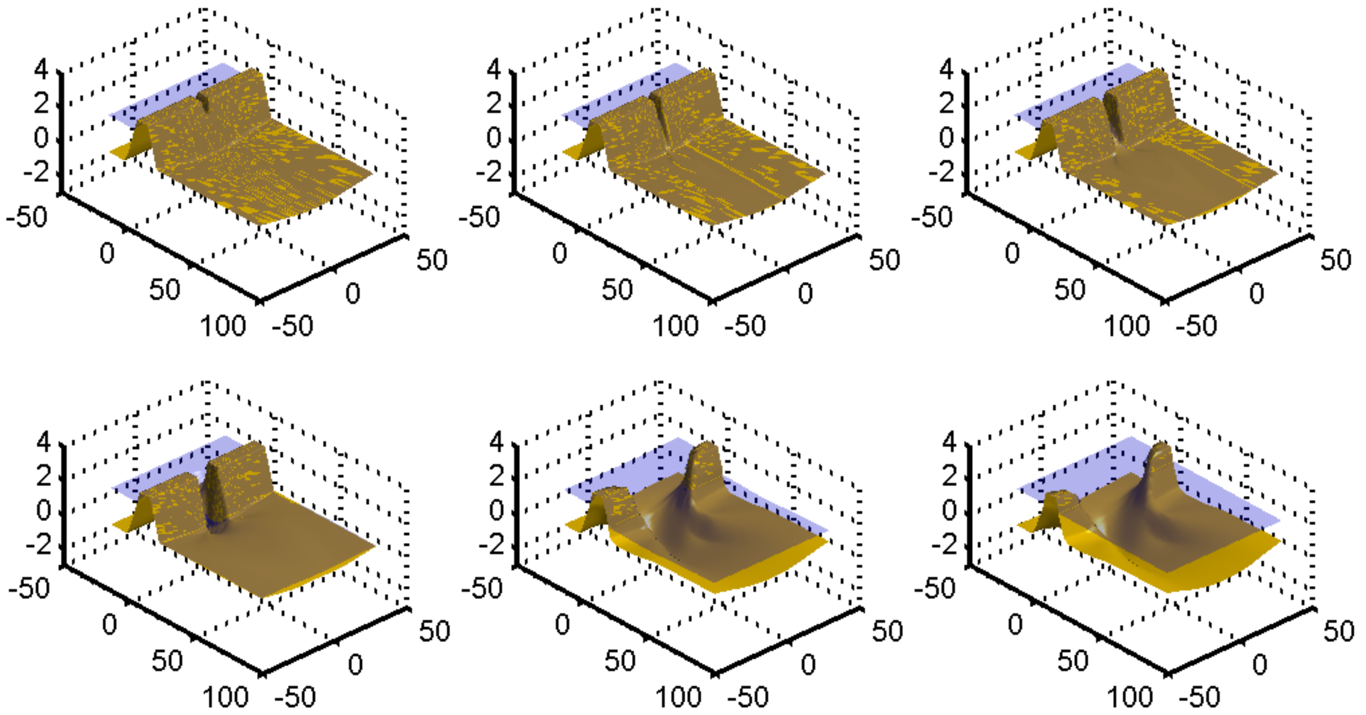

Figure 5.4: Do nothing scenario, snapshots taken at t = 0, 8, 15, 20, 40 and 65 minutes

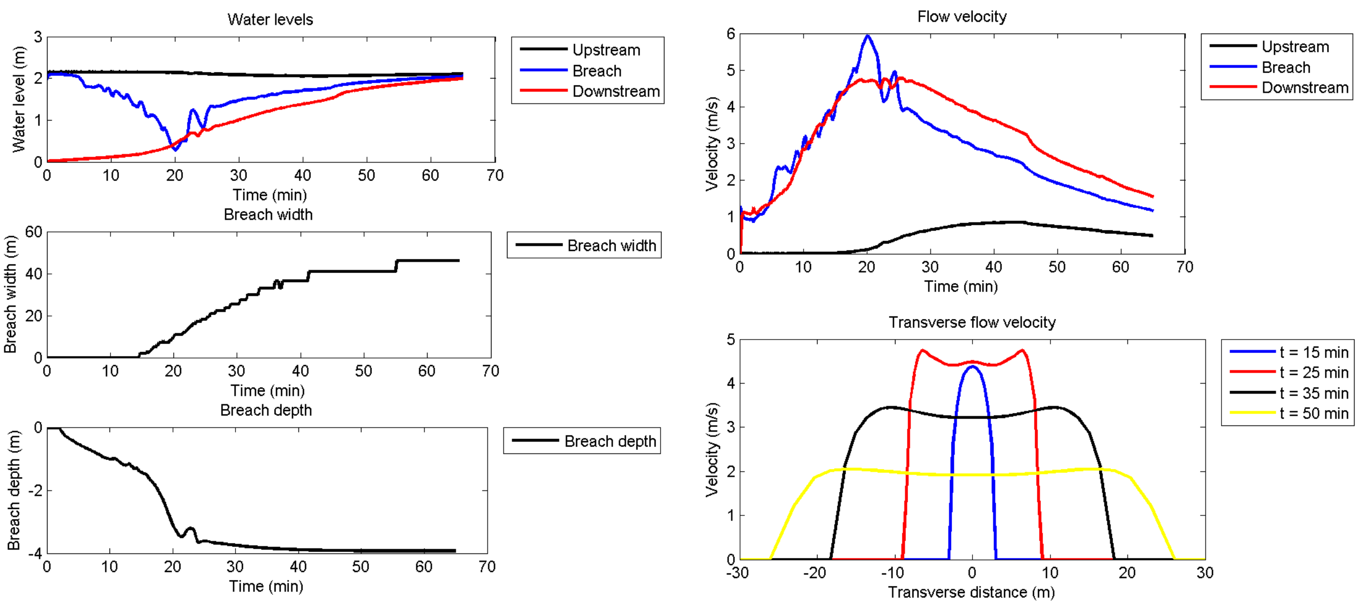

Figure 5.5: Water level, breach dimensions, and flow velocities of a non-interfered breach

Analysis
Figure 5.4 displays the non-interfered development of the breach. Six snapshots are taken from the process, at t = 0, 8, 15, 20, 40 and 65 minutes. The breach starts by cutting through the dike body first and then expanding parallel to the crest, exactly as described in Chapter 3.2.2.

Figure 5.5 displays the water levels, breach width and depth and flow velocities. In the upper left plot the water levels at the upstream location, in the breach itself and at the downstream location are shown. The upper water level stays at a constant level of 2.15 m. Downstream of the breach the water level in the polder rises towards the level of the outer water during the breach development. The water level in the breach starts at the level of the depression made in the dike and decreases as the breach is cutting through the dike. It rises again as the polder area is filling up with water and consequently the water level in the breach rises with it. The water level in the breach has an irregular shape. This is caused by the high morphodynamical and hydrological activities in the breach. The width of the breach is displayed in the left centre plot, the depth of the breach in the lower left plot. The width development starts with a delay compared to the depth development. This is the case because the breach first cuts itself through the dike body. For both plots hold, the further in time, the slower the development. The breach width develops in the end stepwise. This is due to the avalanching process only since sediment transport does not take place anymore because of the low flow velocity. Upper right the velocities upstream, in the breach itself and downstream (measured at the locations indicated with the numbers 1, 2 and 3 in Figure 5.3) are plotted. The shape of the velocity curve is typical for non-interfered breach development. Top velocity in the breach is almost 6 m/s. The irregular shape is again due to the high morphodynamical and hydrological activities in the breach. The lower right plot displays the transverse velocity (measured at the location indicated with number 4 in Figure 5.3). If the shape is compared to the velocity profile of Figure 3.12 and Figure 3.13 a strong similarity is observed. At the first stage of the breach, the velocity is the highest in the centre of the breach. As the breach develops, the velocity profile changes to a profile where the velocity at the slopes is somewhat higher than in the centre. This is also observed in Ren (2012). Probably, this is due to the change from subcritical to critical flow, further research is needed.

Key findings
The breach development in XBeach in the case where no emergency measures are implemented takes place exactly the same as described by literature of Visser (1998).

5.3.3 Big Bags; horizontal closure

Starting point
The emergency closure with Big Bags is modelled as a horizontal closure from t = 30 minutes on. The first 30 minutes are non-interfered breach development and are the same as in the ‘do nothing’ case. There is chosen for 30 minutes because there is time needed to bring in material and equipment. The breach is closed from the sides, see Chapter 3.4.1 for the definition of horizontal closure. The strategy is to stabilize the edges of the breach with Big Bags on the side slopes. Below, the first layer of a closure with Big Bags is simulated. In this way the starting point of this method can be judged. The first layer is important because it shapes the breach development for the remaining closure. In the model there is assumed that the Big Bags are placed at the slopes of the breach, preventing the breach to develop further in width. Big Bags in a horizontal closure are modelled as non-erodible

layers at the sides of a breach, preventing the breach to develop in width. As mentioned before the non-erodible layer is unconditionally stable. For this case the non-erodible layer is applied with a width of 25 m at the crest of the dike. They are applied with a slope of 1:2 and reach until a depth of 5 meter below ground level. The vertical erosion is not able to reach to that level. To illustrate the location of the non-erodible layer and thus the location of the Big Bags Figure 5.6 is composed. Notice that this Figure has a different horizontal and vertical scale.

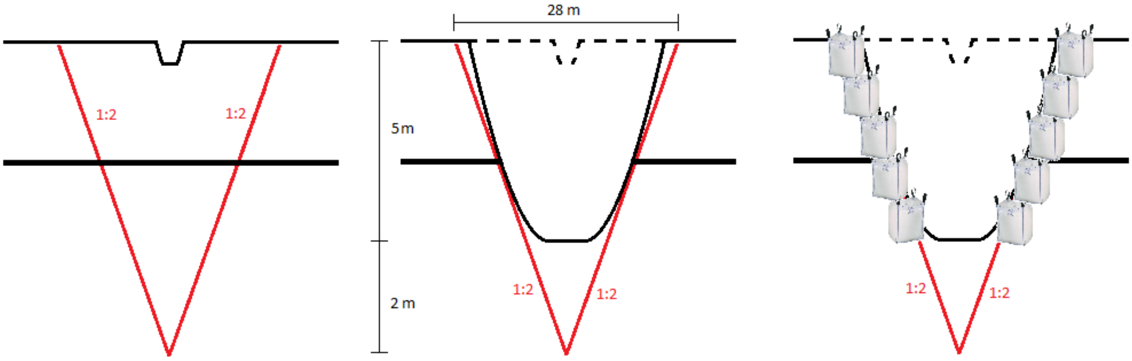

Figure 5.6: Location of the simulated emergency measure Big Bag

In Figure 5.6 three front views of the dike with emergency measure Big Bags are displayed. The black lines are the contours of the dike with the depression visible and the red line is the non-erodible layer of the model which represents the placement of Big Bags. The left image in this Figure is the initial situation. The breach development still needs to start. The centre image is the situation where the Big Bags are implemented. The development in width of the breach is stopped and only vertical erosion is possible. For visualisation purposes the right image is composed. The Big Bags are in this image visible and a better idea about the simulation of the emergency measure can be made.

Analysis

Bathymetry

Figure 5.7 displays the development of the breach, including the implementation of the emergency measure as shown in Figure 5.6. Six snapshots are taken from the process, at t = 0, 8, 15, 20, 40 and 65 minutes. The breach starts as a normal breach, developing by cutting through the dike body first and then expanding parallel to the crest, as described in Chapter 3.2.2. This continues until the non-erodible layer, as a simulation of Big Bags is implemented. From this moment on, the breach can't develop sidewise any more. However, the downward direction is not restricted by the emergency measure. This results in a more powerful vertical erosion leading to a deeper scour hole than in the 'do nothing' case. As comment three things need to be mentioned. The non-erodible layer is always stable, so no matter how large the flow velocity is, it will stay in its original position. This is also true for the scour around the Big Bags what could make it unstable. No matter how large the scour is, the non-erodible layer will stay in its original position. Also, the placement of the Big Bags is not taken into account. The Big Bags will not be placed in the exact position as desired. These things are discussed in more detail later this paragraph.

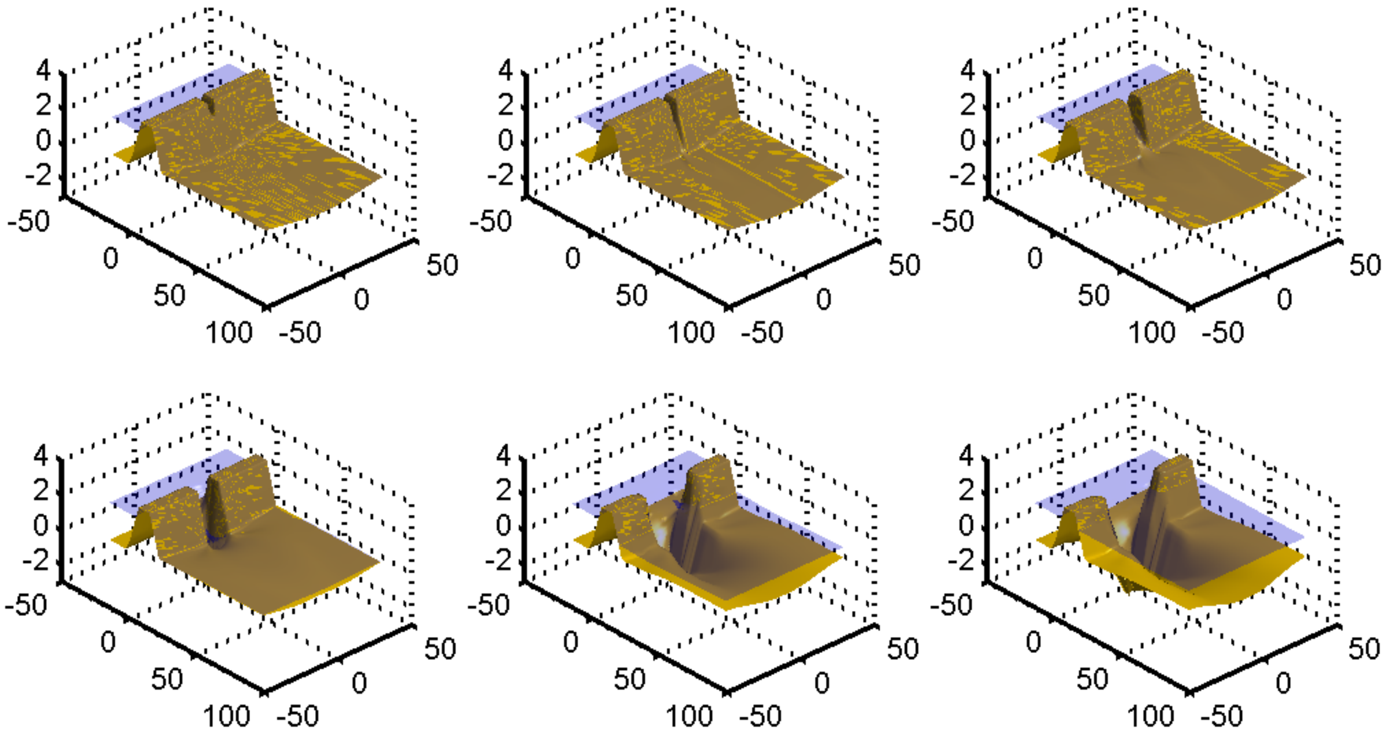

Figure 5.7: Effect of Big Bags in a developing breach, snapshots taken at t = 0, 8, 15, 20, 40 and 65 minutes

<u>Breach dimensions, water levels and velocities</u>
Figure 5.8 displays the water levels, breach width and depth and flow velocities. In the upper left plot the water levels at the upstream location, in the breach itself and at the downstream location are shown. The width of the breach is displayed in the left centre plot, the depth of the breach in the lower left plot. The limitation of the width is clearly visible at t = 30 minutes. If at that moment the development in depth is checked, an increase in depth can be observed. This extra depth can also be seen if the graph is compared to the depth graph of the 'do nothing' scenario in Figure 5.5. Upper right the velocities upstream, in the breach itself and downstream (measured at the locations indicated with the numbers 1, 2 and 3 in Figure 5.3) are plotted. Top velocity in the breach is almost 6 m/s. The velocities the first 30 minutes are the same as in the 'do nothing' case. From 30 minutes on, the velocity is somewhat larger than the 'do nothing' case. This can also be observed in the lower right plot. The lower right plot displays the transverse velocity (measured at the location indicated with number 4 in Figure 5.3). The velocities measured at t = 35 and t = 50 minutes are higher than in the 'do nothing' case.

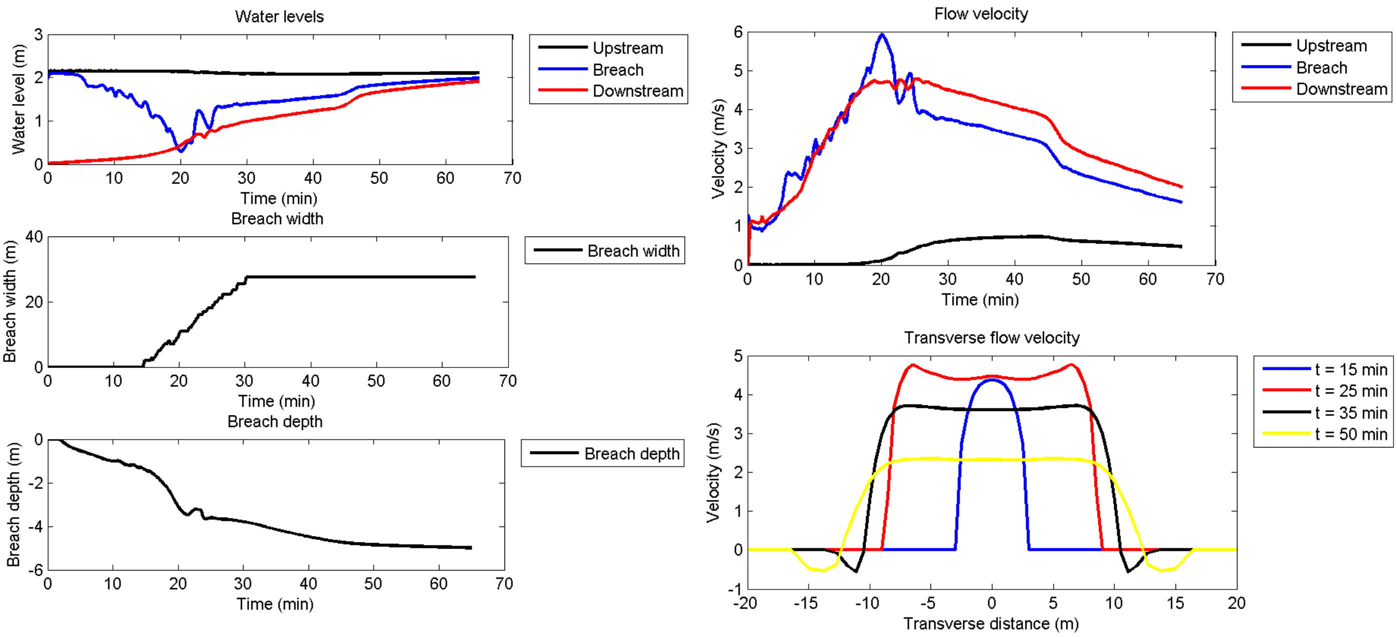

Figure 5.8: Water levels, breach dimensions and flow velocities of a breach with emergency measure Big Bags

Comparison to 'do nothing' scenario

If the effect of the implementation of Big Bags is compared to the scenario where no emergency measures are applied, the effect is visible. In Figure 5.9, the upper plot is the elevation map of the 'do nothing' case, the middle one is the elevation map of the Big Bag case. Both are taken at t = 65 minutes. In the lower plot these two situations are subtracted: the situation where Big Bags are implemented is subtracted from the case without emergency measures. The red areas are spots where the bed level is higher in the case of Big Bags than in the case without emergency measures, the opposite holds for the blue areas. In the case without emergency measures, a scour pit develops as well, however, in the lower plot of Figure 5.9 the additional depth of the scour hole when implementing Big Bags is shown. It is clearly visible that the implementation of Big Bags, at the sides of the breach, prevents widening of the breach and results in a deeper scour hole. The width of the erosion is reduced with respect to the 'do nothing' case. This is visible by the red areas at the sides of the breach. These are the by Big Bags protected parts of the dike. The outer water is on the left side in the Figures.

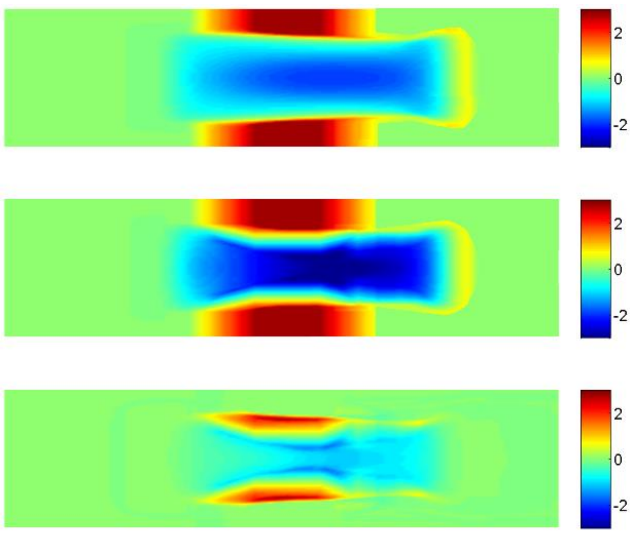

Figure 5.9: Elevation map of 'do nothing' case, Big Bag case and the differences in bed level in meters at 65 minutes

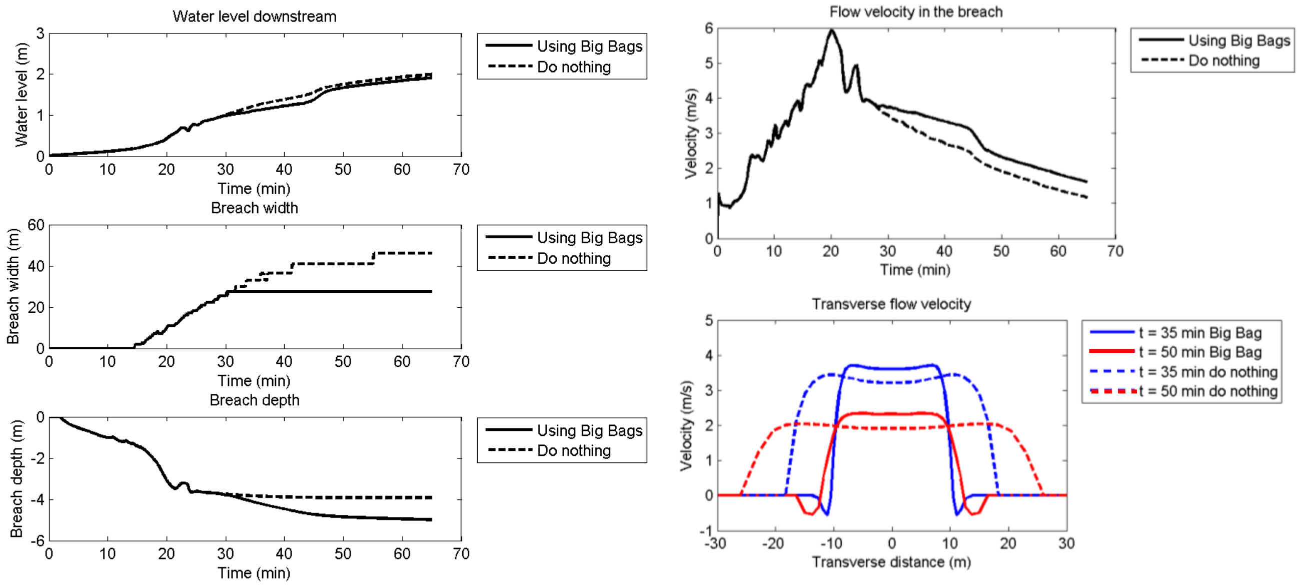

Figure 5.10: Differences water levels, flow velocities and breach dimensions Big Bags and 'do nothing' case

Figure 5.10 displays the same graphs as Figure 5.8, however, the results of the 'do nothing' case are included this time with dashed lines. Regarding the water level, the implementation of Big Bags results in a somewhat lower water level downstream of the breach. The breach width is limited by the Big Bags and therefor the depth is increased if compared to the 'do nothing' case. The flow velocity in the breach is, due to the larger hydraulic head over the breach, somewhat larger. The transverse velocity is higher, but takes place over a smaller distance since the width development is limited.

<u>Applicability ranges</u>

As mentioned, the non-erodible layer in the model is always stable. In reality the flow velocity and scour around the Big Bag can cause instability and subsequently washing away of the Big Bag and thus failure. Also, during the placement of the Big Bags instability can occur. The Big Bags possibly do not arrive at the desired location due to the large currents. To determine the range wherein Big Bags can be applied successfully, hand calculations are done. Instability due to scour and the actual implementation are not treated.

The flow velocity is of an enormous influence on the stability of Big Bags. In Figure 5.11 the calculated velocity at locations where the Big Bags are placed, thus at the sides of the breach is displayed. Along this velocity curve, the breach stages I till V are marked. These are determined by comparing the developing breach to the breach stages discussed in Chapter 3.2.2. The critical velocities of different sized Big Bags are displayed as well. If the flow velocity is below this critical level, Big Bags are stable and can be successful applied. This velocity is calculated with the Izbash formula. Izbash is applicable since it considers forces on individual 'stones'.

$$u_c = 1.2 \cdot \sqrt{2\Delta g d} \qquad [\mathrm{m/s}] \qquad (5\text{-}1)$$

Table 5.1: Calculation critical flow velocity of Big Bags

Size big bag	d [m]	u_c [m/s]
1 m^3 (1350 kg)	1,0	3,1
2 m^3 (2700 kg)	1,26	3,5
3 m^3 (4050 kg)	1,44	3,8

As can be seen in Figure 5.11, the critical velocities of the Big Bags are quite close to each other. Big Bags are only stable in the stages I, II, the first part of stage III and the last part of stage V. The implementation of Big Bags in end of stage III and stage IV is not useful since the large flow velocity will flush away the Big Bags.

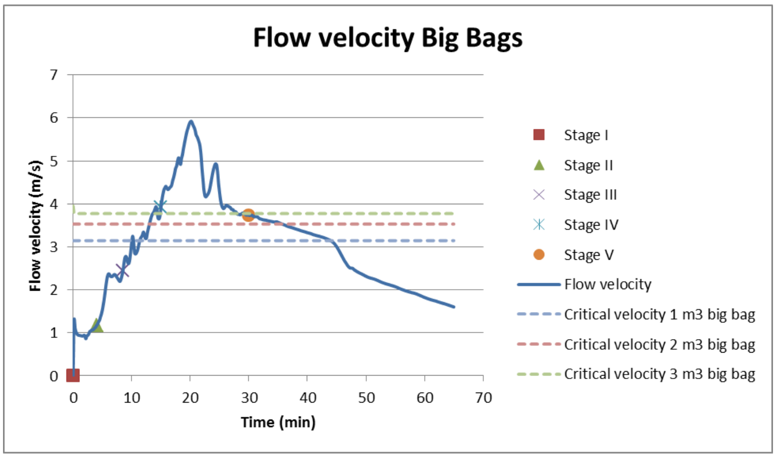

Figure 5.11: Flow velocity Big Bags

Another application range could be set by the dimensions of the breach. This has to do with the placement of the Big Bags. To investigate this, the same approach as for the velocity is followed. The calculated depth and width, including the breach stage, are plotted in Figure 5.12.

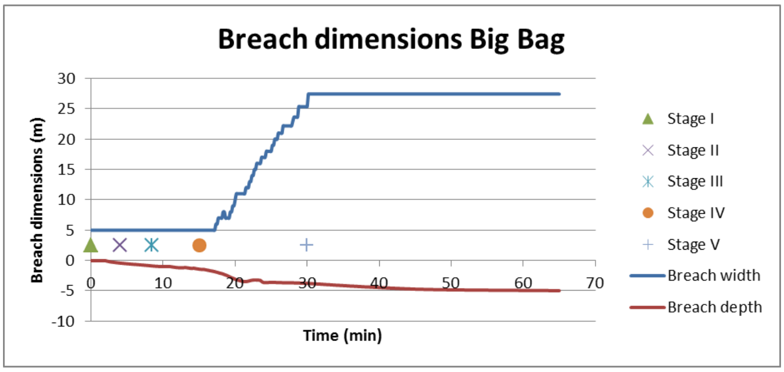

Figure 5.12: Breach dimensions Big Bags

As discussed are Big Bags stable in stages I, II, the first part of stage III and the last part of stage V. If we compare this with the breach dimensions at those moments, stage I and II will not give any trouble implementing these Big Bags. However, the depth of the breach in stage V is much larger than in stages I and II. The flow velocity during stage V is about 2 m/s. This will drag the Big Bags which are sinking down probably to an unwanted spot before they hit the bottom.

Key findings and discussion

Big Bags are according to the calculations applicable in stages I, II and the last part of stage V. For stage V with small flow velocity there is no interest in closing the breach with an emergency measure so this stage will be left out. In the first two stages the stability requirements for flow velocity, breach dimensions and water depth are met. In stage V, when the water depth is large, the Big Bags will probably be dragged away by the currents before they hit the bottom. The implementation of Big Bags does have a favourable effect in breach width a polder water level. For dikes constructed completely out of sand (these dikes do not exist, however, for the research purpose this is assumed) the Big Bags need to be placed within 15 minutes to be effective, otherwise the flow velocity becomes too large. Despite the impossible time limit, the effects of the Big Bags as emergency measure are clearly visible. The water levels, velocities and breach dimensions are influenced by this emergency measure. An important point that is not treated in this thesis is the instability due to scour. Also the time that it takes for the scour to develop to such a level that the Big Bag becomes instable is not treated. Calculations regarding scour are a recommendation for future research. This is also the case for the way of implementation and the movement of the Big Bags before the reach the breach bottom.

5.3.4 Scaffold; vertical closure

Starting point

The emergency measure scaffold is modelled as a vertical or bottom up closure from t = 20 minutes on. The first 20 minutes are non-interfered breach development and are the same as in the 'do nothing' case. See Chapter 3.4.1 for the definition of vertical closure. Below, the first layer of a closure with a scaffold is simulated. The first layer is important because it forms the breach development for the remaining closure. In the model there is assumed that the scaffold is placed at the bottom of the breach, preventing the breach to develop further downward. The scaffold in a vertical closure is modelled as a non-erodible layer at the bottom of the breach. For this case the non-erodible layer is applied at a level of -0.5 m. In practice, a threshold is created, supported with a scaffold to block the objects thrown in at the upstream side against washing away by the current. To illustrate the location of the non-erodible layer and thus the location of the scaffold Figure 5.13 is composed.

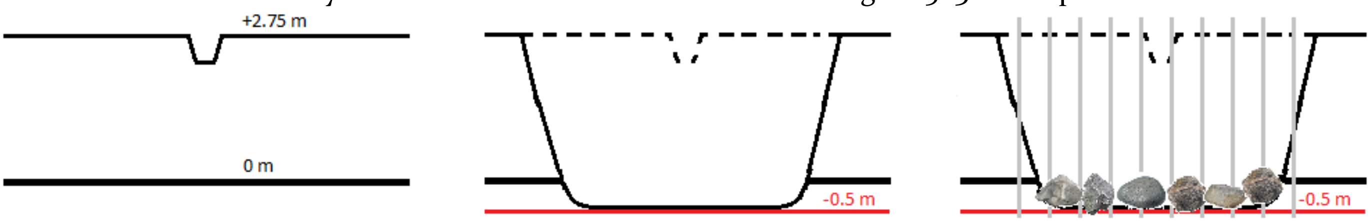

Figure 5.13: Location of the simulated emergency measure scaffold

In Figure 5.13 three front views of the dike with emergency measure scaffold are displayed. The black lines are the contours of the dike with the depression visible and the red line is the non-erodible layer of the model which represents the placement of material in the scaffold. The grey lines are the scaffold itself, holding the rocks in place at the bottom of the breach, resulting in a vertical closure. In the left image, the breach development still needs to start. The centre image is the situation when the non-erodible layer is applied. The development in depth of the breach is stopped and only horizontal erosion is possible. For visualisation purposes the right image is composed. The scaffold is in this image visible and a better idea about the simulation of the emergency measure can be made.

Analysis

Bathymetry

Figure 5.14 displays the development of the breach, including the implementation of the emergency measure as shown in Figure 5.13. Six snapshots are taken from the process, at t = 0, 8, 15, 20, 40 and 65 minutes. The breach starts as a normal breach, developing by cutting through the dike body, as described in Chapter 3.2.2. The cutting process continues until it runs into the non-erodible layer, as simulation of the implementation of the scaffold. If it runs into the layer, vertical erosion is not possible anymore and the breach starts widening. This can go on until the equilibrium situation is reached. Behind the scaffold a scour pit forms. The non-erodible layer is always stable, so no matter how large the flow velocity is, it will stay in its original position. This is also true for the scour around the scaffold what makes it unstable. No matter how large the scour is, the non-erodible layer will stay in its original position. Also, the placement of the scaffold is not taken into account. These things are discussed in more detail in the discussion part of this paragraph this paragraph.

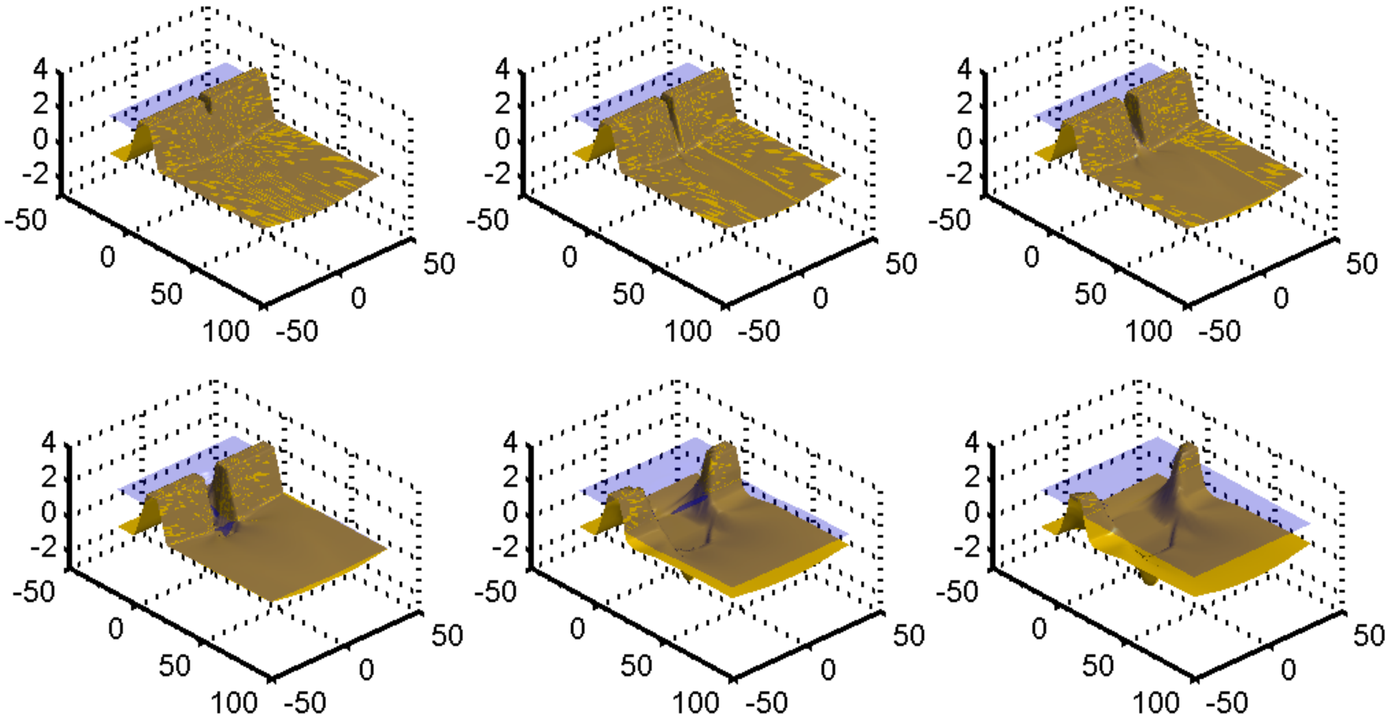

Figure 5.14: Effect of a scaffold in developing breach, snapshots taken at t = 0, 8, 15, 20, 40 and 65 minutes

Breach dimensions, water level and velocities

Figure 5.15 displays the water levels, breach width and depth and flow velocities. In the upper left plot the water levels at the upstream location, in the breach itself and at the downstream location are shown. It is visible that the polder water level is raising less quick in the beginning. The width of the breach is displayed in the left centre plot, the depth of the breach in the lower left plot. The width of the breach is developing by steps. This is the case because the avalanching mechanism makes the slopes collapse. Breach development in depth is clearly limited. For that reason the development in width is larger. Upper right the velocities upstream, in the breach itself and downstream are plotted. Top velocity in the breach is somewhat more than 5 m/s. This is lower than the velocity in the Big Bag case. The lower right plot displays the transverse velocity (measured at the location indicated with number 4 in Figure 5.3). The velocities are measured at t = 35 and t = 50 minutes.

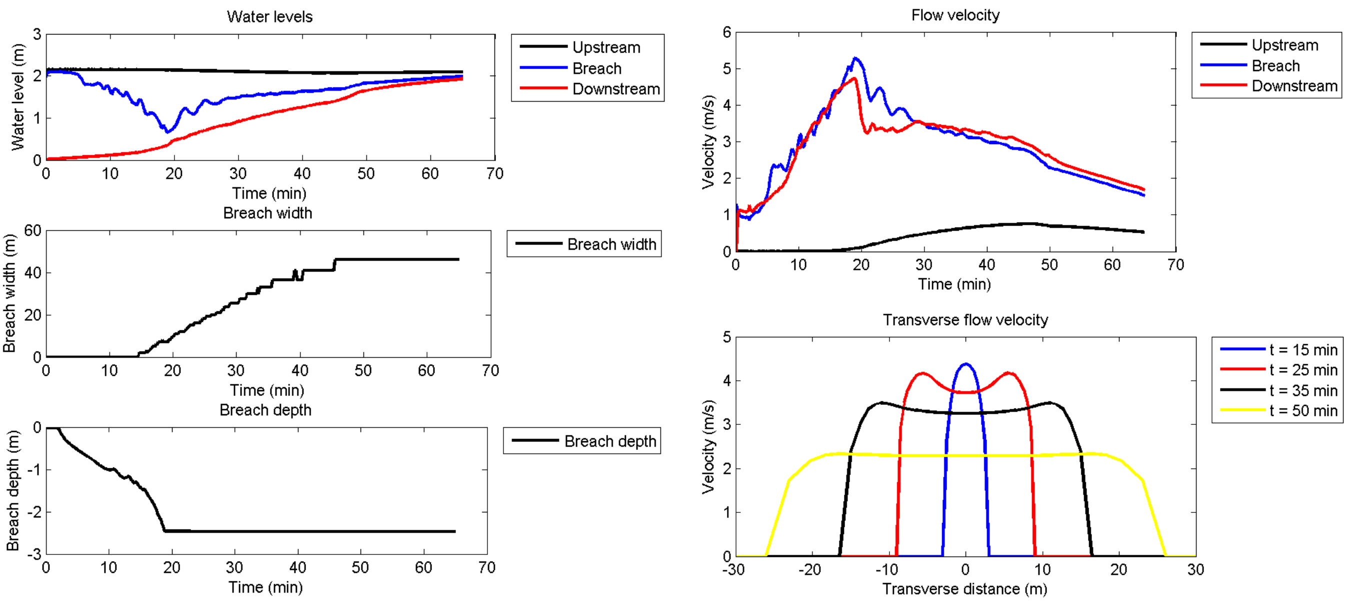

Figure 5.15: Water levels, breach dimensions and flow velocities of a breach with emergency measure scaffold

Comparison to 'do nothing' scenario

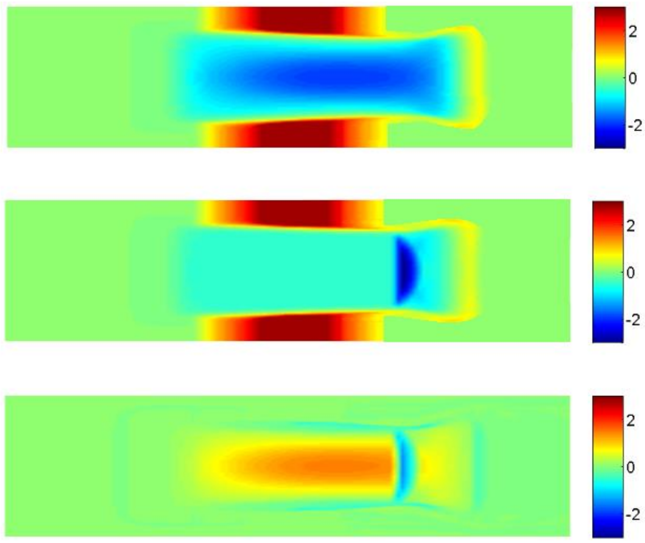

Figure 5.16: Elevation map of 'do nothing' case, scaffold case and the differences in bed level in meters at 65 minutes

If the effect of the implementation of a scaffold is compared to the scenario where no emergency measures are applied, the effect is visible. In Figure 5.16, the upper plot is the elevation map of the 'do nothing' case, the middle one is the elevation map of the scaffold case. Both are taken at t = 65 minutes. In the lower plot these two situations are subtracted: the situation where the scaffold is implemented is subtracted from the case without emergency measures. The red areas are spots where the bed level is higher in the case of the scaffold than in the case without emergency measures, the opposite holds for the blue areas. In the case without emergency measures, a scour pit develops as well, however, in the lower plot of Figure 5.16 the additional depth of the scour hole when implementing a scaffold is shown. It is clearly visible that the implementation of a scaffold prevents deepening of the breach and results in a wider scour hole. It is also visible that the scour hole appears behind the scaffold. In the lower plot the red colour is predominating. This means that the simulated scaffold induces less erosion. The part of the scour hole downstream is smaller but worse with the implementation of a scaffold as can be seen by the blue area, meaning a lower bed level. This scour hole could threat the stability of the scaffold.

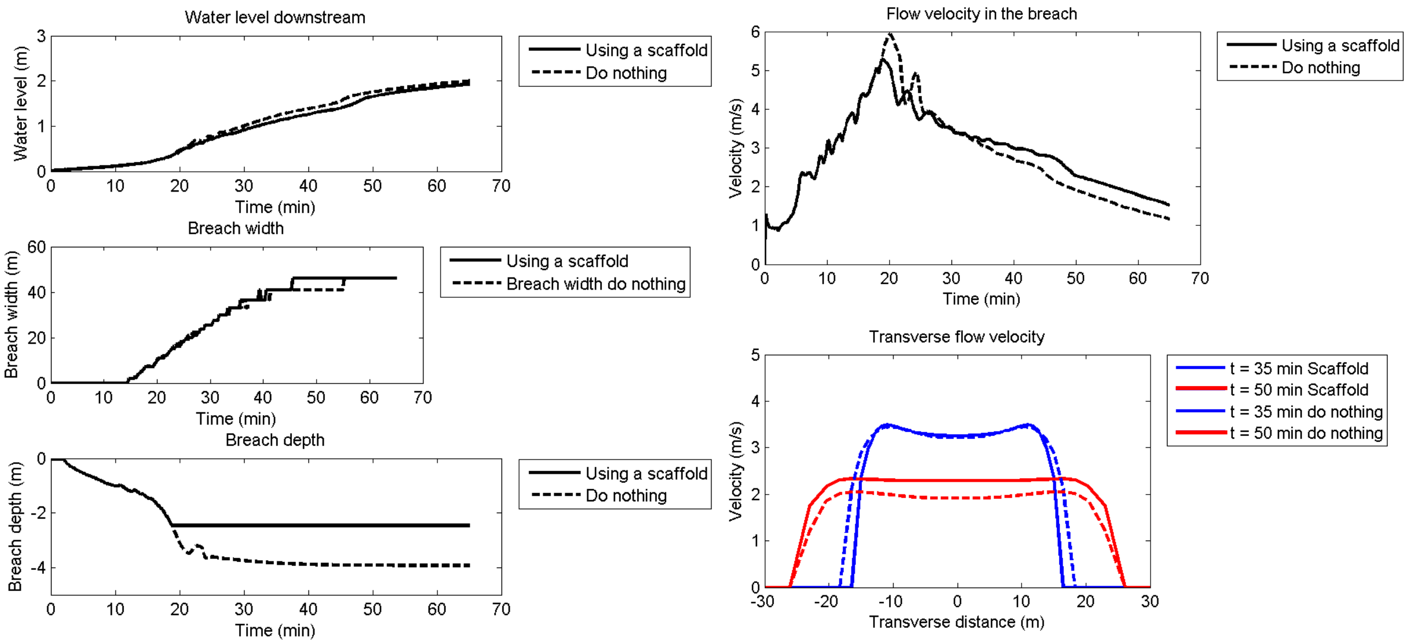

Figure 5.17: Differences water levels, flow velocities and breach dimensions Big Bags and 'do nothing' case

Figure 5.17 displays the same graphs as Figure 5.15, however, the results of the 'do nothing' case are included this time. Regarding the water level, the implementation of a scaffold results in a lower water level downstream of the breach. The flow velocity in the breach is lower, however, stays at that level for a longer time. For this reason the velocity at the end is higher than in the 'do nothing' case. This is the case because the hydraulic head is still present if a scaffold is applied. The breach width develops slower, but becomes in the end larger than in the 'do nothing' case. This is because the discharge is smaller in the beginning but because of the preventing of vertical erosion, the erosion takes place horizontal. The transverse velocity is higher, because of the presence of the larger hydraulic head.

<u>Applicability ranges</u>
In the model, the non-erodible layer is always stable. In reality the flow velocity and scour around the scaffold can cause instability and thus failure. Also, during the placement of the scaffold instability can occur due to the large currents. To determine the range wherein a scaffold can be applied successfully, hand calculations are done. Instability due to scour is not treated.

Due to the scaffold construction it is avoided that the smaller objects are flushed away. However, the scaffold itself still needs to be created. This method is common in China. The limitation in flow velocity during placement has turned out to be 3 m/s (Deltares, 2011).

In Figure 5.18 this critical flow velocity is plotted against the real flow velocity, where the stages are marked. A scaffold is stable to implement at stage I, II and early in stage III and later in stage V. This is true for the successful implementation of a scaffold at a depth of -0.5 m. Flow velocity is of course influenced by this emergency measure. However, the flow velocity is in the first 20 minutes exactly the same as in the 'don nothing' case, so the placement is possible.

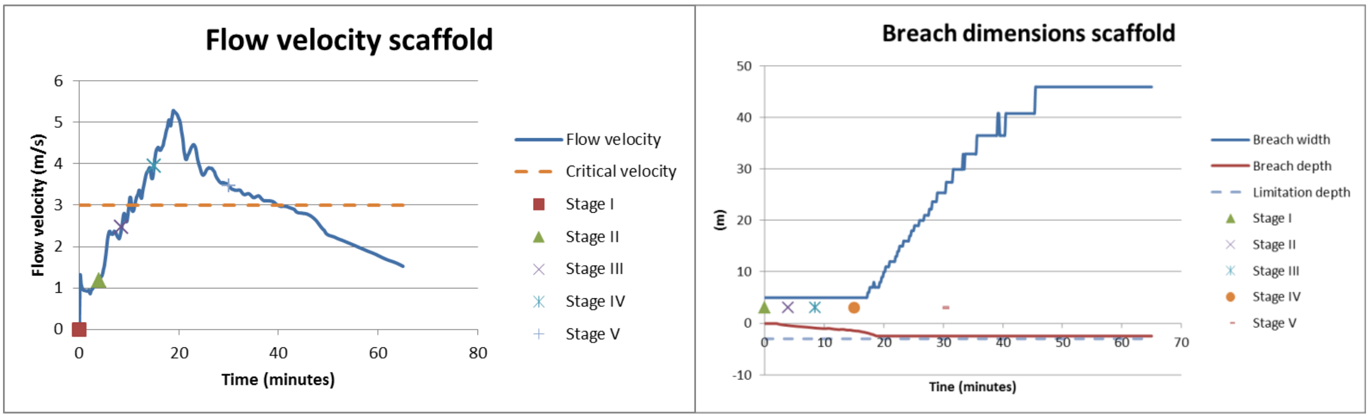

Figure 5.18: Limiting flow velocity and breach dimensions of a scaffold

The limit of the depth of the breach is 3-6 meters (Deltares, 2011). To be on the safe side the limit of 3 meter is assumed. This limit is displayed in the right plot of Figure 5.18. The maximum breach depth is located in the scour hole behind the scaffold. This scour hole is influenced by the scaffold and reaches a depth of about 2.5 meter. This means that the scaffold could be placed without problems if the breach depth could be limited to -0.5 m by successfully implementing the scaffold. This assumption has large influences. If the scaffold failed or is applied earlier or later, other flow velocities and breach dimensions would be obtained from the calculations.

Scour around the implemented scaffold is a possible failure mechanism. The poles of the scaffold are placed in fast flowing water. To get an idea about the scour depth around these poles, formula 5-2 can be used. This formula is used for scour around a cylinder (Schiereck & Verhagen, 2012). It expresses the scour depth as function of the water depth and the diameter of the cylinder. The poles of the scaffold are assumed as cylindrical and no bottom protection is placed. The formula calculates the equilibrium depth.

$$\frac{h_s}{D} = 2\tanh\frac{h_0}{D} \qquad [-] \qquad (5\text{-}2)$$

h_s is the final scour depth , D is the diameter of the cylinder and h_o is the water depth. For the scaffold a diameter of 10 cm is assumed in a water depth of 2 meter.

$$h_s = 2\tanh\frac{h_0}{D}\cdot D \qquad [-] \qquad (5\text{-}3)$$

$$h_s = 2\tanh\frac{2}{0.1}\cdot 0.1 = 0.2 \qquad [\mathrm{m}] \qquad (5\text{-}4)$$

The equilibrium depth of the scour around the poles is 0.2 m. However, floating debris enlarges effective diameter and many poles induce extra scour due to flow constriction (Schiereck & Verhagen, 2012). These comments hold especially for breach closure. This is also the scour induced by the poles only, the objects for the closure induce more scour. The real scour depth will be larger.

As example, the effect of Big Bags instead of a scaffold as vertical closure is investigated. The Big Bags are placed at the location of the scaffold at the bottom of the breach, instead of at the side slopes as was the case in Chapter 5.3.3. This way, the vertical closure is performed with Big Bags.

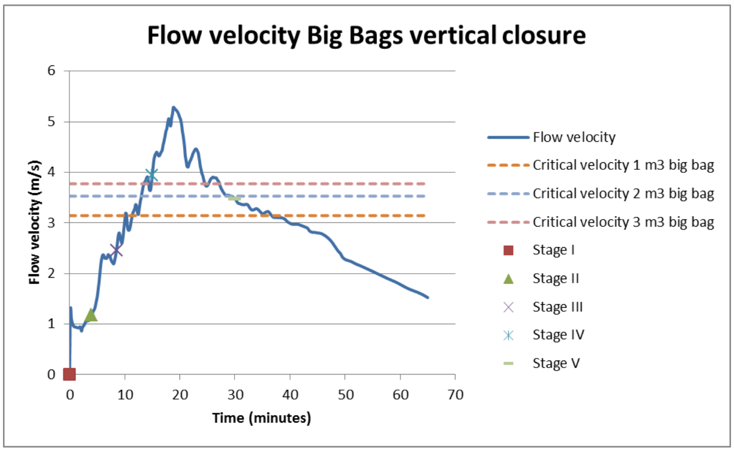

Figure 5.19: Big Bags as vertical closure

If Big Bags were applied for vertical closure, making downward erosion impossible, they would have a larger applicability range since the velocities are smaller than the velocities created by a horizontal closure.

Key findings and discussion
A scaffold is according to the calculations applicable in stages I, II and the end of stage V. In the first two stages the stability requirements for flow velocity and breach dimensions are met. For dikes constructed completely out of sand (these dikes do not exist, however, for the research purpose this is assumed) the scaffold need to be placed within 15 minutes to be effective. Of course it is impossible to construct a scaffold in a breach in this little amount of time. Despite the impossible time limit, the effects of the scaffold as emergency measure are clearly visible. The water levels, velocities and breach dimensions are influenced by this emergency measure.

The calculations in this paragraph are done with the successful implementation of a scaffold at -0.5 m. If this was not the case, the results would look different. The influence of the scaffold on the flow velocities and the breach dimensions is large. Aspects as failure of the placement of the scaffold, a later placement or an earlier placement are not considered in this way. This is a subject for further research.

If the closure was attempted with Big Bags instead of a scaffold, the same effect could be observed. The applicability range is in the same order of magnitude. With Big Bags of 2 m^3 even a larger applicability range could be obtained. The vertical closure technique is preferable above the horizontal closure technique. The velocity and water levels are smaller in case of a vertical closure. Also, the erosion in vertical direction is less worse than in horizontal direction.

5.3.5 Ships and barges

Starting point
The emergency measure of ships and barges is modelled as a large element lying in front of the breach from t = 20 minutes on. In the model, the vessel is modelled as a non-erodible layer in front of the breach. This creates a vessel in front of the breach and is supposed to reduce the discharge through the breach. The dimensions of the vessel are 3 m x 40 m, comparable to a real vessel. The height is the same as the height of the crest of the dike; 2.75 m. The first 20 minutes are non-interfered breach development and are the same as in the 'do nothing' case. There is chosen for 20 minutes because there is time needed to sail in

the vessel. To illustrate the location of the non-erodible layer and thus the location of the vessel Figure 5.20 is composed.

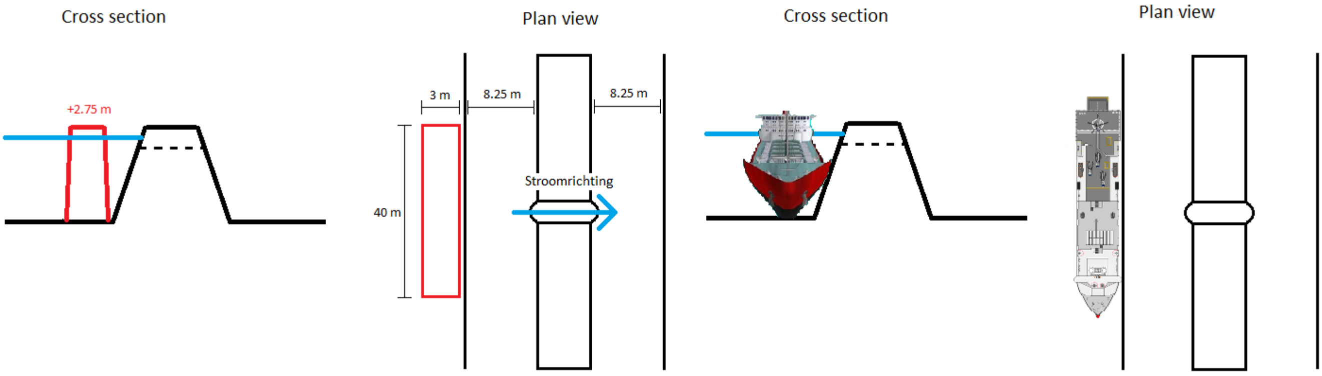

Figure 5.20: Location of simulated emergency measure vessel, side view and top view

In Figure 5.20 a side view (left) and a top view (second left) of the dike with a sunken down vessel as emergency measure are displayed. The black lines are the contours of the dike with the depression visible and the red line is the non-erodible layer of the model which represents the sunken down vessel in front of the breach. Both plots are taken before the breach development has started. For visualisation purposes the two right images are composed. The vessel is in this image visible and a better idea about the simulation of the emergency measure can be made.

Analysis

<u>Bathymetry</u>

Figure 5.21 displays the development of the breach, including the implementation of the emergency measure of sinking down a vessel in front of the breach at t = 20 minutes as shown in Figure 5.20.

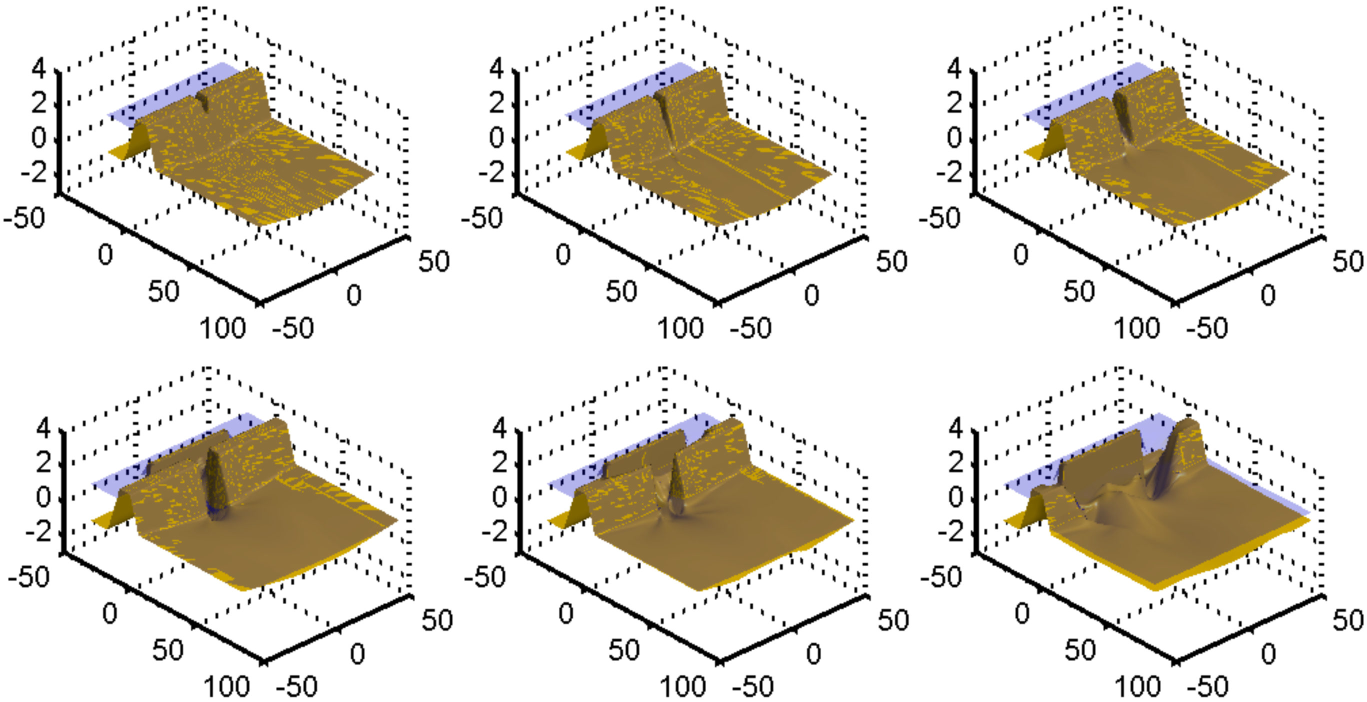

Figure 5.21: Effect of a sunken down vessel in front of a developing breach, snapshots taken at t = 0, 8, 15, 20, 40 and 65 minutes

Six snapshots are taken from the process, at t = 0, 8, 15, 20, 40 and 65 minutes. The breach starts as a normal breach, developing by cutting through the dike body first and then expanding parallel to the crest, as described in Chapter 3.2.2. When the vessel is simulated at t = 20 minutes, the discharge through the breach slows down. However, by the changed flow pattern, the water scours parallel between the dike body and the vessel. The width of the dike base is reduced behind the vessel. Suddenly the breach width increases rapidly due to the quick erosion of the small dike body. The scour hole downstream of the vessel seems limited and forms at the locations where the water flows between the vessel and the dike head. The non-erodible layer will stay in its original position and scour around the vessel will have no effect on its stability in the model. Also, the placement of the vessel is not taken into account. These things are discussed in more detail later this paragraph.

Breach dimensions, water levels and velocities

Figure 5.22 displays the water levels, breach width and depth and flow velocities. In the upper left plot the water levels at the upstream location, in the breach itself and at the downstream location are shown. Until t = 20 minutes, the water level is the same as the 'do nothing' case. The width of the breach is displayed in the left centre plot, the depth of the breach in the lower left plot. The width develops slower but the growth increases rapidly. This is because first the dike body is reduced in width due to the erosion of the dike behind the ship. The breach width stays the same, but the crest width reduces. If the dike body is weakened the breach width is able to grow fast. The water is leaded along the dike heads by the vessel and the erosion at these dike heads is larger. After the implementation of the vessel, the breach depth reduces. This is however due to the measurement location. The measurement is done at the same point of measurement as the water level in the breach. Sediment is stirred towards this location behind and thus sheltered from the currents by the vessel. Later in time the breach depth increases again due to the changed velocity profile (see lower right plot). The drop in velocity seen in the upper right plot is caused by the vessel. The velocity is measured behind the vessel and is thus lower than in a non-interfered situation. The negative velocities at the end of the simulation are probably caused by eddies. Further investigation is needed. The lower right plot displays the transverse velocities.

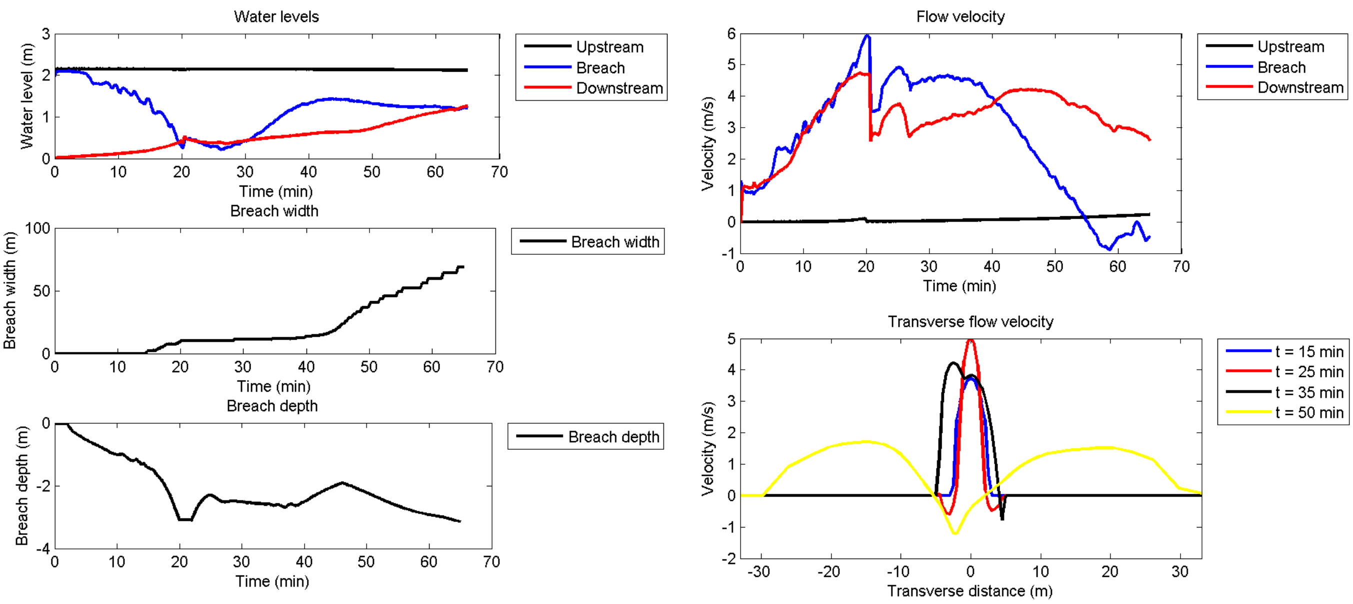

Figure 5.22: Water levels, breach dimensions and flow velocities of a breach with emergency measure vessel

<u>Comparison to 'do nothing'</u>

If the effect of the implementation of a vessel is compared to the scenario where no emergency measures are applied, the effect is visible. In Figure 5.23 , the upper plot is the elevation map of the 'do nothing' case, the middle one is the elevation map of the vessel case. Both are taken at t = 65 minutes. In the lower plot these two situations are subtracted: the situation where a vessel is implemented is subtracted from the case without emergency measures. The red areas are spots where the bed level is higher in the case of the vessel than in the case without emergency measures, the opposite holds for the blue areas. The vessel reduces the scour pit behind the dike. However, a more wider breach is formed. The reduction in scour depth can be seen by the red area in the lower plot. The increase in width is visible by the blue strips at the sides of the breach.

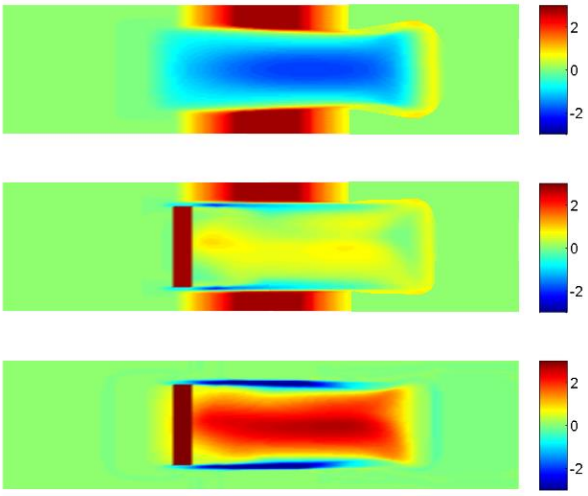

Figure 5.23: Elevation map of 'do nothing' case, vessel case and the differences in bed level in meters at 65 minutes

Figure 5.24 displays the same graphs as Figure 5.22, however, the results of the 'do nothing' case are included this time. Regarding the water level, the implementation of a vessel results in a lower water level downstream of the breach. The breach width develops faster and becomes larger than in the 'do nothing' case. This is because the flow is forced along the dike heads, where thus more erosion takes place.

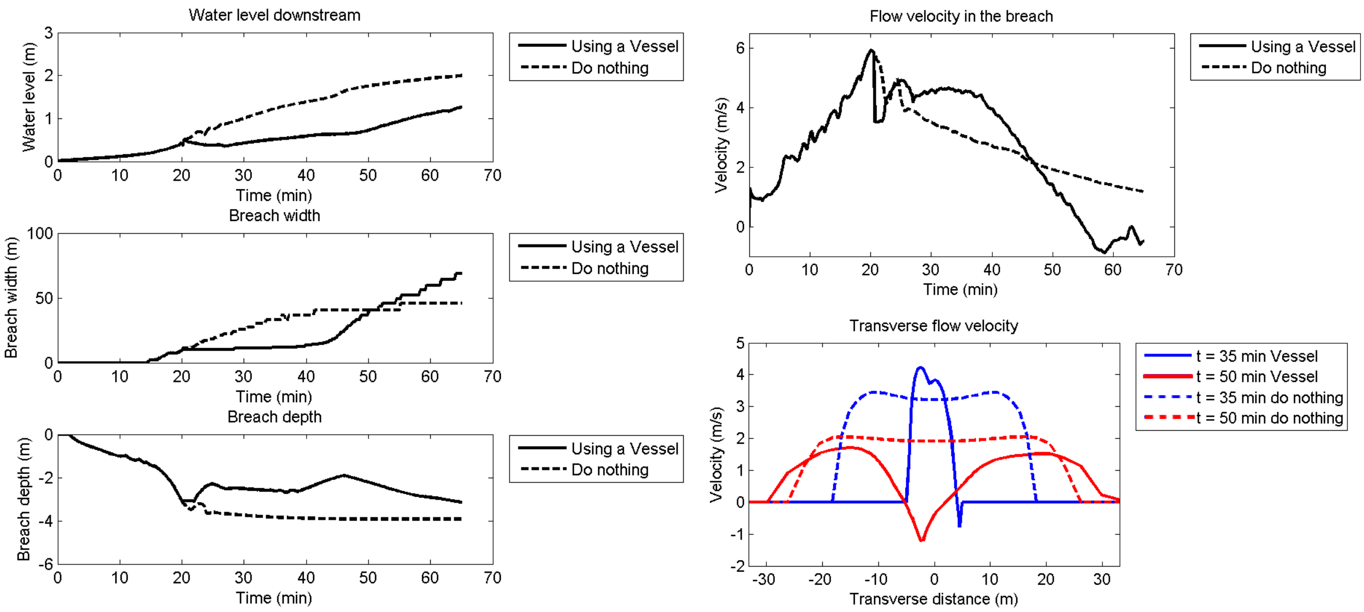

Figure 5.24: Differences water levels, flow velocities and breach dimensions vessel and 'do nothing' case

<u>Applicability ranges</u>
In reality the flow velocity and scour around the vessel can cause instability and thus failure. Also, during the sinking down of the vessel instability can occur. Difficulties with the placement of the vessel due to the large currents can occur. To determine the range wherein a vessel can be applied successfully, calculations are done. Instability due to scour and during the sinking down is not treated.

For the calculation, the same approach as with Big Bags is followed. It is however questionable if the Izbash formula can be applied to objects the size of a ship. For indicative purposes it is done in this thesis. Calculations are done after the vessel is sunk down and lies on the bottom. The most normative condition is just after the vessel is sunk down. The inside of the vessel is filled with water and not yet filled with fill material, the density of the vessel as a whole is therefore assumed at 1100 kg/m³. In Figure 5.25 the calculated velocity in the breach from Figure 5.24 is displayed. Along this velocity curve, the breach stages I till V are marked. These are determined by comparing the developing breach to the breach stages discussed in Chapter 3.2.2. The critical velocity of the vessel is displayed as well. If the flow velocity is below this critical level, the vessel is stable and can be applied successfully. This critical velocity is calculated with the Izbash formula. Izbash is applicable since it considers forces on individual 'stones'.

$$u_c = 1.2 \cdot \sqrt{2\Delta g d} \qquad [\text{m/s}] \qquad (5\text{-}5)$$

Table 5.2: Calculation critical flow velocity of the vessel

Size vessel	d_n [m]	u_c [m/s]
40 x 3 x 2.75 m	6.9	4.4

As can be seen in Figure 5.25, the critical velocity of the vessel is 4.4 m/s. The vessel is not stable during a short period of time. Implementation of a vessel in stage IV is not useful since the large flow velocity will flush it away.

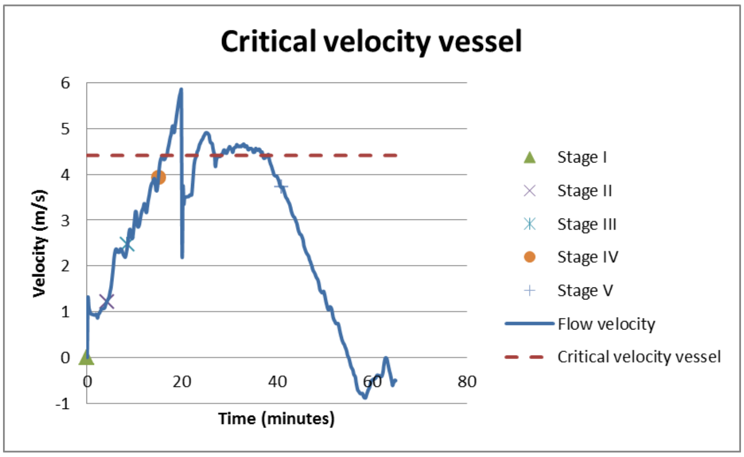

Figure 5.25: Critical velocity vessel

Key findings and discussion
A vessel is according to the calculations applicable in all stages with exception of stage IV. In the first three and the last stage the stability requirements for flow velocity and breach dimensions are met. However, the available time is probably not enough to logistically bring the vessel to the breach location. Despite the impossible time limit, the effects of the

vessel as emergency measure are clearly visible. The water levels, velocities and breach dimensions are influenced by this emergency measure. The vessel reduces the discharge in first instance by slowing down the breach development in width. However, after some time the breach development in width increases fast due to the water that flows in between the vessel and the dike. Additional measures are needed to close the breach if a vessel is implemented.

The calculations in this paragraph are done with the successful implementation of a vessel in front of the breach. If this was not the case, the results would look different. The influence of the vessel on the flow velocities and the breach dimensions is large. Aspects as failure of the placement of the vessel, a later placement or an earlier placement are not considered in this way. From the calculations follow negative flow velocities just downstream of the vessel. These negative velocities can be explained partly. These are subjects for further research.

5.3.6 Emergency dike

Starting point

The emergency measure of an emergency dike is modelled as a u-shaped dike in front of the breach from t = 20 minutes on. This is modelled as a non-erodible layer and creates an emergency dike in front of the breach. This location is chosen since at the inner side of the dike the scour hole and the flow velocity larger. This location is thus unfavourable to construct the emergency dam. To determine the dimensions of the emergency dike, there is searched for the place upstream where the flow velocity is lower than the critical velocity for a Big Bag; 3 m/s. From Figure 5.26, which is measured in the 'do nothing' case, follows that this distance is 25 meter.

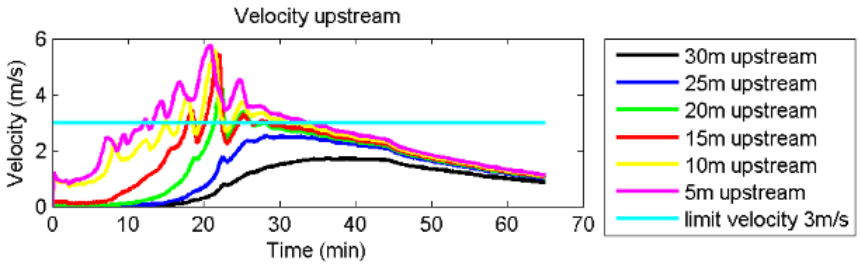

Figure 5.26: Velocity at several distances upstream of the breach

The height of a Big Bag is assumed at 1 m. The length of the emergency dike parallel along the dike is 40 m, 20 m to both side of the breach because the flow velocity is at those locations smaller and therefore not the 25 m length is needed. The emergency dike is present in front of the breach at t = 20 minutes. The first 20 minutes are non-interfered breach development and are the same as in the 'do nothing' case. There is chosen for 20 minutes because there is time needed to place the emergency dike. To illustrate the location of the non-erodible layer and thus the location of the emergency dike Figure 5.27 is composed.

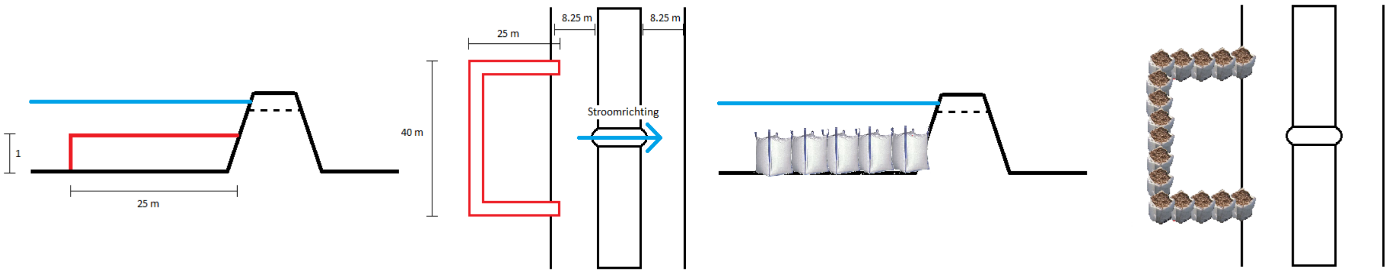

Figure 5.27: Schematization and location of simulated emergency measure vessel, side view and top view

Analysis

Bathymetry

Figure 5.28 displays the development of the breach, including the implementation of the emergency measure dike in front of the breach at t = 20 minutes. Six snapshots are taken from the process, at t = 0, 8, 15, 20, 40 and 65 minutes. The breach starts as a normal breach, developing by cutting through the dike body first and then expanding parallel to the crest.

When the emergency dike is simulated at t = 20 minutes, the discharge through the breach slows down. The emergency dike acts as a weir and reduces the breach development. This is clearly visible in the three bottom plots, where the breach grows slowly. In the model, no matter how large the scour is, the non-erodible layer will stay in its original position. Instability due to scour is not taken into account. Also, the placement of the emergency dike is not taken into account. These things are discussed in more detail later this paragraph.

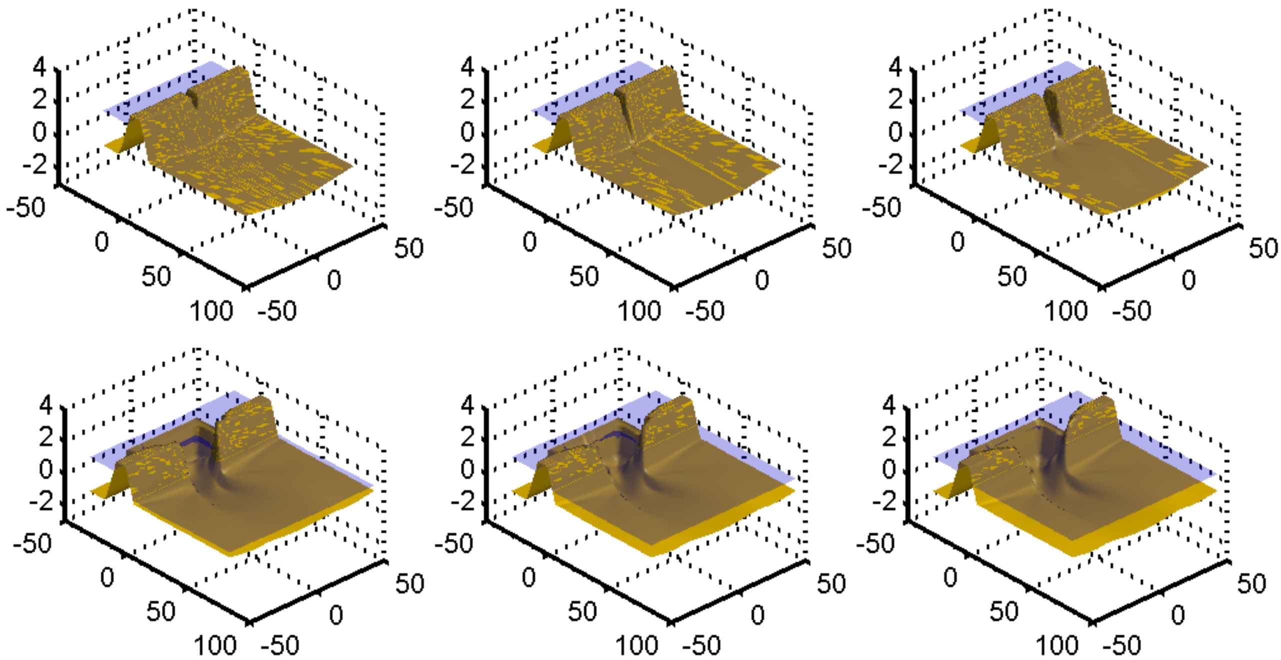

Figure 5.28: Effect of an emergency dike in front of a developing breach, snapshots taken at t = 0, 8, 15, 20, 40 and 65 minutes

Breach dimensions, water levels and velocities

In Figure 5.29, the water level and flow velocities upstream of the breach are measured at the location of the emergency dike. This figure displays the water levels, breach dimensions and flow velocities. In the upper left plot the water levels at the upstream location, in the breach itself and at the downstream location are shown. Until t = 20 minutes, the water level is the same as the 'do nothing' case. Thereafter a drop in the upstream water level is noticeable due to the implementation of the emergency dike, which acts as a weir. The width of the breach is displayed in the left centre plot, the depth of the breach in the lower left plot. The breach width is developing slower from 20 minutes on. The depth of the breach remains at a more or less constant level after 20 minutes. The drop in the downstream and breach velocity in the upper right plot is caused by the emergency dike. The rapid increase in upstream flow velocity is caused by the emergency dike too.

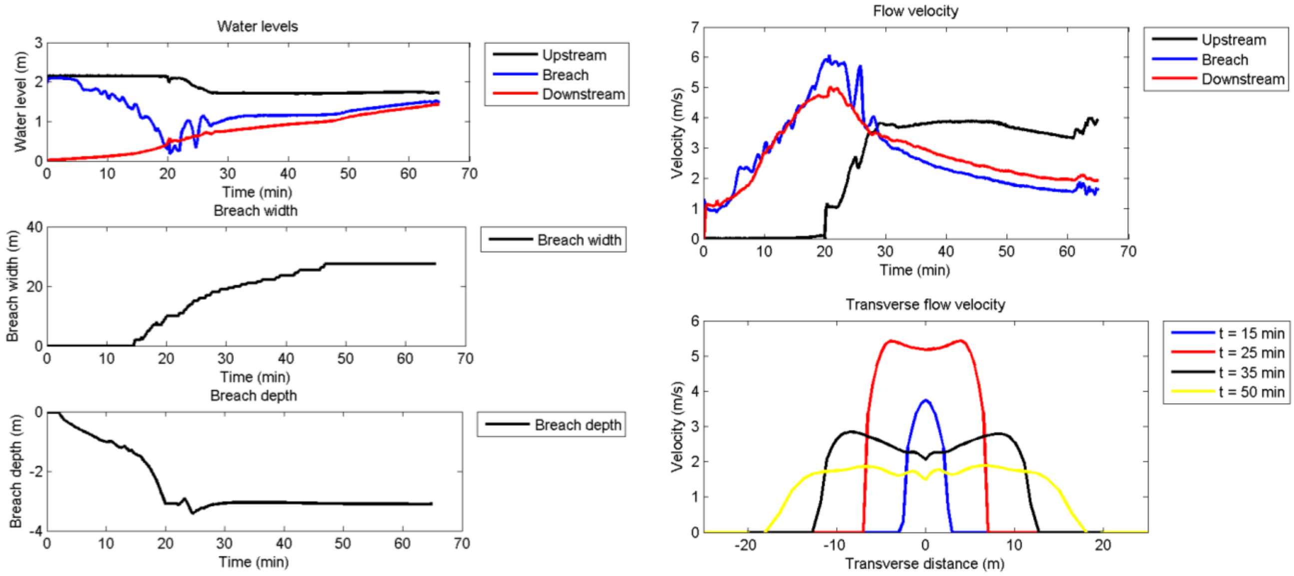

Figure 5.29: Water levels, breach dimensions and flow velocities of a breach with an emergency dike

<u>Comparison to 'do nothing'</u>

If the effect of the implementation of an emergency dike is compared to the scenario where no emergency measures are applied, the effect is visible. In Figure 5.30, the upper plot is the elevation map of the 'do nothing' case, the middle one is the elevation map of the emergency dike case. Both are taken at t = 65 minutes. In the lower plot these two situations are subtracted: the situation where a vessel is implemented is subtracted from the case without emergency measures. The red areas are spots where the bed level is higher in the case of the emergency dike than in the case without emergency measures, the opposite holds for the blue areas. The emergency dike reduces the scour pit behind the dike. The breach width stays within the emergency dike and is also smaller than the 'do nothing' case. The decrease in width is visible by the red strips at the sides of the breach.

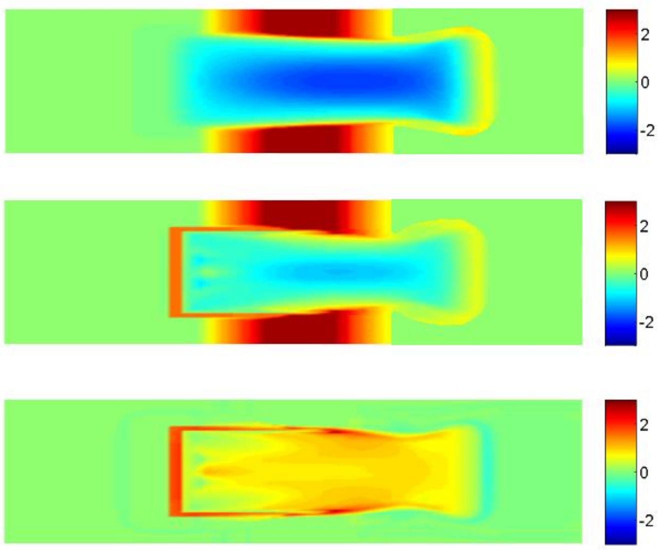

Figure 5.30: Elevation map of 'do nothing' case, emergency dike case and the differences in bed level in meters at 65 minutes

Figure 5.31 displays the same graphs as Figure 5.29, however, the results of the 'do nothing' case are included this time. Regarding the water level, the implementation of an emergency dike results in a lower water level downstream of the breach. Breach dimensions are reduced too, after implementation of the emergency dike they develop slower. The upper right plot displays the velocity at the location of the emergency dike. This velocity increases due to the emegency dike. Transverse velocities in the breach are displayed in the lower right plot. The velocities in case of the emergency dike are in both times lower and active over a smaller distance.

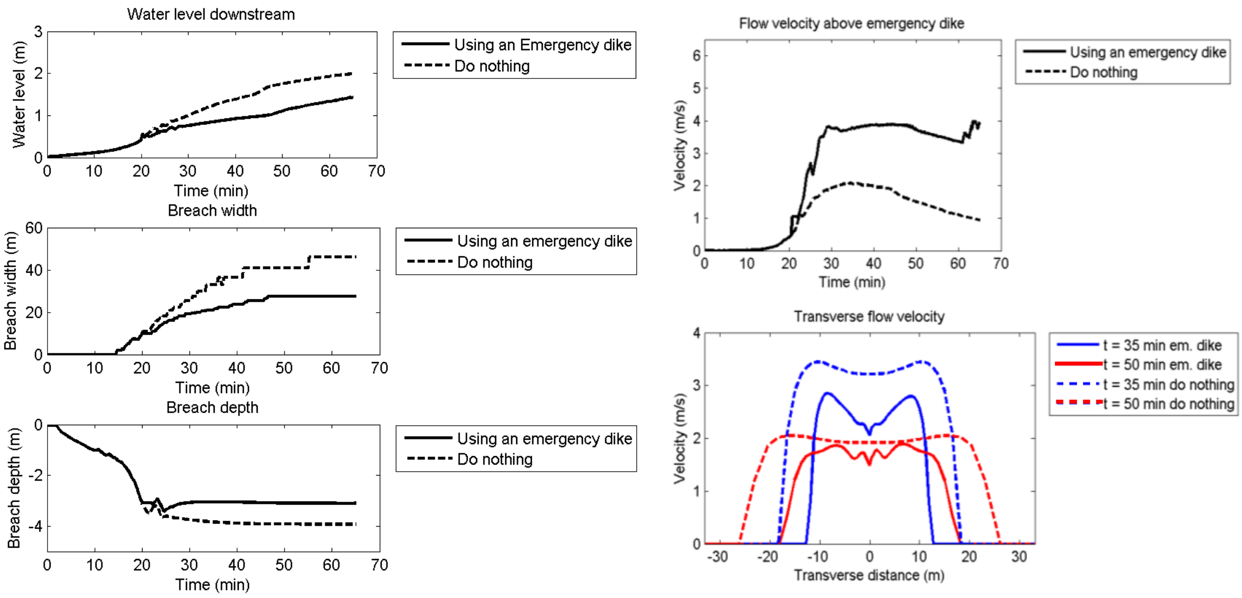

Figure 5.31: Differences water levels, flow velocities and breach dimensions emergency dike and 'do nothing' case

<u>Applicability ranges</u>
The emergency dike is assumed to be constructed out of Big Bags. The stability of these Big Bags needs to be checked. Flow velocity of the water and scour around the Big Bag can cause instability and subsequently washing away of the Big Bag and thus failure. Also, during the placement of the Big Bags instability can occur. To determine the range wherein Big Bags can be applied successfully, calculations are done. Instability due to scour is not treated.

In Figure 5.32 the velocity at the location of the upstream emergency dike is displayed. The critical velocities of different sized Big Bags are displayed as well. If the flow velocity is below this critical level, Big Bags are stable and can be successful applied. This velocity is calculated with the Izbash formula. Izbash is applicable since it considers forces on individual 'stones'.

$$u_c = 1{,}2 \cdot \sqrt{2\Delta g d} \qquad [\text{m/s}] \qquad (5\text{-}6)$$

Table 5.3: Calculation critical flow velocity of Big Bags

Size big bag	d_n [m]	u_c [m/s]
$1\ m^3$ (1350 kg)	1.0	3.1
$2\ m^3$ (2700 kg)	1.26	3.5
$3\ m^3$ (4050 kg)	1.44	3.8
$4\ m^3$ (5400 kg)	1.58	4.0

As can be seen in Figure 5.32, the Big Bags of sizes 1, 2 and 3 m^3 are not stable due to the flow velocity. Big Bags of 4 m^3 are stable in the flow velocity. Big Bags of 4 m^3 may be manufactured by tying two Big Bags of 2 m^3 together.

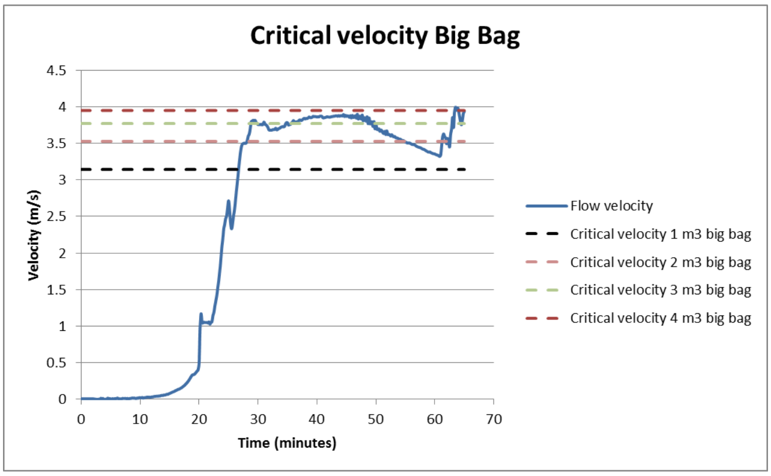

Figure 5.32: Critical velocity emergency dike

Key findings and discussion

Closure by means of an emergency dike seems promising. An advantage is that the breach is not closed on the location of the breach itself, but upstream. This reduces the velocities and the conditions to implement the measure are better. Big Bags of 4 m^3 are needed for a stable implementation. The instability due to scour is not taken into account which probably is an important failure mechanism. Also, the way of getting the Big Bags into the desired position is not taken into account.

The emergency dike is implemented after 20 minutes. If it is possible to start earlier with placing the Big Bags, maybe an even better result can be obtained. Later implementation of the Big Bags is a topic for further research too. The location is chosen at the upstream side of the breach since the flow velocity and scour hole are larger at the downstream side of the breach and thus more unfavourable. Recovered old dike breaches which are closed with this strategy, prove to have the emergency dike at the downstream side. This is probably due to the easy accessibility of the downstream side compared to the upstream side and the eroded soil that is deposited at the edge of the downstream scour hole making an easy start for a new dike. Furthermore, if the dike is recovered at the downstream side, no constriction in the river is made.

The effects of the emergency dike as emergency measure are clearly visible. The water levels, velocities and breach dimensions are influenced by this emergency measure. Calculations in this paragraph are done with the successful implementation of an emergency dike upstream of the breach. If this was not the case, the results would look differently. Aspects as failure of the placement of the emergency dike, a later placement or an earlier placement are not considered in this way. The emergency dike could also be constructed in different shapes, for example a semicircle. These are subjects for further research.

5.3.7 Comparison

To determine the most effective closure method, the above discussed methods are compared. These comparisons are made on flow velocities, water levels and breach dimensions, see Figure 5.33. For explanations of the shape of the curves, reference is made to the paragraph where the associated emergency measure is discussed.

Note that in the lower right plot for the emergency dike the velocity at the location of the emergency dike is plotted. For the other measures the flow velocity in the breach is displayed.

The closure with an emergency dike is selected for a complete closure. This method has the best performance in breach width and second best in breach depth, only beaten by the scaffold where the breach depth is kept constant. The lowest polder water level is obtained by the vessel, but this measure is not capable of stopping the breach. The flow velocity at the place of the emergency dike is not the lowest, but has the lowest peak. This peak velocity is important because it is governing.

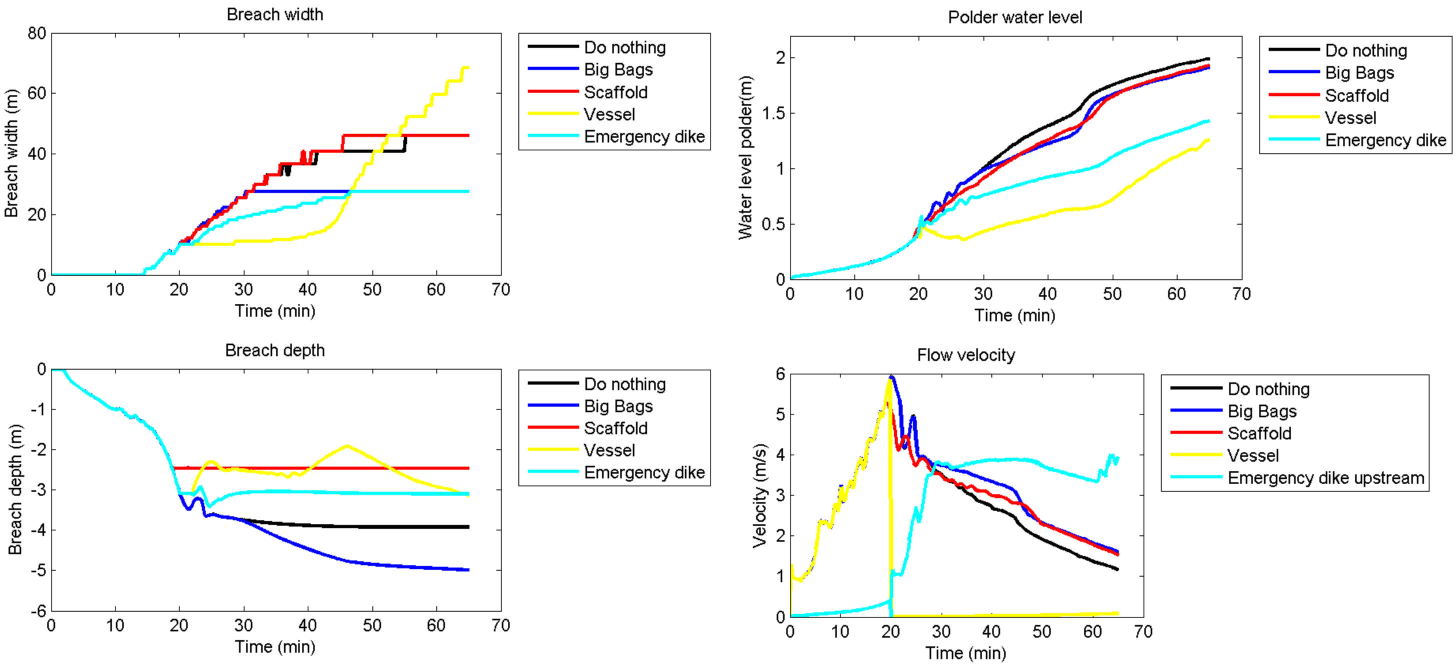

Figure 5.33: Comparison of discussed closure methods

5.3.8 Complete closure

In this paragraph the effect of a complete and stepwise closure of a dike breach in a sandy dike for the most promising measure is investigated; an emergency dike. The effect of the hydraulics and morphology on the stability of the implemented layers of the emergency measures is checked. Furthermore, calculations are made regarding the stability of the measure after closure of the breach.

To simulate a complete closure, several phases need to be modelled. As was explained in Chapter 5.2, it is not possible to let the emergency measure evolve in time during a model run. A solution for this problem is to take the output of a simulation, adjust the non-erodible layer to simulate a next phase of an emergency measure and make the new bathymetry, the water levels and a non-erodible layer the input for the next run.

Two strategies of placement of the emergency dike with Big Bags were tested; strategy A and B, for details of these strategies see Appendix IV. To simulate subsequent closure steps, the bathymetry and water level at the end of phase n are used as input for phase n+1. For example, the bathymetry and water level at the end of phase 0 (t = 20 minutes) are taken as input for the simulation of phase 1. At the beginning of every phase, the non-erodible layer representing the emergency dike is changed to simulate the next step in closure.

Strategy A has proved to be more effective, see Figure 5.34. In this strategy first the emergency dike perpendicular to the real dike and in the next phase parallel to the dike is constructed up to 1 m above ground level. Next, this procedure is repeated to above the water level. Strategy B starts with the emergency dike parallel to the dike and next perpendicular to the dike. Next, this procedure is repeated to above the water level.

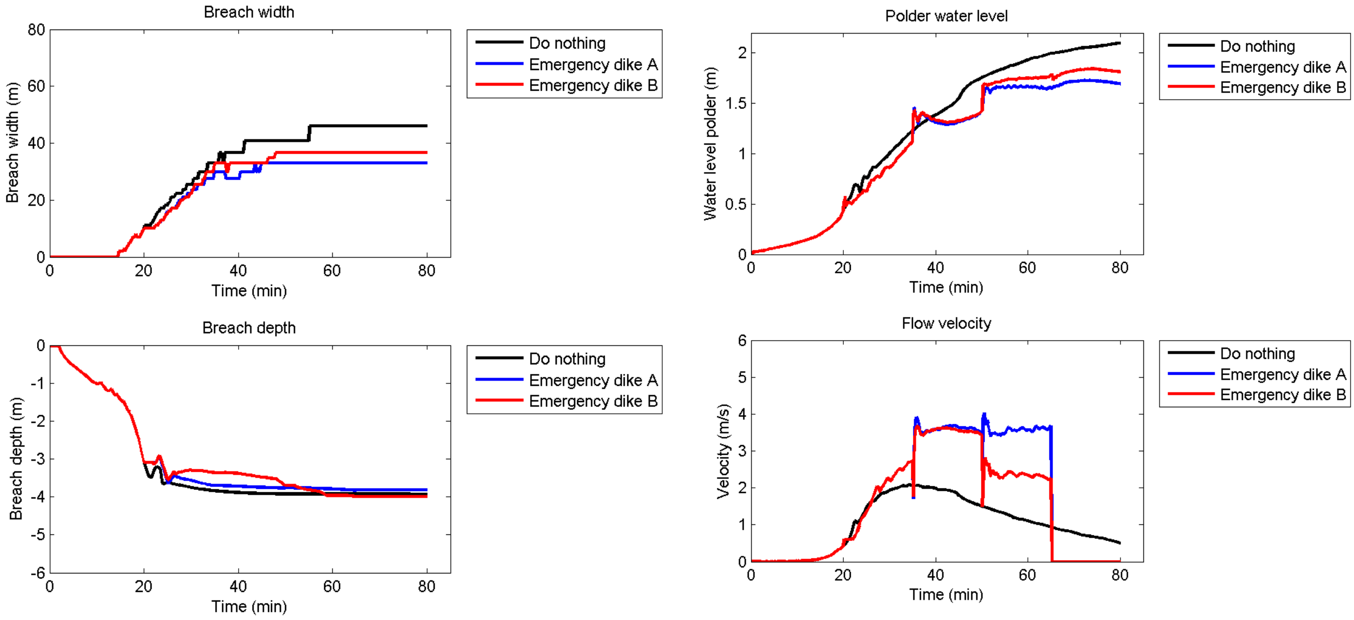

Figure 5.34: Comparison of complete closure with strategy B to 'do nothing'

As can be seen, strategy A has a more favourable effect on the breach dimensions and the water level. The velocity for Strategy B in stage 4 is lower because it is measured just next to the emergency dike.

Calculations

<u>Dynamic</u>

As can be seen the flow velocity that occurs during the closure reaches a maximum value of about 4 m/s. In Chapter 5.3.6 is shown that a single Big Bag is not able to withstand this large flow velocity. Big Bags of 4 m^3 are the smallest size of Big Bags that remain stable, see Figure 5.32. There is assumed for the emergency dike that four Big Bags of 1 m^3 are tied together and form a 4 m^3 Big Bag. Scour around the Big Bags and the actual placement of the Big Bags are not treated.

<u>Static</u>

Now the breach is closed, other stability issues may be a problem. The placed Big Bags which are supposed to hold the water out of the polder area can fail due to:

1) Shearing
2) Rotation
3) Piping
4) Failure of the subsoil

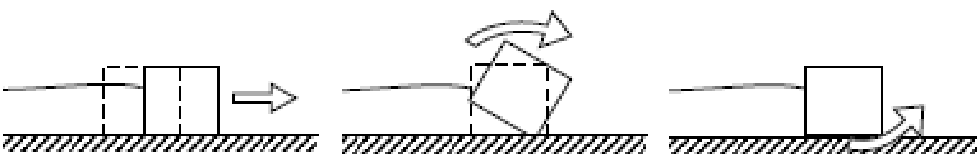

Figure 5.35: Failure mechanisms shearing, rotation and piping, (Boon, 2007)

Ad. 1)
The emergency measure is stable with respect to shearing if the resisting friction force T is larger than the hydrostatic force of the water Fh. The friction force is calculated by multiplying the weight of the structure by a shear factor.

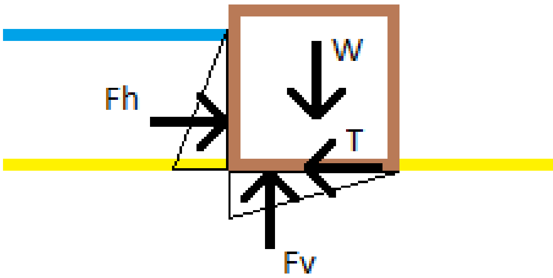

Figure 5.36: Forces on the emergency dike

Since one Big Bag does not have enough height to counter the water, there is assumed that two Big Bags are placed on top of each other. From stability calculations in AppendixV, follows that two Big Bags on top of each other are not stable. The next calculation in this appendix is done for two rows of two Big Bags on top of each other. These two rows of two Big Bags on top of each other are not stable either according to the calculations. Three rows of two Big Bags on top of each other are stable.

Ad. 2)
The emergency dike now consists of six Big Bags. Because of the multiple elements in this emergency dike, rotation is not applicable and is not treated here.

Ad. 3)
Piping is a problem for emergency measures, as was already investigated in the case studies of Chapter 4. Because the subsoil is assumed to be sand, piping will form a threat. The safety coefficient is for Bligh as well as Lane is 0.1, see Appendix V. Immediately after the closure additional measures regarding piping need to be applied.

Ad. 4)
Because the relatively small weight of the structure failure of the subsoil is not considered (Boon, 2007).

Discussion
The closure with an emergency dike according to Strategy A is favourable. This results in a lower polder water level and smaller breach dimensions. However, this is only checked for one situation. The sensitivity for later or earlier implementation of the measure or not being able to implement a complete layer still needs to be investigated. This is also true for several characteristics like the polder area.

The results of the separate runs were combined to one complete closure. Runs needed to be done for every layer separately and the implemented layers are quite large. For this reason XBeach generates the jumps in the plots of the water level and the velocity. This can be improved by a dynamic non-erodible layer in XBeach which changes in time.

After the closure of the breach the static stability forms another problem. There are 3 rows of 2 Big Bags on top of each other needed to make a stable emergency dike which will not shear. Piping is an even more serious problem. Right after the closure additional measures need to be placed to counter piping.

5.3.9 Influence of dike characteristics

Cases calculated in this chapter are all done with the same dike. Dike characteristics are changed to see their effect on breach closure. In Chapter 3.2.1 important dike characteristics are discussed. The characteristics discussed in that chapter are: type of dike, geometry, revetment, subsoil, category of the dike, flood wave and the failure mechanism. In this paragraph, the most important characteristics are varied; polder area, the moment of implementation, a berm and a varying outer water level. The effect of the different characteristics is checked for the case of a closure with an emergency dike, as calculated in Chapter 5.3.8. Simulations are done up to phase 2 of the implementation of the emergency dike from strategy A.

Polder area

To investigate the effect of the polder area, simulations with several polder sizes are done. Polders with an area of 1 km², 5 km² and 10 km² are modelled. The shape of the polder is the same, it is only lengthened perpendicular to the dike. In Figure 5.37, the results are plotted. The polder area of the 1 km² and the 1.5 km² are almost already filled up at the end of phase 2. However, for the 5 km² and 10 km² the water level has not reached the end of the polder. This means that if the polder is chosen larger than 5 km in length, any further lengthening will not have any effect on the simulation.

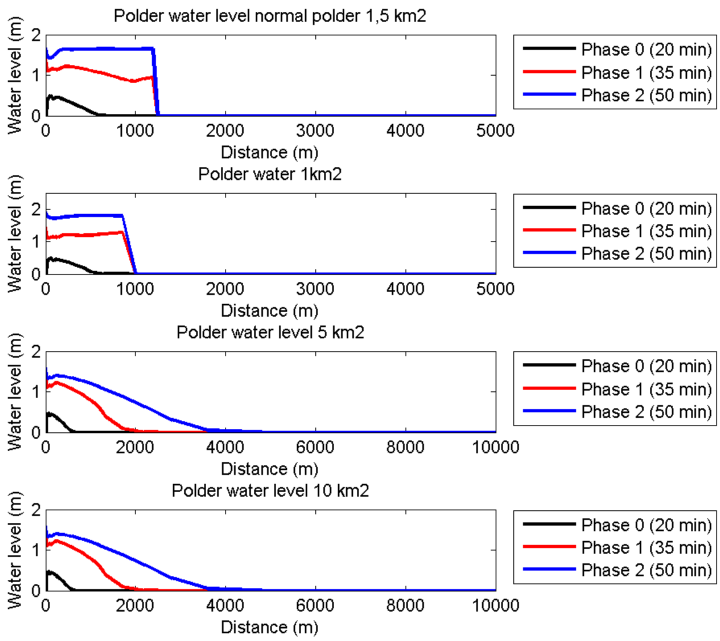

Figure 5.37: Water levels in different polder area sizes

Figure 5.38 displays the polder water levels per phase. There can be observed that the polder water levels in phase 0, after 20 minutes are for all polder areas the same. After phase 1, 35 minutes, the polder area of 1 km² is already filled up. Between the polder area of 5 km² and 10 km² no difference in water level can be observed. The water level near the breach is still the same for all polder areas. After phase 2 different water levels near the breach are active, however, the 5 and 10 km² polder areas still have the same water level.

These polder water level comparisons are made with an in length varying polder. The width of the polder can be varied too. This will probably lead to a different water level distribution. A favourable aspect is that the water level close to the breach is almost equal to the outer water level during phase 2. Therefore, the hydraulic head over the breach is small and the velocities are reduced.

Conclusion: From a polder area of 5 km² enlarging has no effect anymore on this polder shape, within this timeframe. Difference in polder area has a relative small effect on the water level just downstream of the breach.

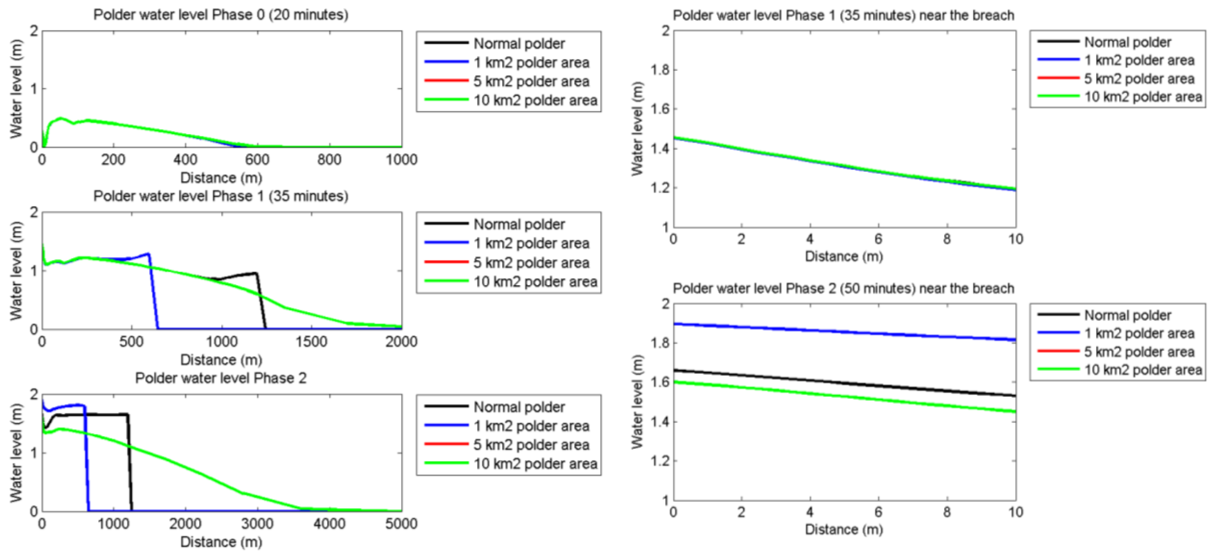

Figure 5.38: Polder water levels per phase

Moment of implementation

The moment the emergency measure is implemented has an effect on the development of the breach. To investigate the sensitivity to earlier or later implementation of the emergency measure, an accelerated and a delayed implementation is simulated. In Table 5.4 the different timespans are displayed.

Table 5.4: Timespan early, normal and delayed implementation

	Phase 0	Phase 1	Phase 2
Early	0 – 10 minutes	10 – 20 minutes	20 – 40 minutes
Normal	0 – 20 minutes	20 – 35 minutes	35 – 50 minutes
Delay	0 – 30 minutes	30 – 55 minutes	55 – 70 minutes

Phase 0 is the non-interfered breach development before the emergency measure is implemented. At the beginning of phase 1 the first layer is implemented and at the beginning of phase 2 the second. The results can be seen in Figure 5.39.

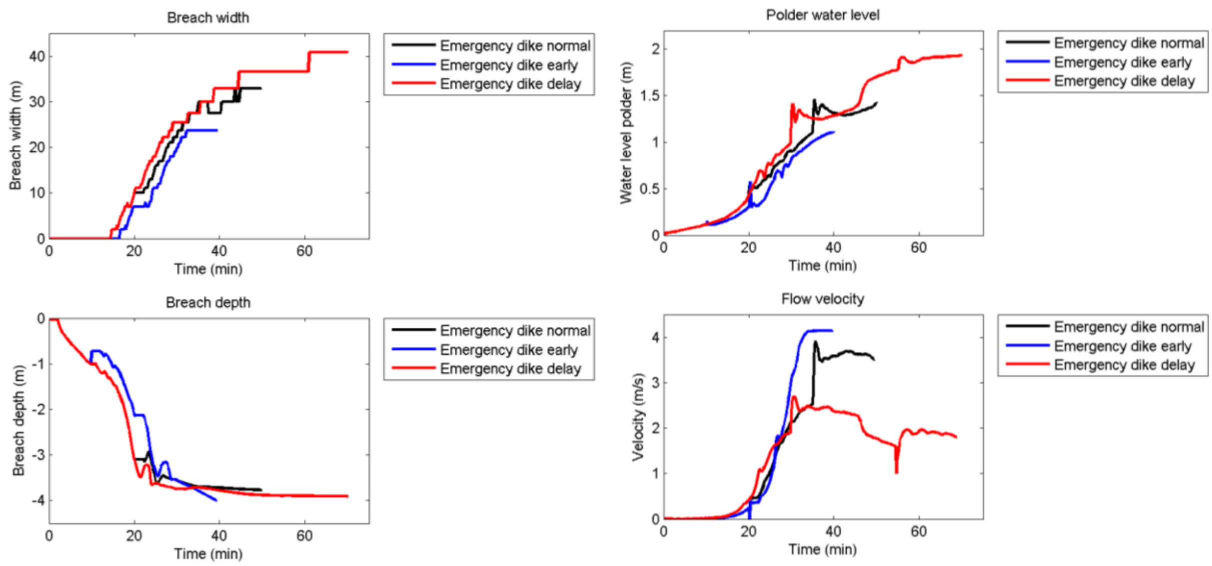

Figure 5.39: Effect of earlier and later implementation of the emergency measure

The width of the breach is influenced by the earlier and later implementation. The upper left plot of Figure 5.39 shows this. The breach width of the early implemented emergency dike is smaller than the normal implementation. Because of earlier implementation, less discharge is able to flow through the breach which induces less erosion. The other breach dimension, the depth, seems to be the same for the three implementation moments at the end of phase 2. However, at the start of phase 2, there is a difference. The breach depth of the early implementation at the beginning of phase 2 (20 minutes) is less than the breach depth of the normal implementation at the beginning of phase 2 (35 minutes) which is less than the breach depth of the delayed implementation at the beginning of phase 2 (55 minutes). If the breach is let alone after the implementation, the depth will develop further. The polder water level is lower if the measure is implemented earlier, since less discharge is able to flow into the polder. If the water level in the polder is lower, a larger hydraulic head over the breach is present which will lead to a higher flow velocity.

Conclusion: Earlier implementation creates a more favourable and later implementation makes matters worse.

Geometry

As variation of the geometry a berm at inner slope of the dike is added. The berm has an elevation of +1 m and is 4 meter wide. In Figure 5.40 the effects of a berm located at the inner slope on the breach dimensions, water level and flow velocity are displayed. The breach dimensions develop somewhat slower with a berm. This is due to the extra soil that needs to be eroded away before the breach can develop further in width or depth. The effect on the polder water level or the flow velocity is negligible. In the end, a berm located at the inner slope of a sand dike does not have any advantageous effects.

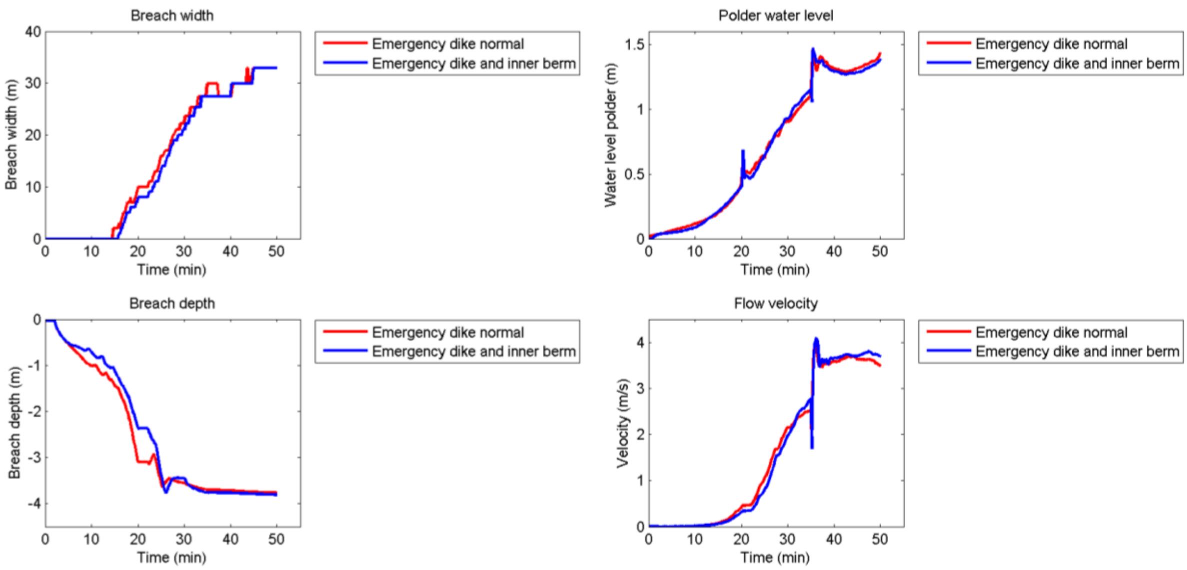

Figure 5.40: Effect of a berm at the inner slope

Conclusion: A berm does not have a large influence on the breach development.

Varying outer water level

Simulations above are done with a constant water level. This is the case for a primary water defence where the water storages can be assumed as infinite. However, if the outer water is not infinite, as is the case with regional water defences, the outer water level will drop. In this simulation the outer water level drops from 2.15 m to 1.75 m in 50 minutes. This is a decrease of 0.8 cm/minute. The drop in outer water level has an advantageous effect on the breach dimensions. The hydraulic head decreases, so the flow velocity and erosion power drop. Less water discharges through the breach, therefore the polder water level is increasing slower, see Figure 5.41.

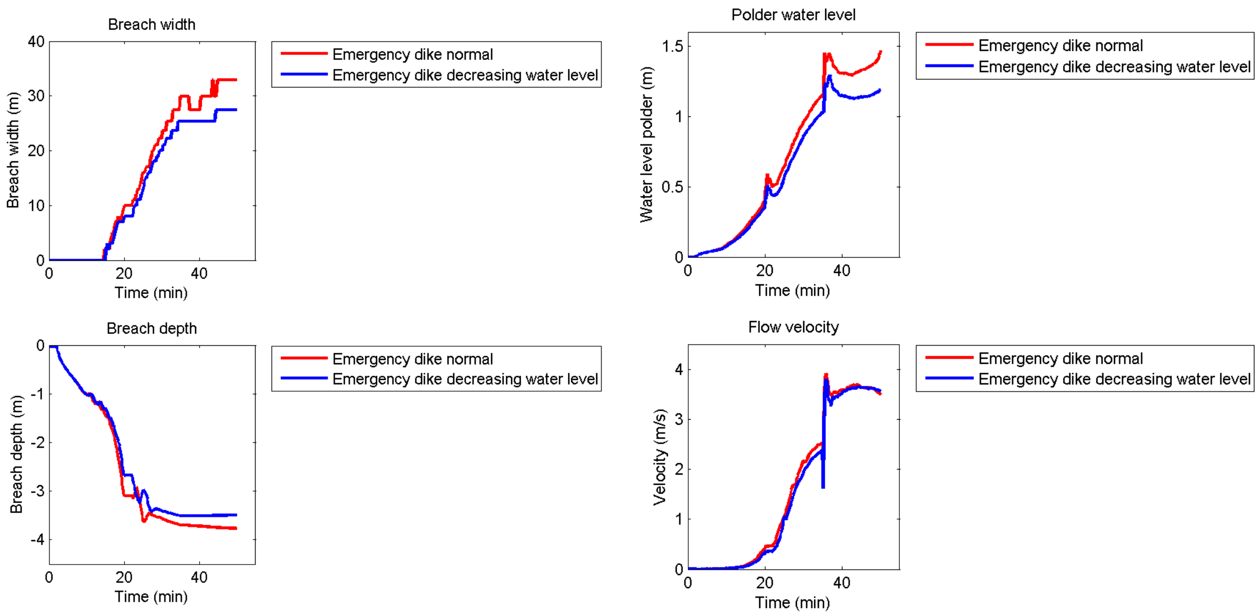

Figure 5.41: Effect of a decreasing outer water level

Conclusion: Dropping outer water level has a favourable effect on the breach development.

5.3.10 More realistic timeframe

More realistic closure time

XBeach simulations in this chapter are done for sand dikes. Plain sand erodes much faster than, cohesive soil like clay. Real dikes are not constructed of plain sand only. As was already discussed in Chapter 3.2.1, the dike itself as well as the subsoil consists of various types of soil. These real dikes don't erode as fast as the dikes modelled in XBeach. However, they go through the same stages of erosion. Visser (1998) for non-cohesive dikes and Zhu (2006) for cohesive dikes both found the same erosion pattern of five stages, see Chapter 3.2.2. The erosion mechanisms differ however. Sand erodes due to a high shear flow velocity and induced by the time varying flow. The second process is the avalanching mechanism. This mechanism makes the side slopes shear off if the gradient of the slopes becomes too steep. Clay erodes due to various mechanisms: flow shear erosion, fluidization of the surface of the slope, scour of the dike foundation and headcut undermining, and discrete headcut slope mass failure (Zhu, 2006). This results in an erosion process of clay where lumps of soil erode at once. However, for time indicative purposes, the achievement of the five breach stages of both soil types are compared.

Since they have the same five stages, it is attempted to obtain a scaling factor to be able to make a time estimation for the implementation of emergency measures in real (clay) dikes.

The scaling factor is calculated by dividing the time needed for the clay dike to get to a certain phase to the time needed for the XBeach simulation to reach the same phase. This factor is applied on the XBeach results to obtain realistic time frames. This scaling is without any scientific background. It is done to figure out an indicative time estimation to implement emergency measures in real dikes only. Table 5.5 compares a field test of non-interfered breaching in clay dike IMPACT Test1-02, carried out in the IMPACT program (Morris, 2011) to a dike with the same geometry water levels and polder areas modelled in XBeach. The field test was carried out on a 6 m high crested clay dike. The slopes are 1:2 and the crest width was 2 m. For further details, see Morris (2011). In this way, the XBeach simulation with a high theoretical but low reality level is compared to a full scale clay dike with a low theoretical level but a high reality level.

Table 5.5: Determination of scaling factor clay dike (IMPACT) and sand dike (XBeach)

	Clay dike IMPACT crest level +6 m	XBeach sand dike crest level +6 m	Scaling factor
Stage			
End of stage I	83 minutes	6 minutes	14
End of stage II	150 minutes	13 minutes	12
End of stage III	200 minutes	17 minutes	12
End of stage IV	250 minutes	37 minutes	7
End of stage V	380 minutes	52 minutes	7
Theoretical value	Low	High	-
Reality value	High	Low	-

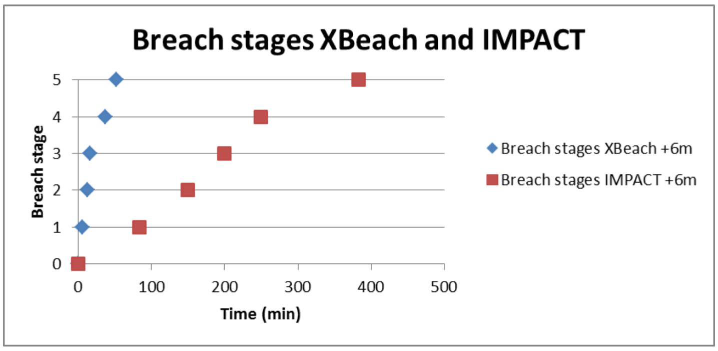

Figure 5.42: Comparison XBeach +6m (sand) and IMPACT +6m (clay)

The first three stages of breach development take considerable more time with a clay dike than with a sand dike. This can be explained since with the lower flow velocities in the first stages the clay particles don't get transported where the sand particles will be transported at these lower velocities. The obtained scaling is applied to the complete closure of Chapter 5.3.8. Here a sand dike with a crest height of +3 m is closed with an emergency dike. The results are visible in Table 5.6.

Table 5.6: Calculation stages of 'scaled dike' using the calculated scaling factor

	XBeach complete closure Chapter 5.3.8	Calculated scaling factor	'Scaled' clay dike
Stage			
End of stage I	5 minutes	14	70 minutes
End of stage II	12 minutes	12	144 minutes
End of stage III	18 minutes	12	216 minutes
End of stage IV	33 minutes	7	231 minutes
End of stage V	Not reached	7	Not reached
Theoretical value	High	-	Low
Reality value	Low	-	High

If the XBeach calculation of the dike with +6 m crest level is compared to the complete closure of Chapter 5.3.8 with a height of almost 3 m, the time needed to reach stage I, II and III do not differ that much. At first sight this may be strange because a lot more soil has to be eroded away for the +6 m dike, however, the driving force of the erosion, the hydraulic head, is a lot larger too. For the dike of almost +3 m crest height, the hydraulic head is smaller too. This seems to equal out.

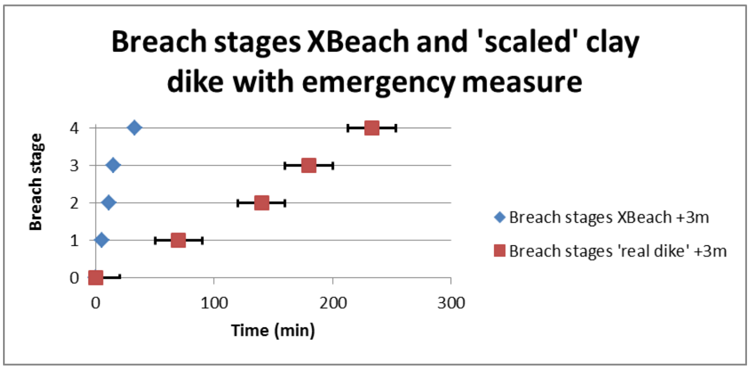

Figure 5.43: Comparison between XBeach +3m (sand) and a 'scaled' dike (clay)

<u>Discussion</u>

The author is aware of the impact of this assumption and that it is probably not fully correct. The scaling factor is based on a single comparison. With different geometry, soil characteristics or polder area this factor could differ from the one calculated here. For example, the compaction of a field experiment and a dike which has been in the same position for decades differ largely. Also, the difference in size of the dikes where the scaling factor is applied to, could be a serious increase in uncertainty. Still, the factor is used to give an indication of the time available in a realistic situation for the logistics of an emergency closure. In Figure 5.43, the timespan of the scaled dike is displayed. These are obtained by multiplying the values of the XBeach sand dike with the calculated scaling factor. However, a large uncertainty is included in this approach. For this reason the uncertainty bars are visible around the time span of the scaled dike. The uncertainty band is not calculated since the time span is used for indicative purposes.

Logistics

Now an indication for a more realistic time span is obtained, the logistical aspects of the closure can be investigated in more detail. The complete closure with an emergency dike, calculated in Chapter 5.3.8 is now checked for logistical aspects. This example is an indicative calculation.

<u>Material</u>

The emergency dike is constructed with Big Bags. The amount of Big Bags per phase is calculated below. There was calculated in Chapter 5.3.8 that at least 3 Big Bags in a row are needed for static stability.

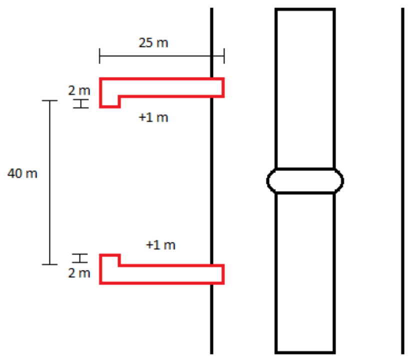

Figure 5.44: Phase 1 of implementation

Phase 1: In phase 1, two arms of 25 m and the first 2 meter of the emergency dike parallel to the dike are implemented, see Figure 5.44.
Number of Big Bags in phase 1 = 2 (3 x 25 + 3 x 2) = 162.

In phase 2, the number of Big Bags is: 3 x 36 = 108. For phase 3: 162 and for phase 4: 108. The total amount is 540 Big Bags. This amount is calculated much too precise for a real life case.

Probably more Big Bags are needed since some Big Bags will get flushed away during placement.

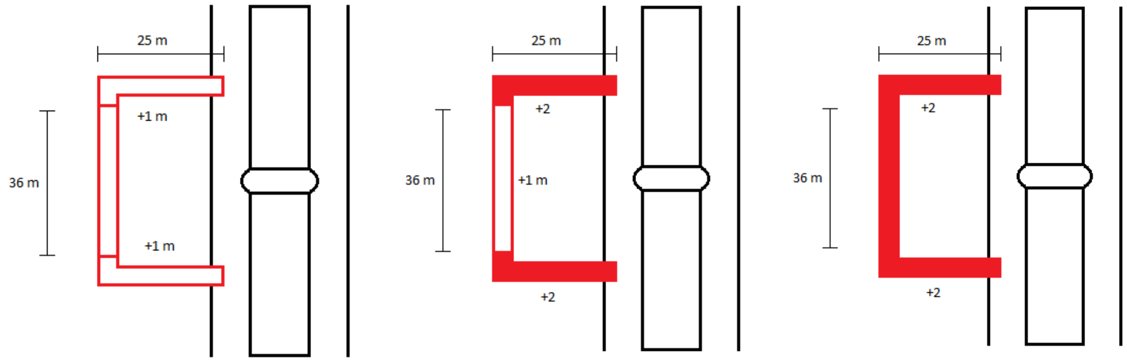

Figure 5.45: Phase 2, 3 and 4 of implementation

Equipment

Big Bags can be brought to the breach location by trucks. A single truck is capable of carrying about 15 m^3 of Big Bags. These are trucks with a self-unloading crane. Trucks will drive with the Big Bags to the location as close as possible and a helicopter will put them into place. The largest helicopter for transport in the Netherlands is the Chinook. This one is able to transport 12700 kilograms or 7 m^3 of Big Bags, when assuming a Big Bag weights 1800 kg.

Placement

The time from noting the weak spot in the dike until the start of breaching process is assumed to be one hour (Gerven, 2004). To implement phase 1, 162 Big Bags are needed. This equals 11 trucks loaded with 15 Big Bags each. The trucks bring the Big Bags to a location as close to the breach as possible, where the helicopters can pick up the Big Bags and place them in the breach. Two helicopters are assumed, working on both sides of the breach.

The trucks first need to get to the loading station. There is assumed that in a radius of 15 km, 11 trucks are available. They drive without load on average 50 km/h and are thus within about 20 minutes at the loading station. At the loading station, there is assumed that there are sufficient Big Bags available. Loading of the trucks takes place by the crane installed on the truck itself. This loading time is estimated to 10 minutes per truck. Distance to the breach location is assumed at 15 km. The velocity of a loaded truck is 40 km/h and it takes thus 25 minutes to arrive at the breach location. Once at the destination, the Big Bags need to be unloaded. This is assumed to take 10 minutes as well. The total time for the Big Bags to arrive at the breach is therefore 65 minutes.

They are placed in the emergency dike by a helicopter. It is assumed that a helicopter is 50 km away from the breach location. Their top speed without load is 300 km/u, so they can be present at the breach location in about 5 minutes. There is assumed that the helicopters only carry the Big Bags taken to the breach by the trucks. The time for a helicopter to place its load of 7 Big Bags is assumed at 10 minutes. For 162 Big Bags 2 helicopters need 12 flights. This takes a total time of 60 minutes. The total time to implement the first layer is thus 65 + 60 = 125 minutes. The time available for the 'scaled dike' is 200 minutes, see Table 5.7. The times in between brackets are the cumulative times.

More trucks will continue bringing in Big Bags and for the following phases there will be enough Big Bag present at the breach location. The helicopters are able to implement them in the calculated available time. This proves that the phasing as modelled in XBeach is logistically feasible.

Table 5.7: Determination logistical feasibility from XBeach phasing

	Big Bags needed	Time available in XBeach	Scaling factor	Time available 'scaled' dike	Actions	Time needed logistics
Phase 0	-	-	-	60 min	Mobilize trucks and helicopters and start transferring Big Bags	65 min
Phase 1	162	20 min	7	140 min (200 min)	Transfer Big Bags to location and implementation	60 min (125 min)
Phase 2	108	15 min	7	105 min (305 min)	Transfer Big Bags to location and implementation	40 min (165 min)
Phase 3	162	15 min	7	105 min (410 min)	Transfer Big Bags to location and implementation	60 min (225 min)
Phase 4	108	15 min	7	105 min (515 min)	Transfer Big Bags to location and implementation	40 min (265 min)
Total	540	65 min	-	515 min	-	265 min

The time needed during breach development is 200 minutes (265 minutes – 65 minutes) since phase 0 takes place before the breaching process starts. In Figure 5.43 and Table 5.6, there can be observed that the breach development time is about 230 minutes. This means that within this time the Big Bags can be placed.

The closure of a dike breach using an emergency dike of Big Bags seems plausible. However, it is based on a lot of assumptions. Once again it is stressed that this time indication is very uncertain and dependents on a lot of factors. The time available is calculated with a single scaling factor. Changes in this factor influence the available time enormous. There is calculated with a 'scaled' dike consisting of clay only. If the dike consists of other soil, the erosion can take place much faster. Also, assumptions about the trucks and helicopters are not certain. There is not taken into account what happens if trucks or helicopters have malfunctions. Another doubtful assumption is the one that the Big Bags are placed correctly at once and that there are no losses of washed away Big Bags. From stability calculations followed that the Big Bags should be tied together to stay stable, this fact is not elaborated any further in the logistic process. As last remark there can be stated that the suggested phases can be optimized regarding the logistical process.

5.4 Conclusions

Conclusions

General

There is a need to simulate breach development including emergency measures. Advantages of simulations are: It allows identification of breach characteristics like duration and breach stages during closure attempts. Furthermore, it can be used to optimize closure strategies and measures and it can help flood managers to prepare for emergency situations. Different scenarios can be simulated and 'off the shelf' strategies can be prepared for specific situations.

Software to make these simulations is barely available and if so the software is for breach development without emergency measures. With some assumptions XBeach can be used for simulating breach development with emergency measures.

XBeach

XBeach is a useful tool to simulate the effect of emergency measures in a developing breach in a sand dike.

XBeach has four main limitations:

1) Implemented non-erodible layers are not adjustable in time and the model is thus not able to simulate different steps in the closing procedure;
2) The non-erodible layer is always stable, thus emergency measures can not flush away by (too) large currents;
3) XBeach is no 3D model, so the piping mechanism can not be modelled;
4) XBeach is capable of simulations with non-cohesive sand only.

These limitations are dealt with in this thesis by:

1) Running multiple simulations after each other (very time consuming);
2) Hand calculations for stability requirements;
3) No modelling of emergency measures where a 3D effect (piping) plays an important role;
4) A translation from non-cohesive to cohesive timespans is made.

Emergency closures

Technique

It is extremely difficult to close a breach. From a technical view, the high flow velocities occurring during the breaching are the main constrain. There is (likely too) little time to close a breach in a sand dike.

Big Bags in a horizontal closure limit the horizontal breach development but enlarge the vertical. They have a small positive impact on the polder water level but enlarge the flow velocity. They are effectively applicable in the breach in stages I and II. The critical velocity of a 1 m^3 Big Bag is about 3 m/s.

A scaffold in a vertical closure limits the vertical breach development but enlarges the horizontal. They have a small positive impact on the polder water level but enlarge the flow velocity. It is effectively applicable in stages I and II. The critical velocity of a scaffold is 3 m/s.

A vessel in front of a breach reduces in first instance the breach development and discharge. However, after some time the breach width grows rapidly to larger dimensions than the 'do nothing' case due to the flow around the ship. The vessel will not be stable just after sinking down and needs to be increased in weight.

An emergency dike is the most promising measure. It makes use of the smaller flow velocities upstream of the breach. The breach dimensions stay smaller and the polder water level is lower. A complete closure is plausible using an emergency dike, however, the Big Bags used for the dike, have to be increased in weight. The static stability after the closure is a point of attention, with piping as most important threat.

Dike characteristics have an influence on the closure procedure. A decreasing water level and the moment of implementation of the emergency measure have a large impact. The size of the polder area is important but becomes irrelevant if the polder area is larger than the distance a flood wave can reach in the considered timespan. The presence of a berm in the geometry is negligible.

Logistics
Through a scaling factor the timespan for a closure in a clay dike is obtained. This timespan is applied to the complete closure with an emergency dike on a clay dike. Logistically seen, a complete closure with an emergency dike for a breach in a clay dike is plausible using trucks to bring in Big Bags and helicopters to place them at the desired location.

6. Field test

6.1 Introduction

Data from real dike breaches are scarce and far from complete and data of developing breaches with the effect of emergency measures are even scarcer. These data can be used to calibrate and validate breach erosion models and to gain more insight in the processes. For this reason field tests are important and one is carried out for this thesis. The experiments done in this thesis are some first experiments in this field.

The physical testing is done in *Flood Proof Holland*[4]. This is a testing and demonstration site for innovative temporary flood defences. In this testing site an experiment is done regarding curative emergency measures. Physical experiments are important because in this way data can be obtained. Therefore, the goal for the physical experiment done in this thesis is to establish if performing a physical experiment for curative emergency measures is worthwhile doing. The experiment contributes to a better understanding of the conditions when implementing a curative emergency measures during a dike breach and to the performance of physical experiments to simulate emergency closures.

The objectives of the physical experiment are listed in Chapter 6.2. In Chapter 6.3 the set-up of the experiment is explained. Results of the experiment are discussed in Chapter 6.4. This chapter ends with conclusions and recommendations for future experiments in Chapter 6.5.

6.2 Objective

The objective is double; to gain insight in the physical processes taking place during the implementation of an emergency measure and to establish if physical experiments with curative emergency measures are worthwhile doing. Furthermore, the goal is to come up with recommendations for future experiments, if the experiments are worthwhile. Logistically and organizationally all aspects are covered, so during the experiment the technical part is investigated. There will not be dealt with scale laws, it will be a basic experiment. The next sentence is composed as hypothesis: "A physical experiment wherein a sudden closure is performed is useful as start of more physical experiments regarding emergency closures of dike breaches."

6.3 Experimental set up

The basins in *Flood Proof Holland* are arranged as displayed in Figure 6.1. Basin 7 has a dike height of 2 m above ground level and is a storage basin. This is the central basin where the water is stocked for the surrounding basins. The other basins have a dike height of about 1 m above ground level. The dimensions of the large basin are about 45 m x 30 m, the

[4] *For more information, see the website: http://floodproofholland.nl/*

dimensions of a small basin are about 15 m x 30 m. In the large basin 1600 m³ of water can be stored. The dotted lines are tubes that transport the water to the basins. All of the tubes have valves to operate the discharge. Tubes that connect the large basin with the small basins have a diameter of 250 mm. For the dewatering of the small basins to the surrounding channel, a tube with diameter of 160 mm is installed. The small basins are connected with each other by the same 160 mm diameter tubes. The soil in *Flood Proof Holland* is a bare peat/clay mixture at the bottom of the small basins. The slopes are covered with grass. Materials used are sandbags, sand and BoxBarriers[5]. BoxBarriers are a temporary flood defence system which can be used to temporarily heighten the crest of a dike, or to make a temporary dike on flat terrain. The plastic box is filled with water and retains in this way the flood water by its own weight.

To carry out the experiments, a set up as displayed in Figure 6.2 is used. In the basin, BoxBarriers are placed from the slopes the centre, leaving about 1.5 m of space in between. The connections with the slopes at the outer sides of the basin are made water tight with sandbags. In between the BoxBarriers a sand dike is constructed. This sand dike has a depression in the middle to control the breaching, making sure the breaching process starts there, see Figure 6.3. For the experiment clean construction sand was used to construct the dike. The height of the dike is just below the BoxBarrier, about 45 cm. Somewhat lower is the depression, with a height of 40 cm above ground level. The slopes of the sand dike are on both sides the same and are about 1:2. A crest width of about 20 cm was constructed.

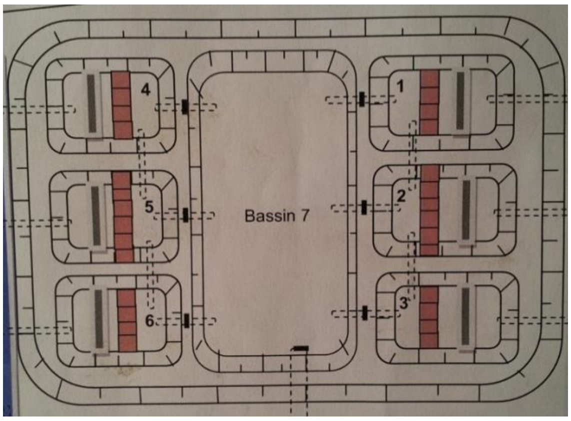

Figure 6.1: Map of *Flood Proof Holland*

Figure 6.2: Experimental set up

Figure 6.3: Depression in the sand dike

Basin 3 is used for the experiment and the basin is filled up with water by the tube which is connected to reservoir 7, visible at the bottom of Figure 6.2. Water fills up the basin until the water reaches the level of the depression in the sand dike. From this moment on the water overtops the dike and the erosion process starts. Emergency measures, simulated by a sandbag, are applied to investigate sudden closure effects.

[5] *For more information about the BoxBarrier, see the website http://www.boxbarrier.com/nl/*

Two experiments were carried out. Both experiments are the same, however, the conditions during both experiments were different. The first experiment was carried out right after the test site was set up. The other experiment was carried out 5 days after the test site was set up. In these 5 days the subsoil got soggy by the leaking water from the large reservoir. This influenced the phreatic line in the sand dike and the soil foundation of the BoxBarriers.

During both experiments, the water filled up the area in front of the test construction. This continued until the water reached the depression in the dike. From this moment on the breach development started. As emergency measure a sandbag is used. The dimensions of a filled sandbag are about 0.6 x 0.3 x 0.1 m. This sandbag is put in the breach at a certain moment. With this emergency measure a sudden closure is simulated. Two moments are picked to implement the bag. These are based on the breach stages as described in Chapter 3.2.2. The first moment is at the beginning of stage II, the second at the beginning of stage III.

6.4 Results

Experiment 1

The results of experiment 1, which is carried out directly after the set-up of the test construction are displayed in Figure 6.4. During this experiment the sand dike is dry and the subsoil is dry too.

Figure 6.4: Experiment 1, snapshots are taken at t = 5 s, 40 s; 55 s and 80 s after overtopping

During this experiment the breach developed exactly as described in Chapter 3.2.2. At the end of stage II, a sandbag was implemented. Right after the implementation piping underneath the sandbag started. The water took sand particles with it and soon a scour hole underneath the sandbag was formed, see the lower left photo in Figure 6.4. This erosion gap developed and the sandbag collapsed into the scour hole. A few moments later it was flushed away by the currents, see the lower right photo in Figure 6.4.

Experiment 2

The results of experiment 2, which is carried out four days after the set-up of the test construction, are displayed in Figure 6.5. During these four days about 15 cm of water was accumulated at the outer side of the test construction. This caused a higher phreatic line in the sand dike body and a soaked soil foundation of the BoxBarrier.

Figure 6.5: Experiment 2, snapshots are taken at t = 5 s, 30 s; 55 s, 70 s; 80 s and 95 s after overtopping

The experiment started in the same way as Experiment 1, however the erosion process took place slower. This is the case since the sandy dike body is saturated. The saturated sand body is stronger than the dry sand because of the water which holds it together by its surface tension. Without further elaboration on this phenomenon, there can be stated that the influence of the saturation of the dike body is large in this experiment. Again, at the beginning of stage II a sandbag was implemented, as displayed in the upper right photo of Figure 6.5. The implementation of this emergency measure stopped the discharge and thus the breach development. 25 seconds after the implementation the sandbag is removed and the breach development continues. At the beginning of stage III the sandbag is implemented again. The centre right photo in Figure 6.5 displays this implementation. This time it did not stop the discharge. Water flowed underneath the sandbag due to piping and some time later it took the sandbag with it as can be seen in the lower right photo.

6.5 Conclusions and recommendations

The conclusions and recommendations of this chapter are focussed on the continuation of physical experiments. Recommendations for more extensive and more detailed experiments are done. Carrying out physical experiments with emergency measures in developing breaches seems worthwhile, however a lot of adaptions are needed for a more true to reality experiment. The experiments can be used to compliment investigations via simulations. The can be used to increase confidence in a tested measure.

Conclusions

General

Experiments for developing breaches with the effect of an emergency measure are scarce. They can however be used to generate data for validation and calibration of models and to gain insight in the breach development and flow patterns. During these experiments the focus is on the technical aspect since logistics and organizational aspects are covered.

Experiments

Physical experiments regarding emergency measures in developing breaches are complex to perform. Despite the relatively simple experiment without scale laws, continuation seems useful. During the experiment, the breach development with the implementation of an emergency measure stays the same as without measure; the five breach stages were recognized.

The largest difference between the numerical model and this field test is that the emergency measure can become instable. After the implementation of the emergency measure at the end of breach stage II, piping underneath the sand bag occurred almost immediately. The erosion gap under the sand bag increases in size and the breach stages as distinguished continue. Even with the implementation of an emergency measure the five breach stages can be recognized. When the breach development reaches stage IV, the sandbag is not supported anymore by the dike and collapses to the bottom of the breach. The flow velocity at this location is high and therefore the sandbag becomes unstable and flushes away. This process is seen in both experiments.

In case of an immediate closure attempt with a sand bag, the initial failure mechanism is piping leading to the collapse. The second failure mechanism is instability due to the high flow velocity. If these failure mechanisms are compared to the model results, the failure mechanism due to flow velocity is captured in the hand stability calculations. However, the failure due to piping is not taken into account in XBeach.

The conditions in which the experiments are carried out play a large role. In the two experiments, the erosion in dry condition takes place faster than in saturated condition. In the second with saturated conditions it was possible to close the breach for a short period of time with a sandbag. As expected, saturation plays an important role in the experiment. With the dike consisting of sand, failure due to piping is likely to occur around the emergency measure. A sandbag as emergency measure is not able to stop the discharge through a sand dike.

Recommendations
The experiment performed was a relatively simple experiment. To get data from an experiment a more true to reality experiment needs to be set up. Several adaptions are recommended.

If the experiment is done on a certain scale, scale laws should be followed. Even better is to do the experiment in a full scale environment. In this way the real effect of the emergency measures can be observed.

Tests with cohesive soil give a more realistic representation. For the Netherlands the common dike types of clay or sand with a clay cover are recommended to adopt in future experiments. Also, revetments, toe constructions and other real-dike aspects should be tested.

External conditions seem to have large influences. There is recommended to construct the test set-up and let the dike body become saturated, which is the case in a real dike too. Especially for sand dikes this has a large effect.

In future experiments, the important characteristics should be monitored more closely. Recommended is to measure the growth of the breach dimensions and flow velocity.

Recommended is to test the emergency dike. The measures can be tested in the physical experiment to compare the results. Furthermore, it can be used to test the real feasibility.

7. Decision support for curative measures

7.1 Introduction

Up to now, the content of the chapters was mainly focussed on the technical aspects. In this chapter there is zoomed in on the logistics and organization that play a role in breaches and breach closures. This chapter combines the gained knowledge from previous chapters and provides considerations on how to arrive at a decision support system that can be used before or during an actual breach development. To investigate this, a review is made of the current practice at the Dutch Water Board Rivierenland. The goal is to identify steps that need to be undertaken to arrive at a decision support system in relation to the closure of breaches.

Chapter 7.2 discusses the current practice at the Dutch Water Board Rivierenland. The emergency response in the preventive phase and the phase after the start of a breach are explained. In Chapter 7.3 the considerations for a decision support are given. Practical instructions are given in Chapter 7.4. Conclusions are drawn in Chapter 7.5.

7.2 Current practice Dutch Water Board Rivierenland

The procedures described in this paragraph are based on an interview with the dike specialist of Water Board Rivierenland. For the complete interview, see Appendix VI.

At this moment the emergency procedures focus on the phase until the start of a dike breach formation. For preventive measures there are emergency plans, organizations and work instructions. If a dike breaches, no such things exists. The Water Board indicates that there is a need for such plans, organization and protocols for the curative emergency phase as well. In the present situation, if a dike breaches, the Water Board would have to improvise.

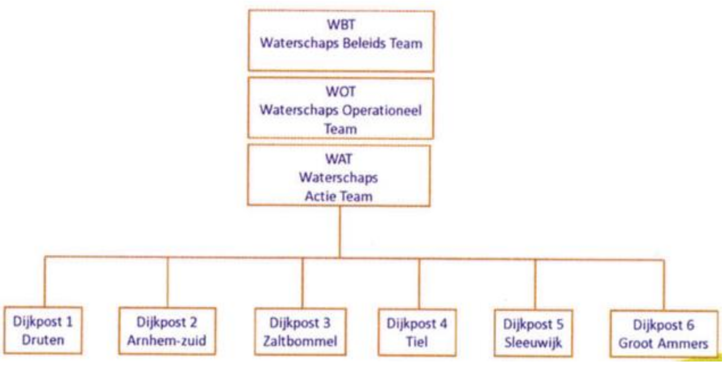

Figure 7.1: Emergency organization Water Board Rivierenland

The current emergency organization of Water Board Rivierenland is displayed in Figure 7.1 and Figure 7.2. At the top of the organization, the Water Board Policy Team (WBT) makes decisions at strategic level during an emergency. Below the Policy Team, the Water Board

Operational Team (WOT) makes decisions at tactical level during an emergency. The Water Board Action Team (WAT) supports the implementation of drastic or risky emergency measures by consulting experts and giving advice. Water Board Rivierenland is split into six so-called dike posts, all responsible for a part of the Water Board's area. These dike posts coordinate the deployment of dike watchers (staff that monitor the condition of flood defences) and have contact with contractors for emergency repairs. The dike watchers are on the dike during high water conditions and monitor the condition of the dike. If damage is detected by the dike watchers they fill out a form making the state of the damage clear to the dike posts. Dike watchers are inspecting the dike by car and by foot. By foot, the dike watchers take approximately five hours to inspect their course. Also, there is a list with potential hotspots per dike stretch to be checked. The procedure after the detection of damage is displayed in Figure 7.3. If the damage needs a drastic emergency measure the WAT will give their advice. If the emergency measure contains no specific technical, juridical or social risk and the costs are below €50,000, the dike watchers in collaboration with the dike post head, are allowed to repair the damage themselves, after consulting the dike post. The reason for the WAT to come into play is that the managers of the dike posts do not have specific knowledge of water defences and the WAT does.

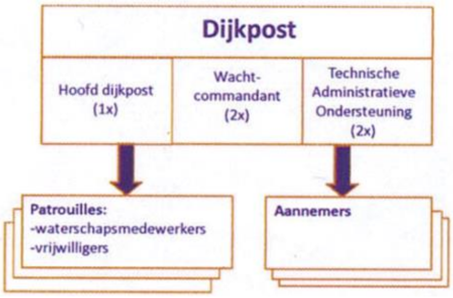

Figure 7.2: Organization dike posts Water Board Rivierenland

To manage the information needed to take decisions during flood events, FLood Information & WArning System (FLIWAS) is used. This tool is a web-based system that provides, shares and communicates up to date information about floods. In FLIWAS information about the water defences, water levels, and emergency measures is stored. The system is designed to take better decisions by providing the information to the right person, at the right time, at the right location (Gooijer, 2010). Implemented emergency measures are fed into FLIWAS as well. Another tool used during flood threat is the use of protocols for the placement of preventive emergency measures.

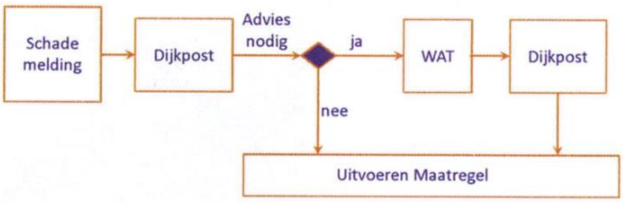

Figure 7.3: Emergency procedure Water Board Rivierenland

If the material and equipment stocked at the Water Board is not sufficient to repair the damage to the dike, a contractor is called in. Contractors are warned before the high water, so their material and equipment is already mobilized. It takes contractors about 12 hours to mobilize their material and equipment.

Above described organization and procedures hold for preventive emergency measures. Currently Water Boards assume that the preventive measures are effective and sufficient to counter the damage to the dike and prevent a breach. The situation of a breach is in fact not considered. The assumption can be made that the damage to the dike is discovered before the breach forms. In the case that a breach occurs the organization will be the same as in the phase before a breach. The implemented measure will be an improvised one. It was pointed out in the interview that the materials of the contractors would probably not be sufficient to close a breach. The transport routes used for preventive measures can however be used in the curative phase.

7.3 Considerations for decision Support

7.3.1 Current state of decision support in flood management

In the ideal world decision support in flood management applications is linked to various (real-time) sources of information. They provide information to support the decision making process of what needs to be done when and where. Water Boards have such systems in place for some daily operations, for example to control the water level in their catchment. However, for preventive emergency measures, decision making is supported via various tools that are mainly on paper. These are for example instructions and protocols on how to detect damage on a flood defence or how to implement a certain emergency response measure. Their success depends on the skills of the staff involved. However no tool exists to optimise which and how preventive emergency measures are implemented. At this moment such a tool is being developed.

Regarding curative emergency response measures, the situation will be even more challenging. Not only are there no protocols, instructions and plans in place, the conditions for carrying out such plans in emergency situations are far more challenging simply because information is likely to be less structured and there is less time. In this section therefore it is important to keep such thoughts in mind when defining steps that can be undertaken by Water Board Rivierenland.

7.3.2 Functionality needed in decision support for curative measures

As mentioned earlier, Water Board Rivierenland pretty much starts from scratch regarding the decision support for the implementation of curative measures. Therefore any decision support should be more aimed at collecting basic information and defining basic plans and instructions for their implementation. Such plans could be based on a scenario type of approach.

In this case the decision support would be a paper plan, which contains instructions. In the future possibly such a plan can be worked out in more detail, possibly even linking it to information systems that make use of real-time information. Information needed is based on the important characteristics from Chapter 3 and model and calculation parameters used in Chapter 5.

The basic information would be aimed at:

- Information on potentially weaker spots in the flood defences;
- Geotechnical build-up and conditions of the subsoil;
- Dike characteristics as geometry, revetments and polder area;
- Expected dominant failure mechanisms;
- Availability of resources, including contractors, storage places and type of existing measures;
- Information on access roads to such spots;
- Indication of times needed to get material and equipment to the breach.

The plans and instructions would be aimed at:

- Defining scenarios related to breaches for such weak spots: duration of breach stages and effective breach closing measures;
- Modelling of such scenarios to test the reliability of such measures;

- Translating this to protocols and instructions that define certain pre-conditions and actions of what needs to be done when.

In other words, the decision support is aimed at providing off-the-shelf assistance. Such plans and instructions could be furthermore tested in exercises, therefore making sure all parties understand what needs to be done where and when and to ensure that what is being proposed is realistic and indeed potentially effective.

7.3.3 Suggestions for decision support for Water Board Rivierenland

The decision support plan for curative emergency measures has to provide a procedure to come up with the most effective emergency measure for a (possibly) developing breach and give advice about the implementation and logistical aspects, connecting to the current practice. The objective of this section is to arrive at the contours for such a plan. In this paragraph, desires from Water Board Rivierenland are taken into account. These are obtained by an interview, see Appendix VI.

Characteristics for the plan are:

- Quick and simple;
- Functioning with the same organization and terminology as preventive measures;
- No hampering of preventive measure team;
- Gives most effective measure;
- Gives logistical advice regarding measure;
- Indicates time needed and available;
- Ideally, the approach used should preferably be suitable for broader application, preferably even on a national scale. However in this section we will limit it to Water Board Rivierenland.

Assumptions that are done:

- Potentially weak spots in the flood defences have been identified, including their dominant failure mechanism as well as geotechnical build-up, subsoil and dike characteristics;
- There is assumed that the damage to the dike is detected before a breach is started;
- Availability of resources, including contractors, storage places, type of existing measures;
- Transport routes and timeframes are known;
- Team to place preventive emergency measures is at the location.

Above all, we need to define what needs to be done beforehand ("cold phase") and what needs to be done when it actually happens ("warm phase"). In the "cold phase" the aim is to prepare the plans and instructions for the main weak spots. In the "warm phase" the aim is to know quickly, based on certain given information "what options are available" and "which instructions" need to be followed. This will be elaborated in the next section.

<u>Cold phase</u>

For the "cold phase" it is recommended that the Water Board comes up with a plan that highlights the various potential weak spots in the flood defence and defines how measures could be successfully implemented for each of them. In the following text box an outline description is given of this plan. In the cold phase the protocol which should be followed in the warm phase is composed too.

In this plan the before known weak spots are listed. For these spots the key characteristics for prediction of breach forming are collected and emergency closures are worked out. These emergency closures are tuned to the logistical requirements. The organization behind these actions is defined. Also, the instructions for the implementation of the closure are set up.

Plan for Curative Measures for Breaches

1. Introduction (objectives, ambition Water Board in dealing with such measures)
2. Overview of organisational setting
3. Overview of weak spots in the flood defences
4. Per weak spot
   - Geotechnical characteristics
   - Dike characteristics
   - Availability of resources
   - Access including timespan to the breach
   - Breach phasing (scenarios)
   - Definition of which measures are still effective when
   - Selection of promising measures and definition of key requirements
   - Define decision making process (what information is needed and what output is expected)
5. Instructions
   - Define start of the process, what information triggers the curative measure process
   - Clear linking of input information to output actions, responsibilities
6. Exercises
   - Suggestions on how to test the instructions, ie what are key success factors for the implementation of the instructions
7. Conclusions & Recommendations

<u>Warm phase</u>

The procedure for actually implementing instructions is described next. This procedure should be prepared in the cold phase and followed in the warm phase. The start of the curative decision support phase takes place at the moment of the detection of the damage. If the damage is serious enough, or on advice of the WAT, a member of the team, which is at the location of the damage informs a team member back at the office. The team at the office stands in direct connection with WAT. The information that the person at the dike needs to transfer is the location and the failure mechanisms with some key characteristics about the degree of damage. Also the location should be referenced to the most relevant reference weak spot for which plans and instructions have been devised.

After this transfer of information the team at the dike can focus on the preventive measure. The team at the office however continues also with the curative phase. The information about the subsoil, dike type, water conditions, type of polder and failure mechanism is gathered and organized and it is reconfirmed that the reference weak spot applies to the location at hand. Analysis of the possible breach development is done. Based on this it is clear where we stand in time and how much time is likely to be available. However, we need to distinguish three paths:

1) The dike is damaged, preventive measures are underway, it is expected that the risk of flooding will be averted → no further actions re curative measures
2) The dike is damaged, preventive measures are underway, it is expected that the risk of flooding will possibly not be averted → actions for curative measures are put in place (the Water Board has a "head start" in preparing)
3) The dike has started to breach, preventive measures have failed and it is essential that curative measures are implemented → actions for curative measures are put in place (there is no time to waste).

Whether you have a head start or not, using the instructions, it should become clear which measures could still be effective and where such measures are available. Updates from the team implementing the preventive emergency measure will inform the office team about the situation and the threat of a breach. Based on well-defined input it becomes clear which measures are worth reviewing. Contact with contractors should allow confirmation whether these measures can be mobilized to the location on time. Based on this a go/no-go decision can be taken.

If a go is decided, there are subsequent instructions, on how these measures can be put in place. These instructions provide a step-wise approach, and are described also using illustrations.

7.4 Practical instructions

With the gained knowledge in the previous chapters some initial general instructions can be drafted. The gained knowledge is transferred into more solid do's and don'ts for laymen wanting to counter breach development. As was observed in the case studies in Chapter 4, people often panic and make wrong decisions. With these non-side specific simple instructions spilled effort can be limited and unnecessary extra damage to the dike can be prevented. The instructions are categorized by the stages of the breaching process. Also, some time before the breaching process starts is taken into account, called preparation time.

Preparation

Flow velocity:	-
Breach width:	0 m
Breach depth:	0 m

The preparation phase can be used to bring material, equipment and manpower to the weak spot. Still, the priority should be preventing a breach.

Do's

- Try to avoid a breach by preventive measures;
- Bring in an emergency team including soldiers from the army;
- Establish transport routes to the weak spot;
- Bring in as much material as possible to close the breach, Big Bags are the most useful and easy material;
- Equipment like trucks need to bring in Big Bags and helicopters need to be prepared to place them in the breach.

Stage I

Indicative flow velocity:	0 - 2 m/s
Indicative breach width:	< 5 m
Indicative breach depth:	< height of the dike

In this phase the flow velocities are small enough to close the breach in the breach itself.

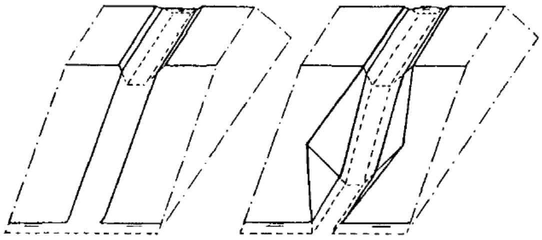

Figure 7.4: First and last image stage I, (Visser, 1998)

Do's

- Try to close the breach with Big Bags in the breach itself;
- Big Bags are stable everywhere in the breach

Stage II

Indicative flow velocity:	1 - 3 m/s
Indicative breach width:	< 5 m
Indicative breach depth:	± height of the dike

In this phase the flow velocities are still small enough to close the breach in the breach itself.

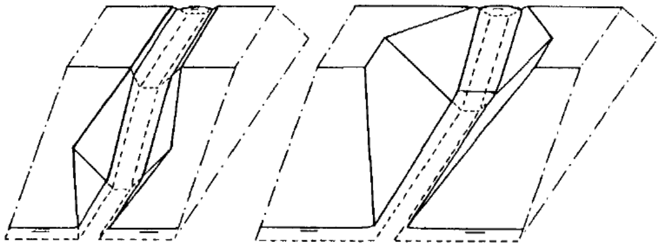

Figure 7.5: First and last image stage II, (Visser, 1998)

Do's

- Try to close the breach with Big Bags in the breach itself;
- Big Bags are stable everywhere in the breach

Stage III

Indicative flow velocity:	2 – 5 m/s
Indicative breach width:	< 5 m
Indicative breach depth:	± height of the dike

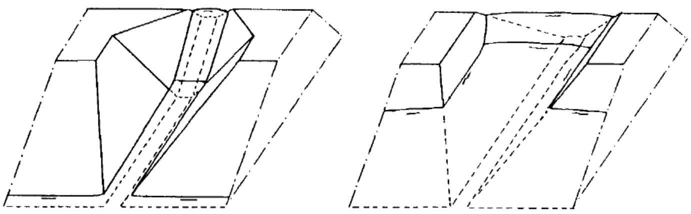

Figure 7.6: First and last image stage III, (Visser, 1998)

From stage III on, the velocity in the breach is too large to close the breach at the location of the breach itself.

Do's

- Place Big Bags upstream of the breach to construct an emergency dike;

Don'ts

- Try to close the breach with Big Bags in the breach itself;

Stage IV

Indicative flow velocity: 4 – 8 m/s
Indicative breach width: > 5 m
Indicative breach depth: ± 2 x height of the dike

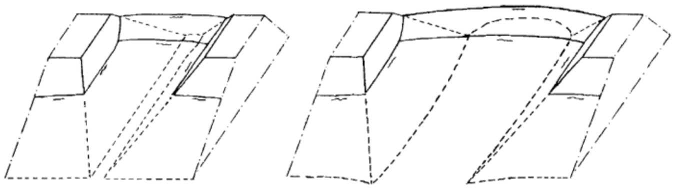

Figure 7.7: First and last image stage IV, (Visser, 1998)

In stage IV the velocity in the breach is too large to close the breach at the location of the breach itself.

Do's

- Place Big Bags upstream of the breach to construct an emergency dike;

Don'ts

- Try to close the breach with material in the breach itself;

Stage V

Indicative flow velocity: 2 – 6 m/s
Indicative breach width: >> 5 m
Indicative breach depth: ± 2 x height of the dike

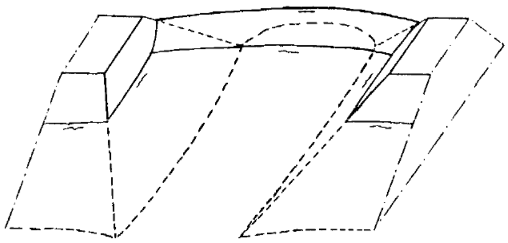

Figure 7.8: First image stage V, (Visser, 1998)

Near the end of stage V, the flow velocity could be decreased to a value where Big Bags are stable in the breach again. However, if the emergency measure still needs to be implemented, it will be not effective anymore since the water level is almost equal on both sides of the dike.

7.5 Conclusions

Currently, there is no decision support regarding curative measures. Dutch Water Board Rivierenland indicates that there is a need to have such a system. However, this needs to be built up from scratch.

Therefore, any decision support should be more aimed at collecting basic information needed for their implementation and defining basic plans and instructions for their implementation. The decision support is divided in a 'cold' and a 'warm' phase. For the cold phase, a plan that highlights the various potential weak spots in the flood defence and defines how measures could be successfully implemented for each of them is drafted. The warm phase should follow these procedures for actually implementing emergency measures.

Both the cold and the warm phase of decision support are first recommendations. A lot of progress still can be made regarding these systems.

8. Conclusions and recommendations

8.1 Conclusions

8.1.1 Main conclusions

This chapter provides the conclusions of this thesis. The aim of this thesis is:

"Find a robust framework for the design, management and operation of an emergency closure of a dike breach."

The conclusions are split in main and side conclusions. First the main conclusions are presented.

Cases

- Common aspects of the successful closures are:
  - Improvisation and quick action, however, it should not go at the expense of hydraulic thinking;
  - Availability of material, equipment and manpower;
- Common aspects of the failed closures are:
  - Lack of a solid closure strategy;
  - Lack of material, equipment or manpower;
  - Pressure from politics and inhabitants influences wrong closure strategies.

XBeach

- With modifications for stability, adjustment of non-erodible layers, 3D effects and the timespan of the erosion for realistic dikes, XBeach is a useful tool to simulate the effect of emergency measures in a developing breach in a sand dike;

Emergency measures

- An emergency dike is the most promising measure. It makes use of the smaller flow velocities upstream of the breach. The breach dimensions stay smaller and the polder water level is lower. A complete closure is technically and logistically plausible using an emergency dike, however, the Big Bags used for the dike do have to be increased in weight. The static stability after the closure could be a problem, with piping as most important threat;
- Dike characteristics have an influence on the closure procedure. A decreasing water level and the moment of implementation of the emergency measure have a large impact. The size of the polder area is important but becomes irrelevant if the polder area is larger than the distance a flood wave can reach in the considered timespan. The presence of a berm in the geometry is negligible.

Field test

- The initial failure mechanism is piping leading to the collapse. The second failure mechanism is instability due to the high flow velocity. If these failure mechanisms are compared to the model results, the failure mechanism due to flow velocity is captured in the hand stability calculations. However, the failure due to piping is not taken into account in XBeach.

8.1.2 Side conclusions

The side conclusions are presented next. After some general conclusions, conclusions per section on the basis of the sub objectives are drawn.

Framework

- The main problems of the emergency closure of a dike breach are:
    1. Limited knowledge about what closure strategy or measure to pick, what its effect will be and if it will be stable during the challenging conditions of a breach;
    2. Limited preparation time and the short time available to mobilize material, equipment and men to close the breach;
    3. Lack of a protocol, flood managers often do not know what to do during a breach or who is responsible.
- These problems can be categorized by, corresponding to the numbers above:
    1. Technical problems;
    2. Logistical problems;
    3. Organizational problems.

  In this thesis the main focus is on the technical problems.
- Zooming in on the technical aspects, two main fields can be distinguished: breach characteristics and emergency measures. These are elaborated on in the cases, simulations and calculations and the field test.
- Logistical and organizational aspects are touched upon briefly in the set-up for the decision support.

General

Conclusions for this part have been drawn for the objective: *"Distinguish the different critical aspects of a dike breach into separate tangible parts that can be understood."*

- The closure of a breach is very difficult and is rarely performed successfully.
- There is a need for research regarding emergency closures of dike breaches. This statement is supported by among others the Dutch Water Board Rivierenland. The research into breach development is still in an early phase, let alone the research regarding emergency measures to counter a breach.
- The type of dike, geometry of the dike, revetment, subsoil, category water defence, flood wave and failure mechanism are important dike characteristics. These characteristics play a major role in the development of a breach.
- Non-cohesive dikes and cohesive dikes both have the same five stages of breach development. The main difference is that the breach process goes slower in a cohesive dike. Also, the way cohesive and non-cohesive dikes erode differs.

- For curative measures, implementation thereof can benefit from what has already been done for preventive emergency measures, e.g. transported equipment and routes to weak places at the dike that can be established and prepared.
- Curative measures with the highest potential Big Bags, a scaffold and the PLUG.

Cases
The conclusions are based on the objective regarding case studies: *"Collect lessons learned from dike breaches and the applied emergency measures."*

- There are very few cases of dike failure where emergency measures were applied and measurements of the breach development have been documented.
- It is useful to investigate cases because in this way common success or failure factors regarding an emergency closure can be compared to other cases and lessons learned. Promising strategies or measures can be investigated in more detail.
- Other conclusions with respect to the investigated cases are:
    - The closure method is often determined by the situation, the stage and development of the breach and the available equipment;
    - If a breach starts developing, panicking people tend to throw in whatever they can find at the sides of the breach, forming a horizontal closure;
    - Horizontal closure methods fail often, because they are started as described above, however they are performed under public pressure;
    - Vertical closure methods are more effective than horizontal closures because the hydraulic conditions are more favourable;
    - Vessels can be useful to reduce the discharge through the breach in early stages of the breach and can therefore be used in supplement of additional measures.

Simulations and calculations
Conclusions are drawn on the objective: *"Understand the effect of an emergency measure and know its application range."*

<u>General</u>

- To get a better understanding of the effect, simulations of developing breaches including emergency measures can be made;
- Advantages of simulations are: Simulation of such processes allows identification of breach characteristics like duration and breach stages during closure attempts. Furthermore, it can be used to optimize closure strategies and measures and it can help flood managers to prepare for emergency situations. Different scenarios can be simulated and 'off the shelf' strategies can be prepared for specific situations;
- Software suitable for this is barely available and if so, the software is for breach development without emergency measures;
- With some modifications XBeach can be used for simulating breach development with emergency measures.

XBeach

- XBeach is a useful tool to simulate the effect of emergency measures in a developing breach in a sand dike;
- XBeach has four main limitations:
  1) Implemented non-erodible layers are not adjustable in time;
  2) The non-erodible layer is always stable;
  3) XBeach is not a 3D model;
  4) XBeach is capable of simulations with non-cohesive sand only.
- These limitations are dealt with in this thesis by:
  1) Running multiple simulations after each other (very time consuming);
  2) Hand calculations for stability requirements;
  3) No modelling of emergency measures where a 3D effect (piping) plays an important role;
  4) Using a scaling to estimate cohesive timespans from non-cohesive ones.

Emergency closures

Technique

- It is extremely difficult to close a breach due to the high flow velocities;
- Big Bags in a horizontal closure limit the horizontal breach development but enlarge the vertical. They have a small positive impact on the polder water level but enlarge the flow velocity. They are effectively applicable in the breach in stages I and II. The critical velocity of a 1 m^3 Big Bag is 3 m/s;
- A scaffold in a vertical closure limits the vertical breach development but enlarges the horizontal. They have a small positive impact on the polder water level but enlarge the flow velocity. It is effectively applicable in stages I and II. The critical velocity of a scaffold is 3 m/s;
- A vessel in front of a breach reduces in first instance the breach development and discharge. However, after some time the breach width grows rapidly to larger dimensions than the 'do nothing' case. The vessel will be stable during the breach development;
- An emergency dike is the most promising measure. It makes use of the smaller flow velocities upstream of the breach. The breach dimensions stay smaller and the polder water level is lower. A complete closure is plausible using an emergency dike, however, the Big Bags used for the dike do have to be increased in weight. The static stability after the closure could be a problem, with piping as most important threat;
- Dike characteristics have an influence on the closure procedure. A decreasing water level and the moment of implementation of the emergency measure have a large impact. The size of the polder area is important but becomes irrelevant if the polder area is larger than the distance a flood wave can reach in the considered timespan. The presence of a berm in the geometry is negligible.

Logistics

- There is (likely too) little time to close a breach in a sand dike.
- With the use of a scaling factor, it has been established that logistically seen, a complete closure with an emergency dike for a breach in a clay dike is plausible using trucks to bring in Big Bags and helicopters to place them at the desired location.

Physical testing
Conclusions for this part are drawn on the objective: *"Find out if field testing is useful."*

General
- Experiments carried out to investigate developing breaches are scarce, the experiments for developing breaches with the effect of an emergency measure even scarcer;
- Experiments can be used to generate data for validation and calibration of models and to gain insight in the processes taking place;
- Furthermore, they can be used to complement investigations via simulations and increase confidence in promising measures;
- Physical experiments regarding emergency measures in developing breaches are complex to perform;
- During these experiments the focus is on the technical aspect since logistics and organizational aspects are covered.

Field test
- Even in small scale experiments the water is powerful and the flow velocity high;
- The breach development with the implementation of an emergency measure stays the same as the situation without measures; the five breach stages were recognized;
- With the dike consisting of sand, failure due to piping is likely to occur around the emergency measure;
- The erosion in dry condition takes place faster than in saturated condition;
- In saturated conditions it was possible to close the breach for a short period of time with a sandbag;
- A sandbag as an emergency measure is not able to stop the discharge through a sand dike;
- Recommendations to include in future experiments are cohesive soil and the testing of promising measures.

Decision Support System
"Connect the knowledge gained in this thesis to the current practice at Dutch Water Boards."

- Dutch Water Board Rivierenland indicates that there is a need to have decision support in place. This could be a protocol or a plan. However, this needs to be built up from scratch.
- Decision support should be more aimed at collecting basic information needed for the implementation of emergency measures and defining basic plans and instructions for their implementation.
- The decision support is divided in a 'cold' and a 'warm' phase. For the cold phase, a plan has been drafted that highlights the various potential weak spots in the flood defence and defines how measures could be successfully implemented for each of them and making sure that tasks, roles and responsibilities are clear. The warm phase should follow procedures for actually implementing emergency measures.
- Both the cold and the warm phase of decision support are first recommendations. A lot of progress can still be made regarding these systems.

8.2 Recommendations

Emergency closures of dike breaches are very complicated processes with unsatisfactory understanding so far. Therefore, there is a need to elaborate on this topic and various parts are still open to further investigation.

Specific topics for further research

- Use of the XBeach model seems promising. However, this thesis can be seen as the first step in modelling of emergency closures and improvement is necessary. The most important points for improvement are:
  - A possibility to make the non-erodible layer or object unstable due to large currents or scour to simulate flushing away of emergency measures;
  - A possibility to model cohesive soil to simulate real dikes;
  - A possibility to make the non-erodible layer adjustable in time to simulate different phases of the closure.

- By improving the model, the following recommendations are made regarding actual breach closure:
  - More elaborate research is desired on the stability of emergency measures due to scour, actual implementation and failure of the measure itself;
  - The effect of failure of parts of the emergency measure on the breach development needs to be studied as well;

- A final recommendation is to apply the model after adaption on real cases of weak spots to come up with plans for emergency closures. In this way, practical problems will be discovered and can be tackled.

General

- In the future, efforts must be made to document actual breaches and measures such that data can be obtained and lessons can be learned.

- Further investigation is needed into improvement of the static stability after the closure.

- The Decision Support System should be worked out in detail to generate smooth decision making.

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APPENDICES

I. Breach process and emergency measures

I.1 Breach process

Several breach situations can be distinguished. If the bottom of the breach is situated in a part of the dike that is not regularly below the water level, the breach can be repaired quite easy in dry conditions, for example using sandbags or local clay. Breaches where water erodes the soil away, thus where the water level is above the bottom of the breach, are less easy to repair. The focus is on the last mentioned breach situation. Sometimes toe protection or an old clay core of the dike resists a quick formation of the breach. The development of a breach has four main drivers. The first is the area and the height of the hinterland. The head difference and the subsoil of the dike are two other drivers. The fourth driver is the characteristic of the foreland and the hinterland around the breach (Rijkswaterstaat & KNMI, 1961). A dike breach can be seen as a gap which is not in equilibrium state. Nature will try to achieve an equilibrium state. In this case by discharging water through the breach to achieve equal water levels at both sides of the dike. The flowing water will damage the dike further and the situation will change more in the direction of an equilibrium situation (Verhagen, et al., 2012).

I.1.1 Failure mechanisms

Overtopping

The overtopping failure mechanism starts at the crest of the dike. Overtopping starts with a single gully. This initial breach is schematized in the existing literature. Overtopping is the most common failure mechanism in the historical analysis (Vorogushyn, et al., 2009). In the existing literature the initial breach is assumed at the crest of the dike, like an initial breach induced by overtopping. The dike body stays intact and the breach cuts from above through the dike body.

Piping

Piping could only happen with a clay dike on a sandy subsoil (Rijkswaterstaat, 2012c). Piping clearly does not start at the crest of the dike. In existing literature the failure mechanism piping is schematized with an initial breach at the crest of the dike. The explanation of the initial breach at the crest of the dike according to Visser (1998) is: “the ‘pipe’ through the dike will collapse after some time, resulting in a breach at the crest of the dike.” In this paragraph the initial breach caused by the piping failure mechanism will be investigated more specifically. The goal is not to describe the formation of the breach mathematically. There is aimed for a schematic description of the initial breach formation. In this way the effect of emergency measures will be more accurate to describe.

The breach formation in case of failure due to piping is recorded for the case study in Jiujiang, see paragraph 4.2.2. This case will be taken as source for the formation of the initial breach for the piping failure mechanism since it is captured in detail (Rage of the River Gods, 2001).

To schematize the breach development initiated by the piping failure mechanism, the same approach as in Visser (1998) and Zhu (2006) is used. The process is schematized in several stages, starting from the moment of the washing out of the first soil particle, based on GeoDelft (2002), Expertisenetwerk Waterveiligheid (2010) and Rijkswaterstaat (2012c):

I. Development of the initial piping channel, from $t=t_0$ till $t=t_1$. The piping channel develops from the washing out of the first soil particle, at $t=t_0$, until the development to a channel where water flows freely through at $t=t_1$. The piping channel is thus fully developed.
II. The piping channel continues to grow in diameter, but only by the erosion force of the water. At $t=t_2$ this stage ends and the channel has become so big that not only the erosional force carries soil away, but also larger parts of soil collapse from the dike body above the channel.
III. More larger parts of soil will collapse from the dike body above the piping channel. The crest will settle and water is able to overtop the dike as well. The dike body is collapsed and the strength is decreased considerably. The overtopping water will erode the dike body away. This will continue until the complete dike body is gone at $t=t_3$. Stage III ends here and from this moment on the next stages is the same as in the existing literature.
IV. This stage is the same as described in the previous paragraph. For $t_3 < t \leq t_4$, the breach will grow in lateral direction. The side slope angle is critical, γ_1. A difference with the stage described in the previous paragraph is found in the material of the dike base. Since piping occurred, the soil must be sandy and relative easily erodible, so the vertical erosion continues at a fast rate. Also, if piping occurred, it is not possible to have a type A dike, since the clay layer would have been collapsed into the piping channel.
V. This stage is also the same as in the previous paragraph. The flow is subcritical in stage V. The breach develops in the same way as in stage IV, with the difference that the growth is influenced by the backwater curve. This means that the flow velocities become smaller and the growth rate decreases. At $t=t_5$ the flow velocities are so small that the breach erosion stops.

If soil particles are washed out and the subsoil is sandy, the erosion process occurs quickly. Stages I till III can thus take place in a short time span. Large lumps of soil collapse. A difference in breach development between the failure mechanism overtopping and piping has to do with the dike base. Since piping starts in the dike base and works its way up from the dike base instead of from the crest down, the dike base will be damaged more. The scour hole will develop more easy and at the moment of $t=t_3$ the dike base will erode further. Factors that influence the piping speed are the thickness of the sand layer and the width of the dike body. The hydraulic head difference and the duration of the high water are also of importance.

Macro instability of the inner slope
Macro instability occurs if large parts of the dike body shear along a straight or curved slip plane. This shearing is caused by instability. On the soil body the following forces are active; a driving moment caused by the weight of the body itself at the left side of the circle center, a counter moment caused by the weight of the body at the right side of the circle

centre and a friction force along the slip plane. As a result of saturation of the dike body the friction forces decrease, the balance is lost and large parts of the inner slope can slip along a slip plane. Macro instability starts with a tear along the crest or slope of the dike. Next, the surface of the dike at the inner side of the tear lowers due to the moving slip circle. The location of the tear is of decisive for the further failure of the dike (GeoDelft, 2002). Large lumps of soil collapse when macro instability occurs.

The approach will be the same as in the previous paragraph. There will be stages distinguished to describe the failure due to macro instability. The steps of the failure mechanism are described in GeoDelft (2002).

Failure of the dike does not necessary starts after the first instability. After the first macro instability, micro instability, overtopping or another macro instability can occur. Assumed is that the failure is caused by macro instability and the process below starts at the instability that will make the dike fail.

I. The start of the movement of the slip circle is defined at $t=t_0$. At the beginning this process takes place slowly. At $t=t_1$ the shear off is completed.
II. After the shear off one of the dike is weakened severely and micro instability, overtopping or another macro instability will take place. This mechanism starts at $t=t_1$ and is finished at $t=t_2$. If one of these mechanisms happened, the crest is lowered and the water starts to overtop the dike.
III. Erosion of the weakened dike body starts at $t=t_2$. The overtopping water will erode the dike body away. This will continue until the complete dike body is gone at $t=t_3$. Stage III ends here and from this moment on the next stages is the same as in the existing literature.
IV. This stage is the same as described in the previous paragraph. For $t_3 < t \leq t_4$, the breach will grow in lateral direction. The side slope angle is critical, γ_1.
V. This stage is also the same as in the previous paragraph. The flow is subcritical in stage V. The breach develops in the same way as in stage IV, with the difference that the growth is influenced by the backwater curve. This means that the flow velocities become smaller and the growth rate decreases. At $t=t_5$ the flow velocities are so small that the breach erosion stops.

The main difference between the failure mode overtopping and the failure mode macro instability has to do with the damaged dike body. If a dike overtops, the damage starts with a single gully. Macro instability results in the failure of large lumps of soil, just as with the piping mechanism. Due to the (large amount of) soil that is slipped off the dike body is weakened. For this reason the erosion in stage III can take place quickly.

I.1.2 Currents in a breach

The currents through a breach vary in the time. Not only the discharge changes, also the flow pattern in the breach varies during the different stages. This has of course impact on the emergency measure that is implemented. In this paragraph the currents per breach stage are investigated and the behaviour of (large) elements in fast flowing water is discussed.

Currents per breach stage

Stage I

In the first stage the water flows over the dike crest and thereafter over the inner slope. In the beginning, the water will infiltrate in the soil. After some time the water erodes soil particles on its way and forms a channel (Dieteren & Pottinga, 1988b). The flow in stage I is displayed in Figure I.1. At the outer side of the dike the flow is subcritical where after it transforms to critical flow at the crest, with the critical depth d_c. On the slope the flow transforms to supercritical flow, with a normal depth. At the toe of the slope a hydraulic jump transforms the flow again back to subcritical.

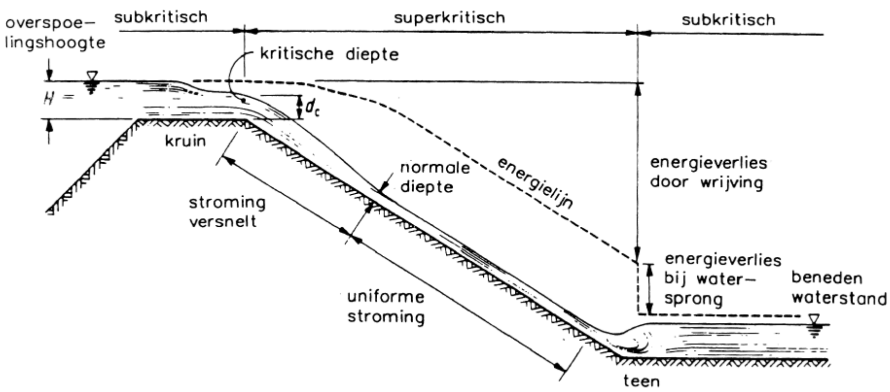


Figure I.1: Flow over a dike with low inner water level (Gerven, 2004)

Stage II and III

The discharge increases as more soil is eroded away and the channel's cross sectional area increases. The flow at the outer side of the dike remains subcritical and the flow on the slope remains supercritical. The critical flow creates a scour hole at the toe of the dike. After this point the flow becomes subcritical. This can have two reasons. The first one is that the water at the end of the scour hole falls back into the hole because the hole is too deep resulting in a subcritical flow. The other is the inner water level which has risen to a level where the hydraulic jump is pushed towards the dike toe. The hydraulic jump moves closer to the toe of the dike until it actually reaches it. The super critical flow enters the hydraulic jump at the toe, which causes turbulence (Dieteren & Pottinga, 1988b).

Stage IV

The dike body is eroded away completely. Most of the water enters the polder in this stage and the next one. In this stage the flow is virtually critical (Visser, 1998).

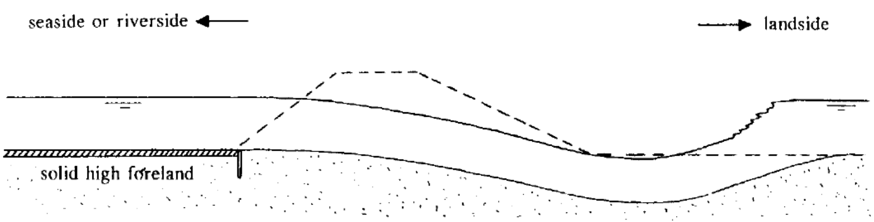


Figure I.2: Flow in a stage IV type A breach (Visser, 1998)

Stage V

At a certain point in time the water in the polder has risen to a level where it influences the incoming flow. The 'weir' becomes imperfect and the hydraulic jump disappears. The flow is subcritical (Visser, 1998).

Elements in currents

The flowing water exerts forces on the implemented emergency measures. The forces that work on an element are: drag force, lift force and shear force. The gravitational force caused by the weight and the friction force caused by the bed are the counter forces (Schiereck & Verhagen, 2012). Drag force is the force of the flowing water around the element. The drag force is in the direction of the flow. Lift force is a force in the direction normal to the flow. It is caused by an asymmetrical flow pattern due to the curvature of the element (Huis in 't Veld, 1987). If an element is stable, the lift force is countered by the weight and the drag and shear force are countered by the friction force. The drag lift and shear forces can be described by (Schiereck & Verhagen, 2012):

$$Drag\ force: F_D = \frac{1}{2} C_D \rho_w u^2 A_D \quad \text{(II-1)}$$

$$Shear\ force: F_S = \frac{1}{2} C_S \rho_w u^2 A_S \quad \text{(II-2)}$$

$$Lift\ force: F_L = \frac{1}{2} C_L \rho_w u^2 A_L \quad \text{(II-3)}$$

Because of the Reynolds number is much larger than 1 (Re >> 1), the elements in a breach can be schematized as non-streamlined element in a weak-viscous flow. For a cylinder this gives a flow pattern as in Figure I.3.

From this figure the following things can be said. There is a thin boundary layer, which is unimportant for emergency measures. Behind the object is a wake with a lot of turbulence. The flow velocity is large at the sides of the object (Battjes, 2002).

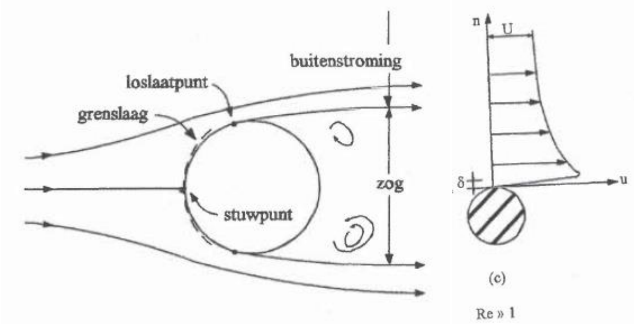


Figure I.3: Schematic flow pattern of a cylinder (Battjes,

I.2 Emergency measures

The equipment and materials that were needed to put the preventive measures in place could be useful for the employment of curative measures. The discussion about preventive measures will be short and focusses on the link to curative measures. Preventive measures that are discussed are the measures that can be used to counter the failure mechanisms overtopping, piping and macro instability of the inner slope. These are selected since the mentioned failure mechanisms are the most common, see Chapter 3.2.1.

Measure against overtopping

Overtopping occurs if the outer water level is higher than the dike crest. Water could overtop the dike at irregular intervals due to wave overtopping. This does not necessary lead to a failure of the dike. The dike height is not exactly equal everywhere. If water overtops the crest constantly, the flow will be concentrated to the parts were the dike is the lowest. At this place erosion will take place and form a deeper gap. Preventive measures for overtopping are placed on the crest of the dike. They are used to raise the retaining height of the dike. Most common possible measures are discussed below.

- Sand bags

Traditionally, the sand bag is the most used preventive measure against overtopping. Despite it is the most used system, it is labour intensive and the deployment rate per hour is rather slow. A lot of bags are needed and they are transported to the location by truck (Dillen, 2001), (Eijk, 2002).

- Water filled structures

The Twin Flex Barrier water filled tube is picked out as water filled structure since it has the best performance on the aspects of costs, deployment, stability and applicability compared to other systems. It exists of two parallel tubes that are connected to each other and are filled with water. It is possible to couple multiple elements in lateral direction. The system is placed by a tractor with trailer. It is placed by a specialized team of four people (Boon, 2007). There have been a lot of experiments in *Flood Proof Holland*. Another promising measure is the Box Barrier which is not considered in the study of Boon (2007).

- Composite barrier

Another measure to raise the retaining height of the dike is the use of a composite barrier. A composite barrier is a structure made of several materials. To set up a composite barrier, a small team of specialized people is needed. The composite barrier is transported to the location by truck (USACE, 2007), (Eijk, 2002).

Measure against piping

Piping is the washing out of soil particles by flowing water from underneath the dike body due to the hydraulic head difference on both sides of the dike. 'Pipes' are formed underneath the dike body, the pipes transport sand and the dike eventually collapses. Sensitive dikes for piping are those with a clay core and a sandy subsoil. Measures to stop the piping process are focussed on the reduction of the hydraulic head difference or the extension of the piping length.

- Containment

Piping can be seen if water (with sand) boils out at the surface behind a dike. A measure to decrease the hydraulic head difference is containment of the sand boil. Sand bags are placed around this sand boil and the water is captured inside. By this measure the driving mechanism of piping, the hydraulic head reduces (Lendering, et al., 2014).

- Piping berm

To increase the pressure on top of the surface, a berm on the inner side of the dike could be placed. This berm counters uplift, where a substantial volume of soil is lifted by the seepage flow. The function of a berm is to extend the seepage length. The berm is composed of sand or water (Boon, 2007), (Deltares, 2013).

Measure against macro instability inner slope

Macro instability occurs if large parts of the dike body shear along a straight or curved slip plane. This shearing is caused by instability. As a result of saturation of the dike body the friction forces decrease, the balance is lost and large parts of the inner slope can slip along a slip plane. A control measure could be a gentler slope, however this is not possible during an emergency situation. The preventive measure against macro stability of the inner slope is a berm.

- Berm

To counter the sliding movement a berm on the inner side of the dike can be constructed. This counterweight will prevent the soil body to shear off. This berm could again be composed of sand or water (Boon, 2007).

II. Calculations cases

II.1 Nieuwerkerk aan den IJssel, The Netherlands, Flood of 1953

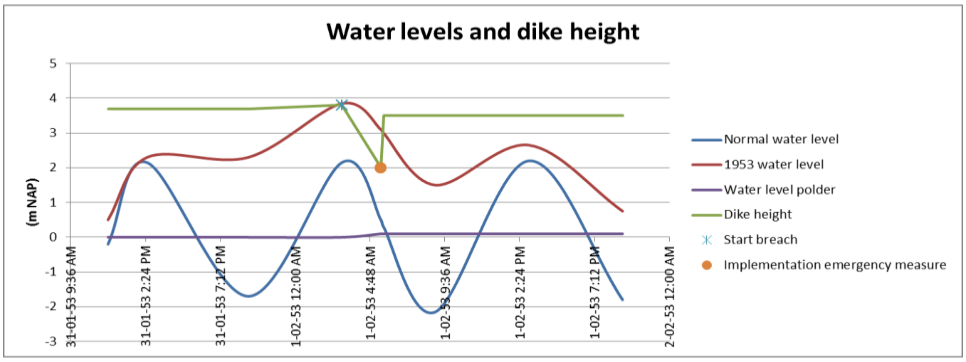


Figure II.1: Water levels and dike height Nieuwerkerk aan den IJssel

The 'normal' and 1953 water level in Figure II.1 (which is the same Figure as Figure 4.2), are derived from calculations for the expected water level and measurements during the storm surge (Rijkswaterstaat & KNMI, 1961).

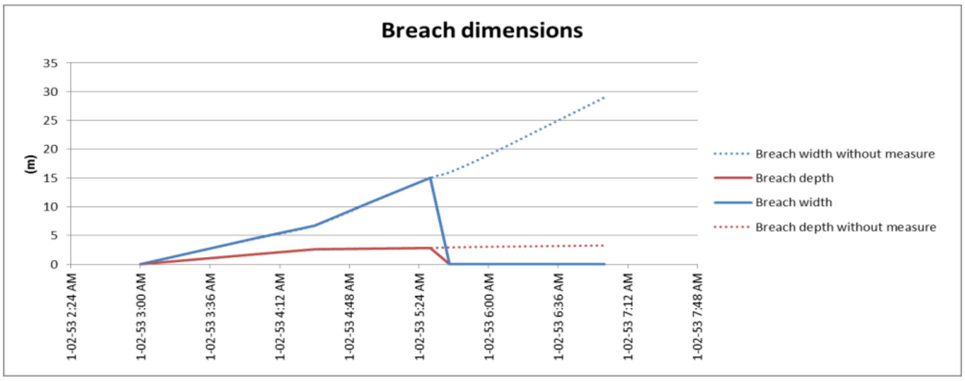


Figure II.2: Development of breach dimensions Nieuwerkerk aan den IJssel

Dimensions of the breach are presented in Figure II.2 (which is the same Figure as Figure 4.5). The time the breach was formed and the dimensions of the breach are derived from literature (Rijkswaterstaat & KNMI, 1961), (Boer, 2007). The dimensions of the breach without measures are calculated. The depth is limited due to the protected toe. This would keep the depth of the breach at a more or less fixed level. The width of the breach would, without emergency measure, be able to develop freely. This is calculated with:

$$E_{bo} = M_e(\tau_b - \tau_c) \qquad [\mathrm{m/s}] \qquad \text{(II-1 and 3-9)}$$

$$\frac{\mathrm{d}B_t}{\mathrm{d}t} = \frac{2E_{bo}}{\tan\beta_1} \qquad [\mathrm{m/s}] \qquad \text{(II-2 and 3-11)}$$

Where M_e consists the soil properties and τ_b the bed shear stress. E_{bo} is an erosion rate. Dependent on the slope of the breach side, β_1, the breach development in time can be calculated.

It was stated that the erosion rate E_{bo} can be described adequately by assuming $\tau_c = 0$ (Zhu, 2006). There could occur difficulties with this assumption if τ_b is just somewhat larger than τ_c. This however will not be the case since $\tau_b >> \tau_c$ because of the critical flow.

For this calculation, M_e needs to be assumed. There is assumed M_e = 0,0006 [s-m²/kg] (Zhu, 2006). The bed shear stress can be calculated with:

$$\tau_b = \frac{g}{C^2}\rho U^2 \qquad [\mathrm{N/m^2}] \qquad \text{(II-3)}$$

If the Chezy value is assumed at 50 $m^{1/2}/s$ and the flow velocity at 1,7 m/s, as at the closure of the breach, see Figure II.3, τ_b becomes:

$$\tau_b = \frac{9{,}81}{50^2} \cdot 1000 \cdot 1{,}7^2 = 11 \qquad [\mathrm{N/m^2}] \qquad \text{(II-4)}$$

$$E_{bo} = M_e(\tau_b - \tau_c) = 0{,}0006(11 - 0) = 6 \cdot 10^{-3} \qquad [\mathrm{m/s}] \qquad \text{(II-5)}$$

With β_1 assumed at 80°, formula (III-2) becomes:

$$\frac{\mathrm{d}B_t}{\mathrm{d}t} = \frac{2 \cdot 0{,}006}{\tan 80} = 2{,}2 \cdot 10^{-3} \qquad [\mathrm{m/s}] \qquad \text{(II-6)}$$

This calculation holds for t = 6:00 am. For t = 6:30 am and t = 7:00 am follows, with a flow velocity of 2 m/s, see Figure II.3, $E_{bo} = 8 \cdot 10^{-3}$ and $dB_t/dt = 2{,}8 \cdot 10^{-3}$.

Table II.1: Calculation of the breach width

Time	E_{bo} [m/s]	dB_t/dt [m/s]	Breach width [m]
5:30 am	-	-	15
6:00 am	$6 \cdot 10^{-3}$	$2{,}2 \cdot 10^{-3}$	19
6:30 am	$8 \cdot 10^{-3}$	$2{,}8 \cdot 10^{-3}$	24
7:00 am	$8 \cdot 10^{-3}$	$2{,}8 \cdot 10^{-3}$	29

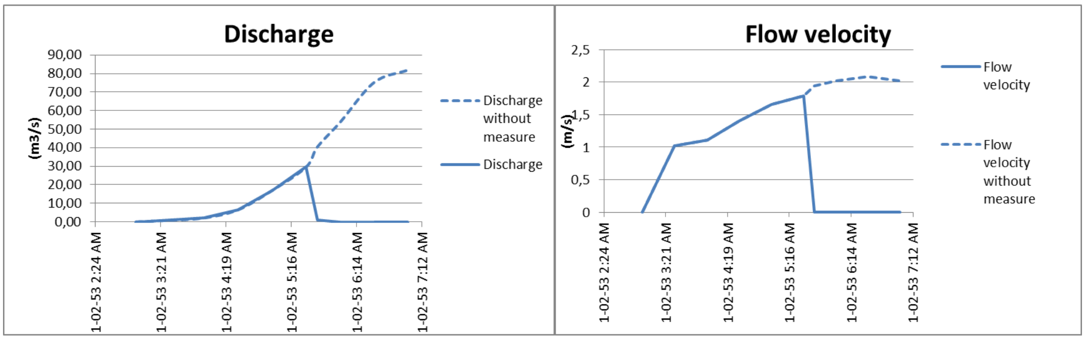


Figure II.3: Development of discharge and flow velocity

The discharge and velocity are presented in Figure II.3 (which is the same Figure as Figure 4.8). Since the water level in the polder rose just slightly, the breach didn't reached stage IV yet so formula (3-7) is applicable to calculate the discharge through the breach.

$$Q_{br} = m\left(\frac{2}{3}\right)^{3/2}\sqrt{g}B(H_w - Z_{br})^{3/2} \quad [\mathrm{m}^3/\mathrm{s}] \qquad \text{(II-7 and 3-7)}$$

In this formula, there is assumed m = 1. The breach width is calculated in Table II.1 and H_w-Z_{br} is calculated by subtracting the breach depth (Figure II.2) transformed to m NAP from the water level (Figure II.1).

$$U = \frac{Q_{br}}{Bd} \quad [\mathrm{m/s}] \qquad \text{(II-8 and 3-22)}$$

Table II.2: Calculation of discharge and flow velocity

Time	H_w - Z_{br} [m]	Q_{br} [m³/s]	U [m/s]
3:00 am	0	0	0
3:30 am	0,3	0,8	1,0
4:00 am	0,7	2,1	1,1
4:30 am	0,9	6,5	1,4
5:00 am	1,0	16,9	1,6
5:30 am	1,1	29,5	1,7
6:00 am	1,2	53,7	2,0
6:30 am	1,3	75,2	2,1
7:00 am	1,4	81,9	2,0

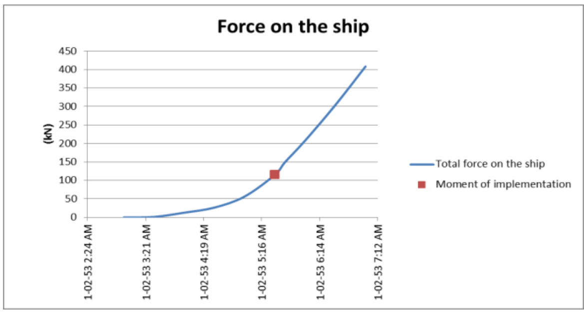


Figure II.4: Force on the ship

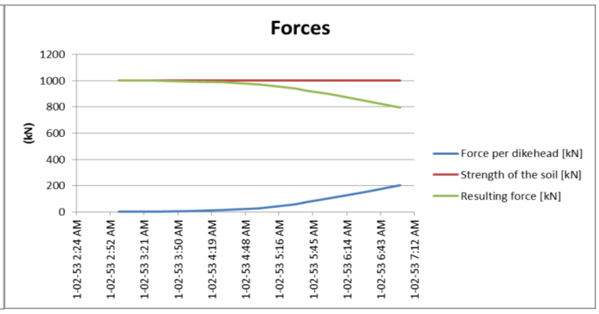


Figure II.5: Forces per dike head

The forces on the ship are presented in Figure II.4 and the forces on the dike head in Figure II.5 (which are Figure 4.9 and Figure 4.10 respectively).

$$F_{static} = \frac{1}{2}\cdot\rho\cdot g\cdot h^2\cdot B \quad [\mathrm{N}] \qquad \text{(II-9)}$$

$$F_{dynamic} = \frac{1}{2}\cdot\rho\cdot U^2\cdot A \quad [\mathrm{N}] \qquad \text{(II-10)}$$

Where h is the depth of the breach, which is equal to H_w - Z_{br}. The strength of the dike is calculated by the shear stress of the soil times the area. The shear stress of the soil is assumed at 40 kN/m² (Tol & Everts, 2007). The contact area is calculated: 25 m². This gives 1000 kN resistance. Probably the dike head is weakened because of saturation. This still would be no problem since the strength is much larger than the force of the ship.

Table II.3: Forces on the ship

Time	F_{static} [kN]	$F_{dynamic}$ [kN]	Total force on the ship [kN]
5:30 am	0	0	0
3:30 am	1	0,4	1,4
4:00 am	11	2	13
4:30 am	21	5,3	26,3
5:00 am	43	13	56
5:30 am	89	26	115
6:00 am	157	50	207
6:30 am	230	73	303
7:00 am	320	89	409

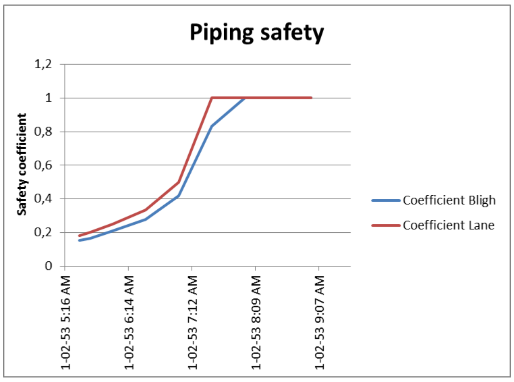


Figure II.6: Piping safety

Calculations for piping hazard are plotted in Figure II.6 (which is the same Figure as Figure 4.11). The coefficients are calculated by dividing the actual present piping length by the needed length, calculated by Bligh and Lane. This means, that when the coefficient reaches 1, the piping safety is fulfilled. In this calculation the additional measures like sandbags are not taken into account.

Bligh:

$$L = 1{,}5 \cdot C \cdot \Delta H \qquad \text{[m]} \qquad \text{(II-11)}$$

$$C_{Bligh} = 12$$

Lane:

$$L = C \cdot \Delta H \qquad \text{[m]} \qquad \text{(II-12)}$$

$$C_{Lane} = 5$$

Actual piping length Bligh: 3 m, Lane: 1 m.

Table II.4: Calculations piping safety

Time	ΔH [m]	Coeff. Bligh [-]	Coeff. Lane [-]
5:30 am	1,1	0,15	0,18
6:00 am	0,8	0,21	0,25
6:30 am	0,6	0,28	0,33
7:00 am	0,4	0,42	0,5
7:30 am	0,2	0,83	1
8:00 am	0	1	1
8:30 am	0	1	1

II.2 Jiujiang, China, Flood of 1998

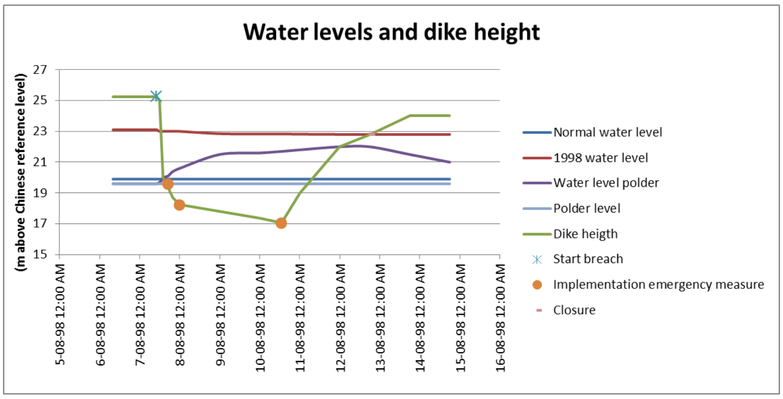


Figure II.7: Water levels and dike height Jiujiang

The 'normal' and 1998 water level in Figure II.7 (which is the same Figure as Figure 4.13), are derived from calculations for the expected water level and measurements during the flood (Chen & Li, 2000), (Anonymous, 2008), (Rage of the River Gods, 2001).

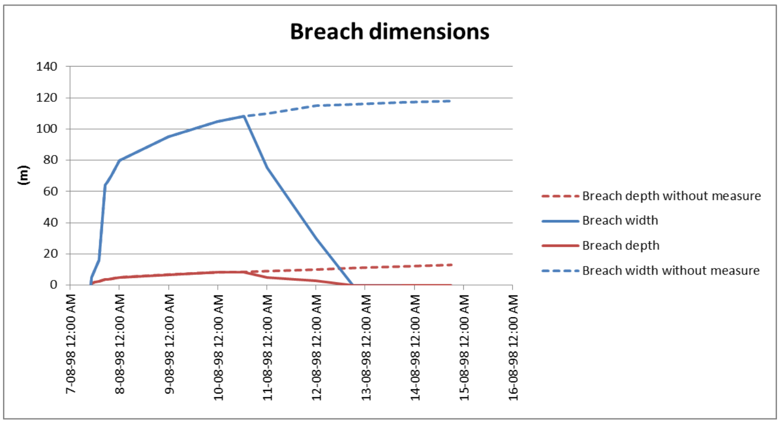


Figure II.8: Development of breach dimensions Jiujiang

Dimensions of the breach are presented in Figure II.8 (which is the same Figure as Figure 4.18). The time the breach was formed and the dimensions of the breach were derived from literature (Chen & Li, 2000), (Anonymous, 2008), (Rage of the River Gods, 2001). The dimensions of the breach without measures are calculated. In this graph the effect of the ship as emergency measure is not displayed. In Figure II.9, presenting the discharge through the breach, this emergency measure is taken into account. The width of the breach would, without emergency measure, be able to develop freely. This is calculated with:

$$E_{bo} = M_e(\tau_b - \tau_c) \qquad [\text{m/s}] \qquad \text{(II-13 and 3-9)}$$

$$\frac{\mathrm{d}B_t}{\mathrm{d}t} = \frac{2E_{bo}}{\tan\beta_1} \qquad [\text{m/s}] \qquad \text{(II-14 and 3-11)}$$

Where M_e consists the soil properties and τ_b the bed shear stress. E_{bo} is an erosion rate. Dependent on the slope of the breach side, β_1, the breach development in time can be calculated.

It was stated that the erosion rate E_{bo} can be described adequately by assuming $\tau_c = 0$ (Zhu, 2006). There could occur difficulties with this assumption if τ_b is just somewhat larger than τ_c. This however will not be the case since $\tau_b >> \tau_c$ because of the critical flow.

For this calculation, M_e needs to be assumed. There is assumed M_e = 0,00018 [s-m²/kg] (Zhu, 2006). The bed shear stress can be calculated with:

$$\tau_b = \frac{g}{C^2}\rho U^2 \qquad [\text{N/m}^2] \qquad \text{(II-15)}$$

If the Chezy value is assumed at 50 $m^{1/2}$/s and the flow velocity at 0,4 m/s, as at the closure of the breach, see Figure II.10, τ_b becomes:

$$\tau_b = \frac{9{,}81}{50^2} \cdot 1000 \cdot 0{,}4^2 = 0{,}6 \qquad [\text{N/m}^2] \qquad \text{(II-16)}$$

$$E_{bo} = M_e(\tau_b - \tau_c) = 1{,}8 \cdot 10^{-4}(0{,}6 - 0) = 1{,}1 \cdot 10^{-4} \quad [\text{m/s}] \qquad \text{(II-17)}$$

With β_1 assumed at 80°, formula (II-14) becomes:

$$\frac{dB_t}{dt} = \frac{2 \cdot 1{,}1 \cdot 10^{-4}}{\tan 80} = 3{,}8 \cdot 10^{-5} \qquad [\text{m/s}] \qquad \text{(II-18)}$$

This calculation holds for 10-8-1998 12:00 am until 13-8-1998 12:00 am. For 14-8-1998 12:00 am follows, with a flow velocity of 0,2 m/s, see Figure II.10, $E_{bo} = 3{,}4 \cdot 10^{-5}$ and $dB_t/dt = 1{,}2 \cdot 10^{-4}$.

Table II.5: Calculation of the breach width

Time	E_{bo} [m/s]	dB_t/dt [m/s]	Breach width [m]
10-8-1998 24:00	-	-	108
11-8-1998 24:00	$1{,}1 \cdot 10^{-4}$	$3{,}8 \cdot 10^{-5}$	111
12-8-1998 24:00	$1{,}1 \cdot 10^{-4}$	$3{,}8 \cdot 10^{-5}$	114
13-8-1998 24:00	$1{,}1 \cdot 10^{-4}$	$3{,}8 \cdot 10^{-5}$	117
14-8-1998 24:00	$3{,}4 \cdot 10^{-5}$	$1{,}2 \cdot 10^{-5}$	118

The development of the breach depth is difficult to calculate. The soil of the dike base is unknown and very uncertain. For this reason the depth of the breach is assumed.

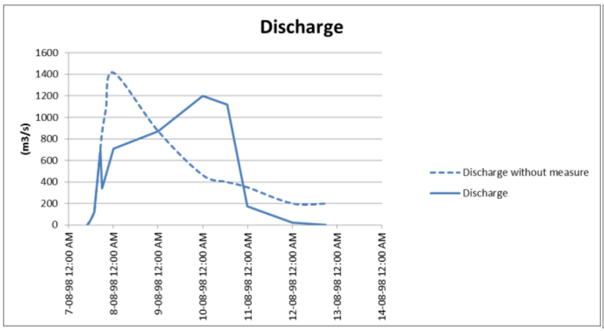


Figure II.9: Discharge through the breach Jiujiang

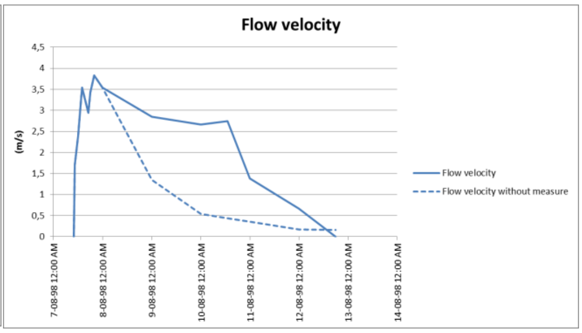


Figure II.10: Flow velocity through the breach Jiujiang

The discharge is calculated by two formulas. This is the case since the water level in the polder is of influence for the discharge at a certain point. The discharge for breach stages I to IV can be calculated with formula (II-18). In this formula the water level in the polder does not influence the discharge. The discharge in breach stage V can be calculated with formula (II-19). In this formula the water level in the polder does influence the discharge. The same holds for the velocities calculated with formulas (II-20) and (II-21)

$$Q_{br} = m\left(\frac{2}{3}\right)^{3/2}\sqrt{g}B(H_w - Z_{br})^{3/2} \quad [m^3/s] \qquad \text{(II-19 and 3-7)}$$

$$Q_{br} = m\sqrt{2g}B\left(H_w - H_p\right)^{1/2}\left(H_p - Z_{br}\right) \quad [m^3/s] \qquad \text{(II-20 and 3-21)}$$

In this formula, there is assumed m = 1. The breach width is calculated in Table II.5 and H_w-Z_{br} is calculated by subtracting the breach depth (Figure II.8) transformed to m Chinese reference level from the water level (Figure II.7).

$$U = \frac{Q_{br}}{Bd} \quad [m/s] \qquad \text{(II-21 and 3-22)}$$

$$U = \sqrt{2g(H_w - H_p)} \quad [m/s] \qquad \text{(II-22 and 3-23)}$$

Table II.6: Discharge with emergency measures

Time	H_w - Z_{br} [m]	Breach width including vessel [m]	Q_{br} [m³/s]
7-8-1998 10:00	0	0	0
7-8-1998 14:00	3	16	123
7-8-1998 17:00	3,7	64	696
7-8-1998 18:00	3,9	26 (implementation vessel)	339
7-8-1998 20:00	4,3	30	459
8-8-1998 24:00	4,8	40	708
9-8-1998 24:00	5,1	55	870
10-8-1998 24:00	5,5	65	1118 (III-19)
11-8-1998 24:00	3,8	25 (enclosing scaffold)	173
12-8-1998 24:00	0,8	10	20
12-8-1998 18:00	0	0	0

Table II.7: Discharge without emergency measures

Time	H_w - Z_{br} [m]	Breach width [m]	Q_{br} [m³/s]
7-8-1998 10:00	0	0	0
7-8-1998 14:00	3	16	123
7-8-1998 17:00	3,7	64	696
7-8-1998 18:00	3,9	66	859
7-8-1998 20:00	4,3	70	1071
8-8-1998 24:00	4,8	80	1416
9-8-1998 24:00	5,1	95	873 (III-19)
10-8-1998 24:00	5,5	105	463
11-8-1998 24:00	6	110	402
12-8-1998 24:00	6,3	115	351
12-8-1998 18:00	6,6	116	211

II.3 New Orleans, United States, Hurricane Katrina, 2005

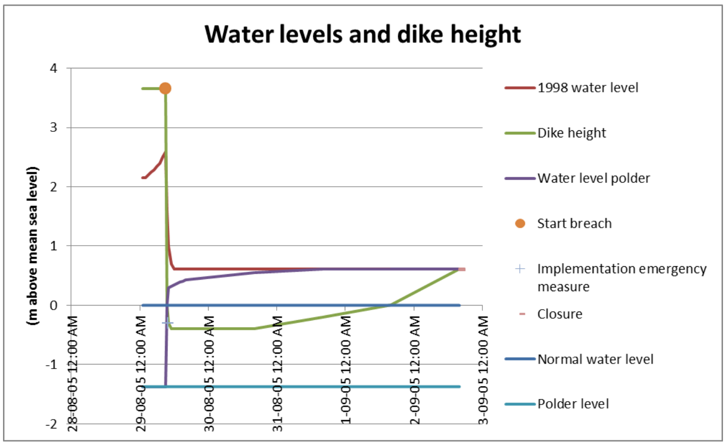
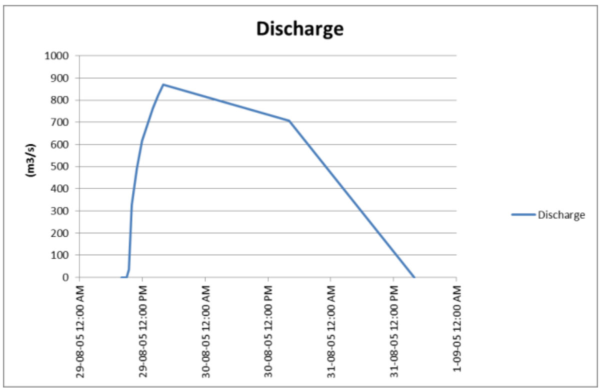


Figure II.11: Water level and dike height New Orleans

The 'normal' and 1998 water level in Figure II.11 (which is the same Figure as Figure 4.27), are derived from calculations for the expected water level and measurements during the flood (Seed, et al., 2008c), (Chaudhry, et al., 2010). The same holds for the breach dimensions. If they could not be found, they were assumed. Attempts to close the breach failed. The breach development could for this reason take place in an undisturbed way.

Table II.8: Breach width New Orleans

Time	Breach width [m]	Breach depth [m]
29-8-2005 9:00	0	0
29-8-2005 9:30	10	3,6
29-8-2005 10:00	20	3,8
29-8-2005 11:00	40	3,8
29-8-2005 12:00	60	3,8
29-8-2005 13:00	70	3,8
29-8-2005 14:00	80	3,8
29-8-2005 16:00	100	3,8
30-8-2005 16:00	137	3,8
31-8-2005 16:00	137	3,8



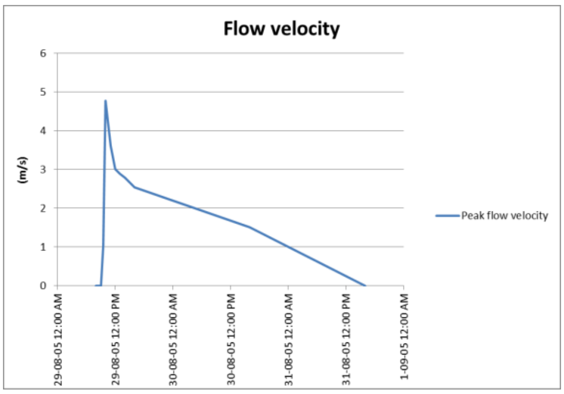


Figure II.12: Development discharge and flow velocity New Orleans

The discharge is calculated by two formulas. This is the case since the water level in the polder is of influence for the discharge at a certain point. The discharge for breach stages I to IV can be calculated with formula (II-22). In this formula the water level in the polder does not influence the discharge. The discharge in breach stage V can be calculated with formula (II-23). In this formula the water level in the polder does influence the discharge. The same holds for the velocities calculated with formulas (II-24) and (II-25).

$$Q_{br} = m\left(\frac{2}{3}\right)^{3/2}\sqrt{g}B(H_w - Z_{br})^{3/2} \quad [\mathrm{m^3/s}] \qquad \text{(II-23 and 3-7)}$$

$$Q_{br} = m\sqrt{2g}B\left(H_w - H_p\right)^{1/2}\left(H_p - Z_{br}\right) \quad [\mathrm{m^3/s}] \qquad \text{(II-24 and 3-21)}$$

In this formula, there is assumed m = 1. The breach width is presented in Table II.8 and H_w-Z_{br} is calculated by subtracting the breach depth (Figure II.11) transformed to m above mean sea level from the water level (Figure II.11).

$$U = \frac{Q_{br}}{Bd} \quad [\mathrm{m/s}] \qquad \text{(II-25 and 3-22)}$$

$$U = \sqrt{2g(H_w - H_p)} \quad [\mathrm{m/s}] \qquad \text{(II-26 and 3-23)}$$

$$U_{peak} = 1.1 \cdot U \quad [\mathrm{m/s}] \qquad \text{(II-27)}$$

Table II.9: Calculation of the discharge New Orleans

Time	H_w - Z_{br} [m]	Breach width [m]	Q_{br} [m³/s]	U [m/s]	U_{peak} [m/s]
29-8-2005 9:00	0	0	0	0	0
29-8-2005 9:30	1,6	10	35	0,94	1,1
29-8-2005 10:00	1,3	20	326 (III-23)	4,3	4,8
29-8-2005 11:00	1,1	40	494	3,2	3,6
29-8-2005 12:00	1,0	60	617	2,7	3,0
29-8-2005 13:00	1,0	70	694	2,6	2,9
29-8-2005 14:00	1,0	80	763	2,5	2,8
29-8-2005 16:00	1,0	100	821	2,3	2,7
30-8-2005 16:00	1,0	137	706	1,4	1,5
31-8-2005 16:00	0,81	137	0	0	0

The Big Bags applied in New Orleans did not have any effect since they were washed away by the strong currents. The forces on the Big Bags and the stability are calculated for several sizes.

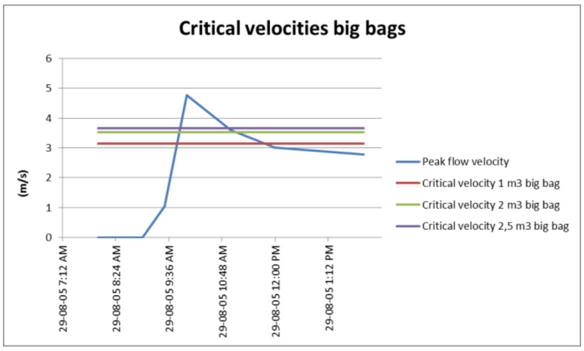


Figure II.13: Critical velocities on Big Bags

The critical velocities of the Big Bags are calculated with the Izbash formula. Izbash is applicable since it considers forces on individual 'stones'.

$$u_c = 1{,}2 \cdot \sqrt{2\Delta g d} \qquad [\text{m/s}] \qquad \text{(II-28)}$$

Table II.10: Calculation critical flow velocity

Size big bag	d_n [m]	u_c [m/s]
1 m^3 (1350 kg)	1,0	3,1
2 m^3 (2700 kg)	1,26	3,5
2,5 m^3 (3200 kg)	1,36	3,7

II.4 Fischbeck, Germany, Flood of 2013

The 'normal' and 2013 water level in Figure II.14 (which is the same Figure as Figure 4.37), are derived from calculations for the expected water level and measurements during the flood (Delft University of Technology & Technische Universitat Dresden, 2013). The same holds for the breach dimensions. If they could not be found, they were assumed. Attempts to close the breach failed.

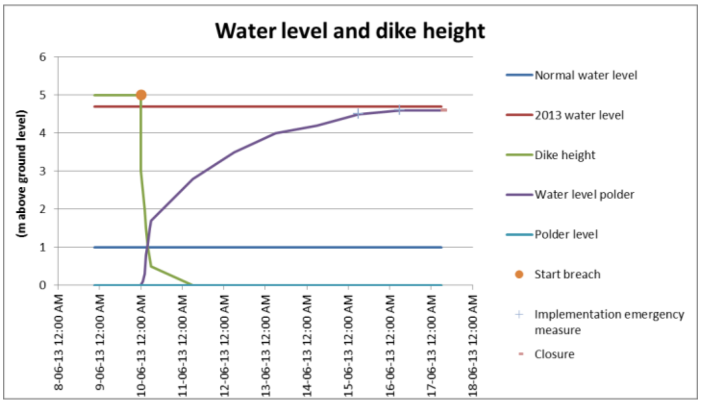


Figure II.14: Water levels and dike height Fischbeck

The breach dimensions are displayed in Table II.11.The breach was already fully developed. Without emergency measures the breach width would remain 100 m and the depth 5 m.

Table II.11: Breach dimensions Fischbeck

Time	Breach width [m]	Breach depth [m]
9-6-2013 23:00	0	0
10-6-2013 00:00	50	2
10-6-2013 01:00	65	2,5
10-6-2013 02:00	75	3
10-6-2013 03:00	85	3,5
10-6-2013 04:00	90	4
10-6-2013 05:00	95	4,3
10-6-2013 06:00	100	3,5
11-6-2013 06:00	100	5
14-6-2013 06:00	100	5
15-6-2013 06:00	50	5
16-6-2013 06:00	20	5
17-6-2013 06:00	0	0

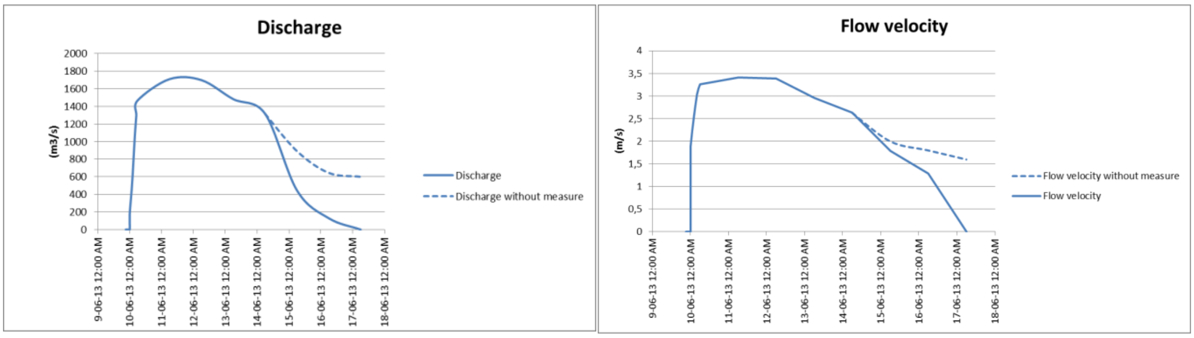


Figure II.15: Development of discharge and flow velocity Fischbeck

The discharge is calculated by two formulas. This is the case since the water level in the polder is of influence for the discharge at a certain point. The discharge for breach stages I to IV can be calculated with formula (II-28). In this formula the water level in the polder does not influence the discharge. The discharge in breach stage V can be calculated with formula (III-29). In this formula the water level in the polder does influence the discharge. The same holds for the velocities calculated with formulas (II-24) and (II-25).

$$Q_{br} = m\left(\frac{2}{3}\right)^{3/2} \sqrt{g}B(H_w - Z_{br})^{3/2} \qquad [m^3/s] \qquad \text{(II-29 and 3-7)}$$

$$Q_{br} = m\sqrt{2g}B\left(H_w - H_p\right)^{1/2}\left(H_p - Z_{br}\right) \qquad [m^3/s] \qquad \text{(II-30 and 3-21)}$$

In this formula, there is assumed m = 1. The breach width is presented in Table II.11 and H_w-Z_{br} is calculated by subtracting the breach depth (Figure II.14) transformed to m above mean sea level from the water level (Figure II.14).

$$U = \frac{Q_{br}}{Bd} \qquad [m/s] \qquad \text{(II-31 and 3-22)}$$

Table II.12: Discharge Fischbeck

Time	H_w - Z_{br} [m]	Q_{br} [m³/s]	U [m/s]
9-6-2013 23:00	0	0	0
10-6-2013 00:00	1,7	188	1,9
10-6-2013 01:00	2,2	361	2,2
10-6-2013 02:00	2,7	567	2,5
10-6-2013 03:00	3,2	829	2,8
10-6-2013 04:00	3,7	1092	3,0
10-6-2013 05:00	4	1296	3,2
10-6-2013 06:00	4,2	1467	3,3
11-6-2013 06:00	4,7	1709	3,4
14-6-2013 06:00	4,7	1315	2,6
15-6-2013 06:00	4,7	445	1,7
16-6-2013 06:00	4,7	128	1,3
17-6-2013 06:00	0	0	0

Without emergency measures the discharge and velocity will decrease too, since the water level in the polder will come close to the outer water level. However, this will happen somewhat later in time.

The critical velocities of the Big Bags are calculated with the Izbash formula. Izbash is applicable since it considers forces on individual 'stones'.

$$u_c = 1{,}2 \cdot \sqrt{2\Delta g d} \qquad \text{[kN]} \qquad \text{(II-32)}$$

The density of the ships is assumed at 1100 kg/m^3. This is done because if the ships are just sunk down, they are not filled with big bags yet. The overall density of a ship will be just above that of water.

The diameter of the ships is calculated by taking the third power root of the volume of the ship.

Table II.13: Calculation critical flow velocity

Size Ship (lxbxh) [m]	d_n [m]	u_c [m/s]
50 x 4 x 4	9,3	5,1
20 x 3 x 3	5,6	4,0
40 x 3 x 4	7,8	4,7

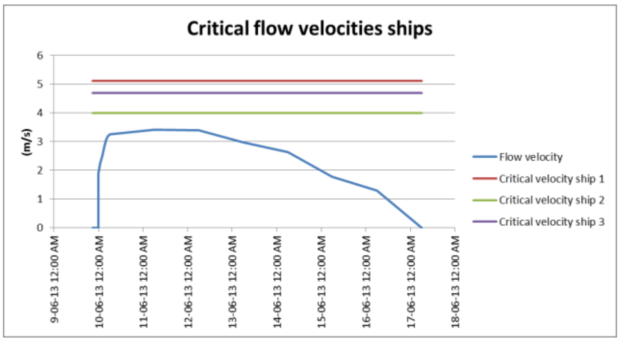


Figure II.16: Critical velocity of the ships

The critical velocity of the ships stays above the real velocity, explaining the stability of the ships in the high velocity flow.

III. Modelling files

III.1 XBeach

III.1.1 Parameters

The parameters used for the XBeach model are presented below.

<u>grid input</u>
nx = 135
ny = 100
dx = 100
dy = 100
xori = 0.
yori = 0.
alfa = 0.
depfile = zbnew
xfile = xnew
yfile = ynew
posdwn = -1
vardx = 1

<u>wave input</u>
swave = 0
hmin = 0.001
rho = 1025
g = 9.81
instat= 0

<u>flow input</u>
zsinitfile = zsoinitial.zs
zsofile = zsoinput.dat
tideloc = 1
paulrevere = 0
tidelen = 68
front = 0
left = 1
right = 1
back = 0
C = 65.
eps = 0.005
umin = 0.0
tstart= 0
tint = 10.
tstop = 3900.

CFL = 0.7
smag = 1
nuh = 0.15
nuhfac= 0.0

<u>sed input</u>
dico = 1.
D50 = 0.0003
D90 = 0.00045
rhos = 2650
morfac = 1
facsl = 1.6
wetslp = 0.3
dryslp = 1
tsfac = 0.1
hswitch = 0.1
bed = 0
struct = 1
ne_layer = ne.input

outputformat = netcdf

nglobalvar = 4
zb
zs
u
v

III.1.2 Grid

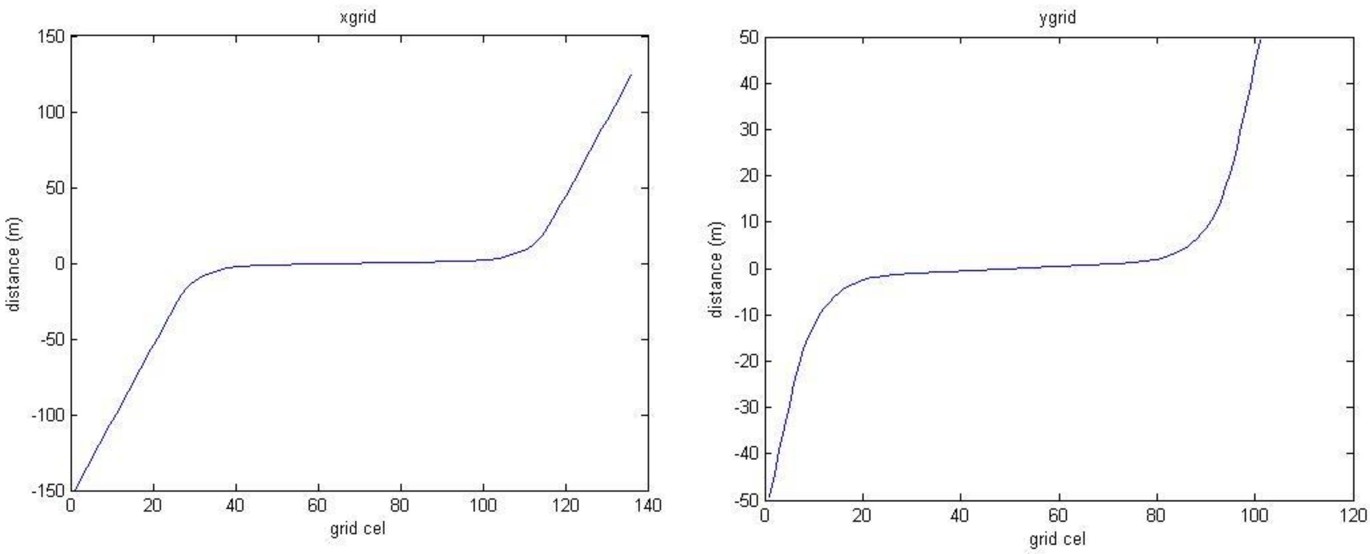


Figure III.1: Grid composition

Figure III.1 displays the distances between the grid cells for the x and y direction.

III.2 Matlab

This script is written to visualize the output of the XBeach model.

```
% This script plots the results of the XBeach model of the breaching
% process in a sand dike and the interaction with emergency closure
% measures. The measures are modelled as non-erodible layers.
%
% The computations are based on the XBeach Zwin case model tested by
% Deltares to validate XBeach for breaching.

clear all; close all; clc

% Load Toolbox into Matlab
addpath('D:\albers\Documents\CIE5060-09 MSc Thesis\4. Thesis report\7.
XBeach\XBeach\Checkout\')
oetsettings

% Specify output location
dirs.output = 'D:\albers\Documents\CIE5060-09 MSc Thesis\4. Thesis
report\7. XBeach\XBeach\Zwin\4. Dijk 1op3 2.7 NE scaffold\Output\';

% Read dimensions
d        = xb_read_dims;

% Output times for Figure 1
t =      find(ismember(d.t/60, [0 8 15 20 40 65]));

% Read XBeach output
xbo      = xs_peel(xb_read_output(pwd, 'vars', {'zb' 'zs' 'u' 'v'}));

% Compute breach depth
Zt = [];

for i = 1:d.globaltime
    Zt(i)    = (min(min(min(xbo.zb(i,47:55,59:83))))-2.5);
end;

% Compute breach width
Bt = [];

for i = 1:d.globaltime
    Bt(i)    = min(d.y(squeeze(xbo.zb(i,:,:))>3.29&abs(d.x)<20&d.y>=0)) -
 ...
              max(d.y(squeeze(xbo.zb(i,:,:))>3.29&abs(d.x)<20&d.y<=0));
end

% Figure properties
figure;
set(gcf,'color','w');

for i = 1:length(t)

    switch i
        case 1
            s1 = subplot(231);
```

```
        case 2
            s1 = subplot(232);
        case 3
            s1 = subplot(233);
        case 4
            s1 = subplot(234);
        case 5
            s1 = subplot(235);
        case 6
            s1 = subplot(236);
    end

    subplot(s1);

    b        =
surf(d.x(17:85,40:110),d.y(17:85,40:110),squeeze(xbo.zb(t(i),17:85,40:11
0))); material dull; hold on;
    s        =
surf(d.x(17:85,40:110),d.y(17:85,40:110),squeeze(xbo.zs(t(i),17:85,40:11
0))); shading interp;

    axis([-50 100 -50 50 -3 4]);
    daspect([10 10 1]);

    set(gca,'linewidth',2');
    set(b,'facecolor',[1 .8 0]);
    set(s,'facecolor','b','facealpha',0.3);

    material shiny;
    daspect([10 10 1]); caxis([-5 5]);

    camorbit(85,0);
    camlight;
    lighting phong;

end

% Write figure
xb_write_plot(gcf, dirs.output, 'fig2');

figure;

subplot(411);

plot(   d.t/60,squeeze(xbo.zs(:,d.ny/2+1,35)),'k-',  ...
        d.t/60,squeeze(xbo.zs(:,d.ny/2+1,70)),'b-',  ...
        d.t/60,squeeze(xbo.zs(:,d.ny/2+1,103)),'r-', ...
        'linewidth',2)

xlabel('time (min)');
ylabel('water level (m)')
legend('Upstream','Breach','Downstream','location','bestoutside')
axis([0 70 0 3]);

subplot(412);
```

```
plot(    d.t/60,squeeze(xbo.u(:,d.ny/2+1,35)),'k-',    ...
         d.t/60,squeeze(xbo.u(:,d.ny/2+1,70)),'b-',    ...
         d.t/60,squeeze(xbo.u(:,d.ny/2+1,103)),'r-',    ...
         'linewidth',2)

xlabel('time (min)');
ylabel('velocity (m/s)')
legend('Upstream','Breach','Downstream','location','bestoutside')
axis([0 70 0 6]);

subplot(413);

plot(    d.t/60,Bt,'k-',                    ...
         'linewidth',2)

xlabel('time (min)');
ylabel('breach width (m)')
legend('B comp','location','bestoutside')
axis([0 70 0 60]);

subplot(414);

plot(    d.t/60,Zt,'k-',                    ...
         'linewidth',2)

xlabel('time (min)');
ylabel('breach depth (m)')
legend('Z comp','location','bestoutside')

xb_write_plot(gcf, dirs.output, 'fig1');

% Make animation
opengl software;
f = figure();


vidObj = VideoWriter('zwinNE.avi');
vid.FrameRate = 10;
    open(vidObj);
for j = 1:390

    bb =
surf(d.x(17:85,40:110),d.y(17:85,40:110),squeeze(xbo.zb(j,17:85,40:110))
); material dull; hold on;
    ss =
surf(d.x(17:85,40:110),d.y(17:85,40:110),squeeze(xbo.zs(j,17:85,40:110))
,'facealpha',0.1); shading interp;
    set(bb,'facecolor',[1 .8 0]);
    set(ss,'facecolor','b','facealpha',0.3);
    camorbit(80,0);
    camlight;
    lighting phong;
    axis ([-40 90 -40 40 -2 4]);%vis3d;%
    daspect([10 10 1]);
    caxis([-5 5]);
    hold off
```

```
        currFrame = getframe(f);
        writeVideo(vidObj,currFrame);
end
close(vidObj);

% Animation differences
xb1 = xb_read_output('D:\albers\Documents\CIE5060-09 MSc Thesis\4.
Thesis report\7. XBeach\XBeach\Zwin\1. Zwin Origineel\Input');
xb2 = xb_read_output();

zb1 = xs_get(xb1, 'zb');
zb2 = xs_get(xb2, 'zb');

figure;
subplot(211)
pcolor(squeeze(zb2(150,:,:)-zb1(150,:,:)));
shading interp;
caxis([-3 3]);
colorbar;
title('Differences with scenario without emergency measures (m)')

subplot(212)
pcolor(squeeze(zb2(391,:,:)-zb1(391,:,:)));
shading interp;
caxis([-3 3]);
colorbar;
xb_write_plot(gcf, dirs.output, 'fig3');
```

IV. Complete closure strategies

IV.1 Strategy A

In this strategy the emergency dike is started with two arms perpendicular to the real dike until a height of + 1 m above ground level. Thereafter, the part between the arms with a height of + 1 m above ground level is implemented. After the closure up to a height of + 1 m, the first implementation of the arms is repeated only now until a height above the water level. The last layer is the complete closure of the emergency dike. This strategy is simulated below.

Phase 0

Phase 0 is defined as the phase before implementation of the emergency dike. Assumed is that after 20 minutes of non-interfered breach development the emergency measure is implemented. After the implementation phase 1 starts.

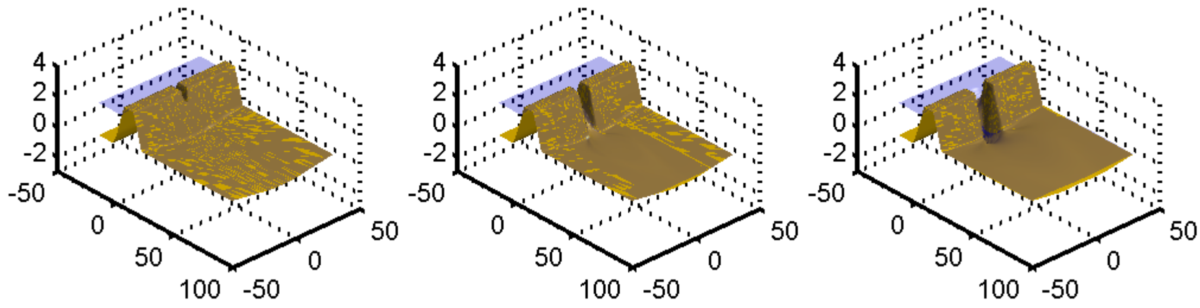


Figure IV.1: Strategy A, Phase 0, snapshots taken at 0, 15 and 20 minutes

Phase 1

In the first phase the two arms are placed. Both placed perpendicular to the dike. This is done because there is assumed that the emergency dike can not be placed at ones. The layer placed in phase 1 and the effects are visible in Figure IV.2. 15 minutes later, layer 2 is placed.

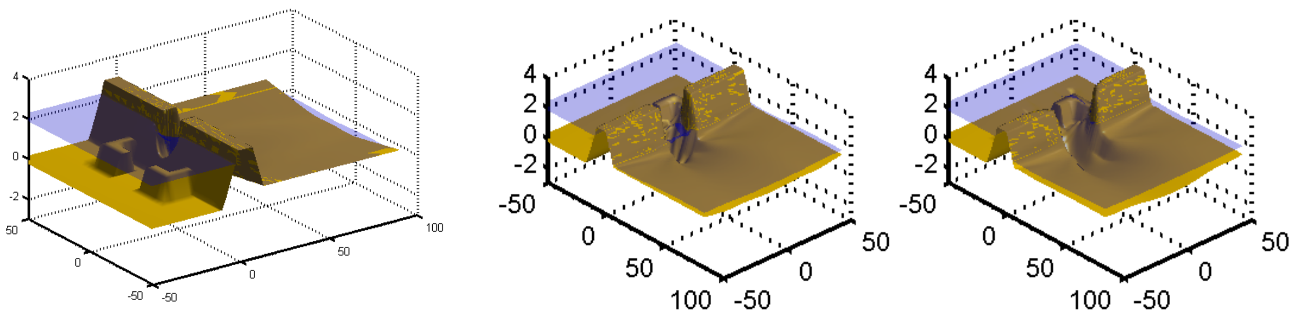


Figure IV.2: Strategy A, Phase 1 emergency dike and snapshots at 28 and 35 minutes

Phase 2

Next, the emergency dike is completed until a height of + 1 m above the surrounding ground level. Figure IV.3 shows the emergency dike placed in phase 2 and the effect of this layer. 15 minutes later layer 3 is placed.

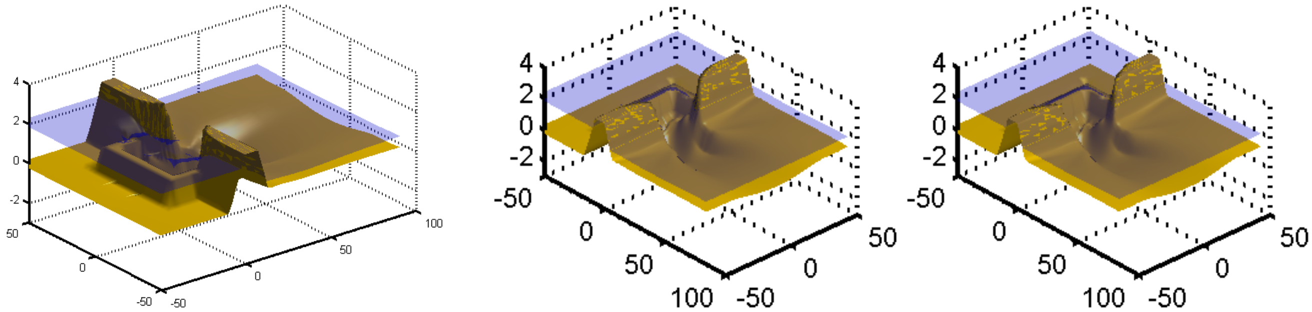


Figure IV.3: Strategy A, Phase 2 emergency dike and snapshots at t = 43 and 50 min

Phase 3

This phase is comparable to the shape of phase 1, the level of the two arms is now raised above the water level. The left plot in Figure IV.4 shows the emergency dike at phase 3 and two snapshots. After 15 minutes layer 4 is placed.

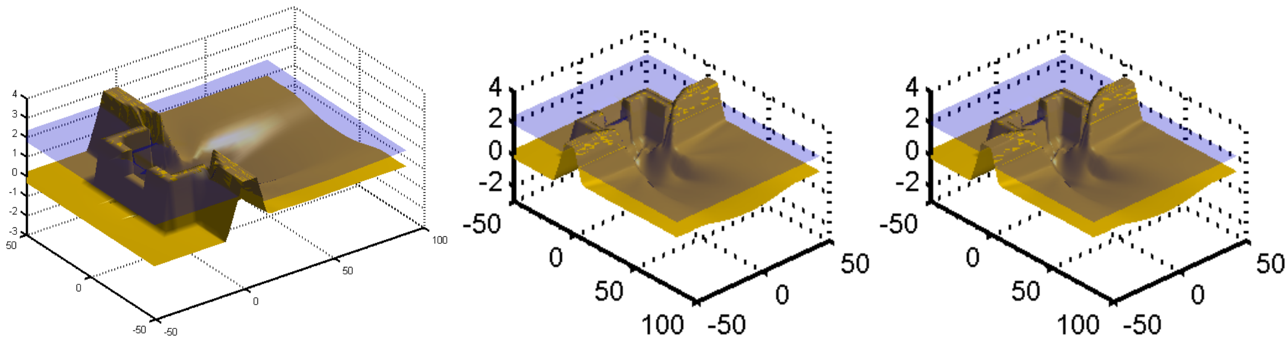


Figure IV.4: Strategy A, Phase 3, emergency dike and snapshots at t = 58 and 65 min

Phase 4

This phase is comparable to the shape of phase 2, however, the emergency dike emerges completely above the water level and a closure is made. In Figure IV.5 the completed emergency dike is visible. The snapshots are exactly the same since no more water is flowing through the breach.

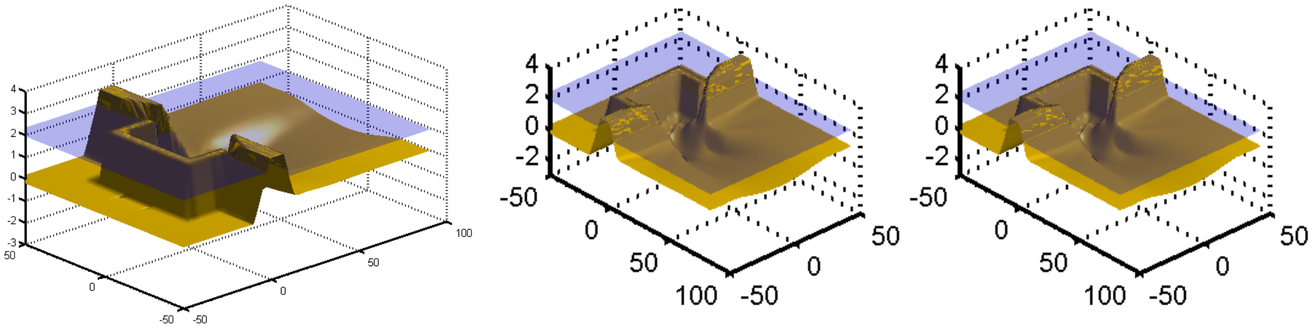


Figure IV.5: Strategy A, Phase 4 emergency dike and snapshots at t = 73 and 80 minutes

Results and discussion method A
The performance of the complete closure with the emergency dike is plotted in Figure IV.6 against the 'do nothing' scenario.

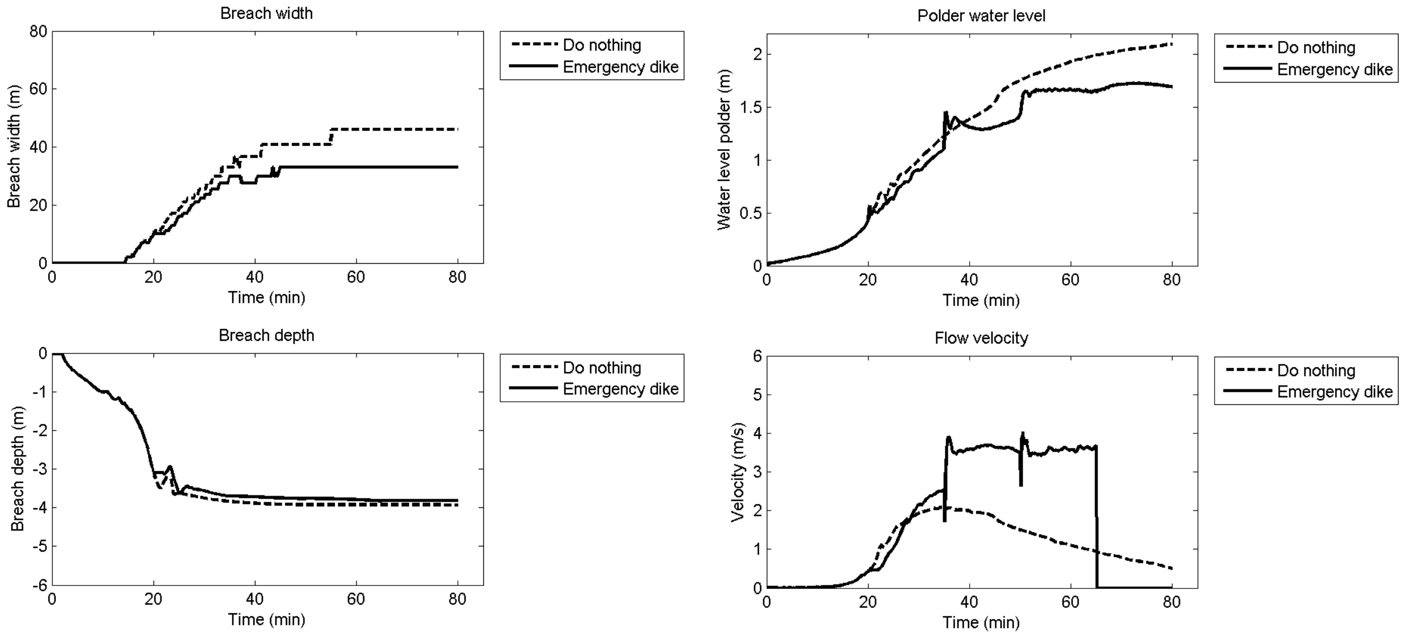


Figure IV.6: Comparison of complete closure with strategy A to 'do nothing'

There can be stated that the breach width is reduced due to the implementation of an emergency dike. The breach depth is however almost the same. The water level in the polder is lowered due to the emergency measure. As already predicted, the velocity increases at the place where the emergency dike is placed. Two remarks can be made when looking at these plots. At first sight the implementation of the emergency dike looks like spilled effort. The effect of the measure on the breach dimensions, water level and velocity is little. However, this simulation is done for a sand dike. The breach develops fast and the discharge too. Besides the polder area is relatively small. This was chosen originally to see the effect of emergency measures on all breach stages. More realistic and impressive results could be obtained by increasing this area in a next simulation. There must be kept in mind that the velocities will be higher and the breach is more difficult to close. The second remark is about the jumps in the plots of the water level and the velocity. The results of the separate runs were combined to one complete closure. Because runs needed to be done for every layer separately and the implemented layers are quite large, jumps are produced by the model. This can be improved by a dynamic non-erodible layer in XBeach which changes in time.

IV.2 Strategy B

In this strategy the emergency dike is implemented in reversed order. There is started with the part of the emergency dike right in front of the breach. Next, the two arms perpendicular to the real dike until a height of + 1 m above ground level are placed. After the closure up to a height of + 1 m, the first implementation of the part in front of the breach is repeated. The last layer is the complete closure of the emergency dike by placing the arms perpendicular to the real dike to a level above the water level. This strategy is simulated below.

Phase 0

Phase 0 is defined as the phase before implementation of the emergency dike. Assumed is that after 20 minutes of non-interfered breach development the emergency measure is implemented. After the implementation phase 1 starts.

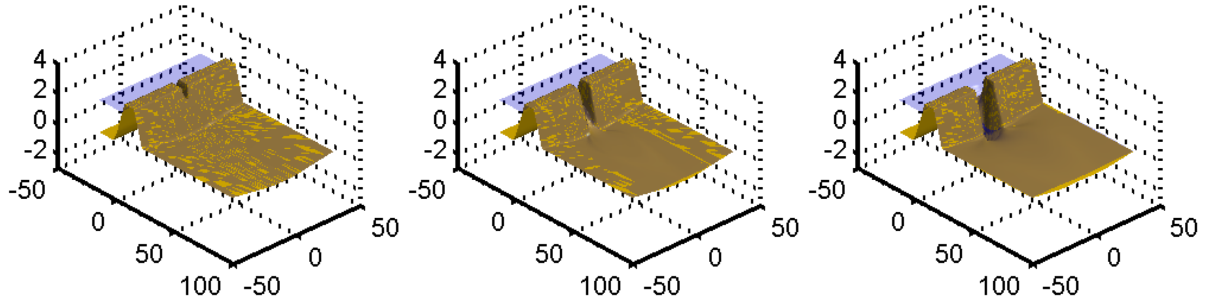


Figure IV.7: Strategy B, Phase 0, snapshots taken at t = 0, 15 and 20 minutes

Phase 1

In the first phase the emergency dike is placed until a height of + 1 m above the surrounding ground level in front of the breach. Figure IV.8 shows the emergency dike placed in phase 1 and the effect of this layer. After 15 minutes layer 2 is placed.

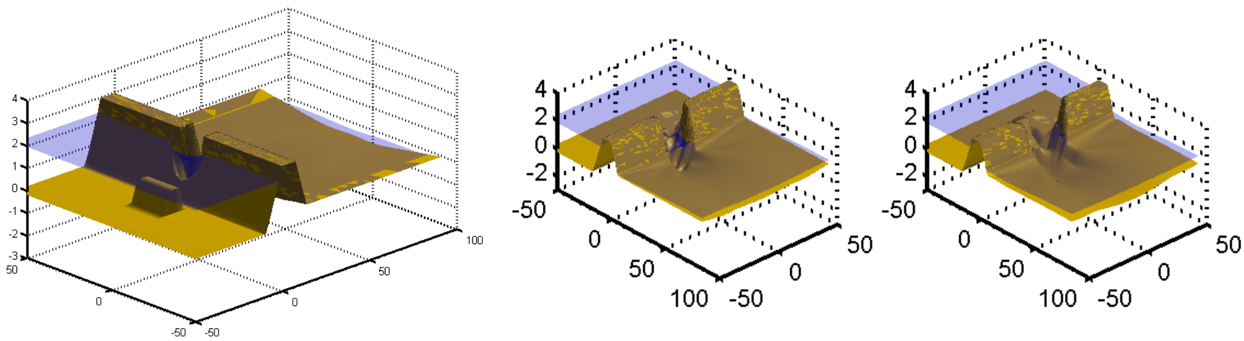


Figure IV.8: Strategy B, Phase 1, snapshots taken at t = 28 and 35 minutes

Phase 2

Next, the emergency dike is completed until a height of + 1 m above the surrounding ground level. Figure IV.9 shows the emergency dike placed in phase 2 and the effect of this layer. After 15 minutes layer 3 is placed.

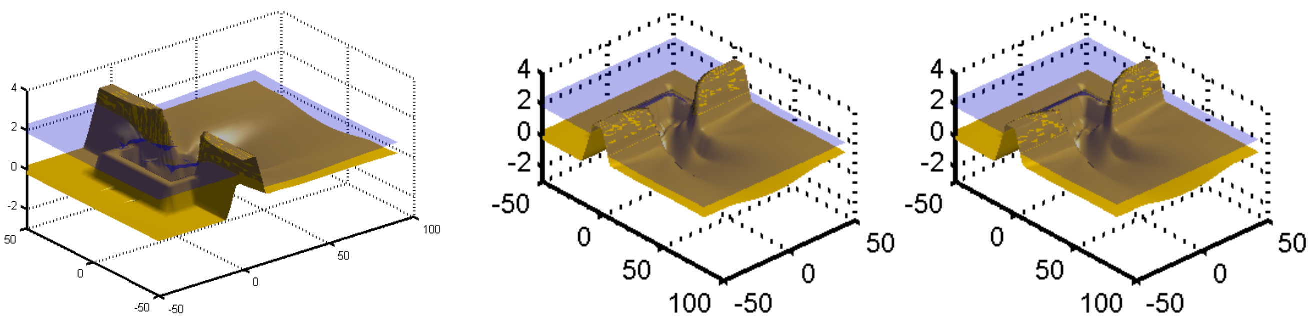


Figure IV.9: Strategy B, Phase 2 emergency dike and snapshots at t = 43 and 50 min

Phase 3

This phase is comparable to the shape of phase 1, the level of the emergency dike in front of the breach is now raised above the water level. The left plot in Figure IV.10 shows the emergency dike at phase 3 and two snapshots. After 15 minutes layer 4 is placed.

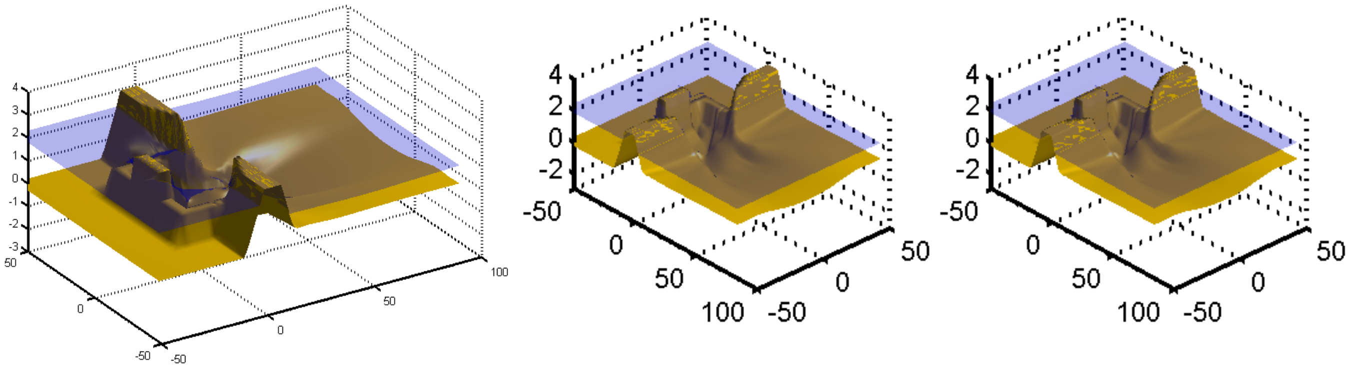


Figure IV.10: Strategy B, Phase 3, emergency dike and snapshots at t = 58 and 65

Phase 4

This phase is comparable to the shape of phase 2, however, the emergency dike emerges completely above the water level and a closure is made. In Figure IV.11 the completed emergency dike is visible. The snapshots are exactly the same since no more water is flowing through the breach.

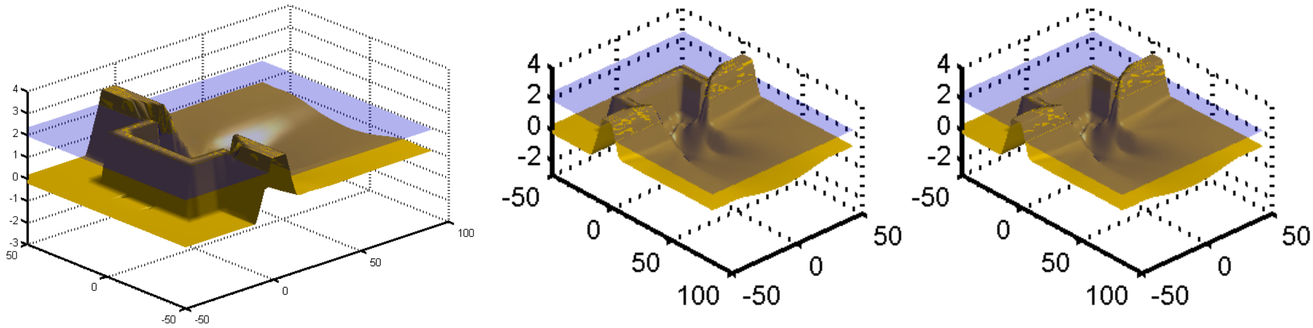


Figure IV.11: Strategy B, Phase 4 emergency dike and snapshots at t = 73 and 80 minutes

V. Static stability calculations

V.1 Shearing

For shearing the safety factor is calculated by:

$$FS = \frac{T}{F_H} \qquad [-] \qquad \text{(V-1)}$$

Where, T is the friction force calculated by:

$$T = f \cdot G \qquad [\text{kN}] \qquad \text{(V-2)}$$

G is the resulting vertical force and f is the shear coefficient.

F_H is the horizontal hydrostatic force of the water and F_v is the vertical hydrostatic force of the water. W is the weight of a Big Bag.

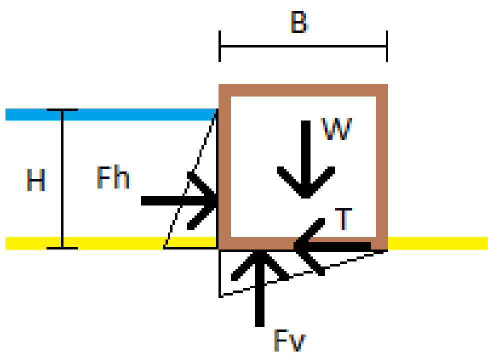


Figure V.1: Forces on the emergency dike

Below the shear stability is calculated per running meter.

Horizontal water force with two meters of water is:

$$F_H = \frac{1}{2} \cdot \rho \cdot g \cdot H^2 = \frac{1}{2} \cdot 1000 \cdot 9.81 \cdot 2^2 = 20\ kN \qquad \text{(V-3)}$$

The width of one Big Bag is 1 m.

$$F_V = \frac{1}{2} \cdot \rho \cdot g \cdot B \cdot H = \frac{1}{2} \cdot 1000 \cdot 9.81 \cdot 1 \cdot 2 = 10\ kN \qquad \text{(V-4)}$$

The density of wet sand is assumed at 1800 kg/m^3, therefore the weight of two Big Bags on top of each other is:

$$W = 2 \cdot 9.81 \cdot 1800 = 35\ kN \qquad \text{(V-5)}$$

The shear coefficient for wet sand and the plastic of a Big Bag is assumed at 0.3 (Boon, 2007).

Formula V-1 becomes:

$$FS = \frac{0.3 \cdot 25}{20} = 0.38 < 1 \qquad \text{(V-6)}$$

The safety factor is smaller than 1, so the emergency dike is not safe for shearing.

To improve shear stability, there is now assumed that the emergency dike consists of two rows of two Big Bags on top of each other. The horizontal hydrostatic force stays the same, however the weight of the Big Bags and the vertical hydrostatic force increase.

$$F_H = 20\ kN \qquad \text{(V-7)}$$

$$F_V = 20\ kN \qquad \text{(V-8)}$$

$$G = 71\ kN \qquad \text{(V-9)}$$

Now, Formula V-1 becomes: $FS = \frac{0.3 \cdot 51}{20} = 0.77 < 1$

The safety factor is larger than 1, so the emergency dike is stable for shearing.

If the emergency dike consists of 3 rows of two Big Bags on top of each other:

$$F_H = 20\ kN \qquad \text{(V-10)}$$

$$F_V = 30\ kN \qquad \text{(V-11)}$$

$$G = 106\ kN \qquad \text{(V-12)}$$

Now, Formula V-1 becomes:

$$FS = \frac{0.3 \cdot 76}{20} = 1.1 > 1$$

V.2 Piping

For the piping calculation the Bligh and Lane formula are used:

Bligh:

$$L = 1{,}5 \cdot C \cdot \Delta H \qquad \text{[m]} \qquad \text{(V-13)}$$

$$C_{Bligh} = 12$$

$$L = 1{,}5 \cdot 12 \cdot 2 = 36\ m$$

The actual piping length is 3 Big Bags in a row which is 3 m. The safety coefficient is: 3/36 = 0.1.

Lane:

$$L = C \cdot \Delta H \qquad \text{[m]} \qquad \text{(V-14)}$$

$$C_{Lane} = 5$$

$$L = 5 \cdot 3 = 15\ \text{m}$$

The actual piping length according to Lane is 1/3 * 3 = 1 m. The safety coefficient is: 1/15 = 0.1.

VI. Interview Dutch Water Board

This interview is documented in Dutch: Dit document is opgesteld voor het interview met Hans Knotter van het Waterschap Rivierenland, d.d. 28 mei 2014, ten behoeve van het afstudeerwerk van Tijmen Albers, student Civiele Techniek, TU Delft. Het afstudeerwerk wordt uitgevoerd in samenwerking met Deltares en heeft als onderwerp noodmaatregelen bij dijkdoorbraken. In dit document staan vragen aan Waterschap Rivierenland over het handelen van een Waterschap voor en tijdens een dijkdoorbraak. Doel is om opgedane kennis in het afstudeerwerk toe te passen in (of aanbevelingen te doen voor) een Decision Support System aansluitend bij de huidige aanpak van Nederlandse Waterschappen.

Aanwezig: Hans Knotter (Coördinator dijkbewaking Waterschap Rivierenland), Eric Huijskes (Deltares), Tijmen Albers

Interviewer: Tijmen Albers
Geïnterviewde: Hans Knotter (Waterschap Rivierenland)

1. Wat is het huidige protocol van Waterschap Rivierenland bij een dijkdoorbraak en de fase hiervoor?

Er is op dit moment bij Waterschap Rivierenland geen protocol tijdens het moment van een dijkdoorbraak. Dit wordt door Waterschap Rivierenland echter wel wenselijk geacht. De reden dat er op dit moment nog geen protocol bestaat is omdat de kennis over bresgroei of bresgroei remmende noodmaatregelen voor specifieke dijken ontbreekt. Er zijn op dit moment geen tijd en middelen beschikbaar vanuit het Waterschap om hier onderzoek naar te doen. Als zich op dit moment een dijkdoorbraak voordoet zal er vanuit de bestaande organisatie geïmproviseerd worden.

Op de momenten voor een dijkdoorbraak lopen er dijkwachters over de dijken. Sommigen te voet en sommigen in de auto. Te voet wordt aangeraden omdat hierbij een betere inspectie gedaan kan worden. De auto kijkt vooral naar de kruin en het buitentalud en de dijkwachters te voet letten op het binnentalud, de teen en het achterland. Er wordt uitgegaan dat de dijkwachters te voet gemiddeld 2 km/u afleggen. Het te inspecteren traject bedraagt 10 km. Dit houdt in dat de dijkwachters te voet 5 uur doen over hun ronde. Op bepaalde ‘hotspots’ wordt specifiek gecontroleerd met een afvinklijst per patrouille vak.

a. Wat is de organisatie achter het protocol en wie heeft de verantwoordelijkheid?

Er is geen protocol dus ook geen speciale organisatie tijdens een dijkdoorbraak. Er zal in een dergelijke situatie verder gewerkt worden met de organisatie die staat zoals tijdens een gewoon schadebeeld.

b. Wanneer wordt het protocol in werking gesteld?

Er is geen protocol met hoe te handelen tijdens een dijkdoorbraak. De handelingen volgen aansluitend aan de bestaande calamiteitenorganisatie.

c. Hoeveel tijd kost het doorlopen ervan?

Er is geen protocol met hoe te handelen tijdens een dijkdoorbraak. Het inspecteren van het dijkvak kost maximaal 5 uur. De invultijd en behandeltijd van de formulieren verschilt, waarbij door middel van een relatief simpel puntensysteem prioriteit wordt gegeven aan de meest kritieke schade.

d. Is het personeel getraind en in hoeverre kunnen ze direct actie ondernemen?

Er worden oefeningen georganiseerd en er kan verondersteld worden dat de dijkwachters (enige) kennis hebben. Schademeldingen zullen altijd via de dijkpost gaan. Dit is het geval om de prioriteit te bepalen en om de schademelding te kunnen controleren. De dijkwachters worden niet geacht zelf direct actie te ondernemen.

e. Hoe waarschijnlijk is het dat de schade aan de dijk niet wordt vastgesteld voordat er een bres ontstaat?

Dat verschilt per schadebeeld. Een minder goed zichtbaar faalmechanisme zoals piping zal eerder niet gedetecteerd worden dan bijvoorbeeld schade aan de kruin. Uit het onderzoek van de TU Delft (Lendering, et al., 2014), volgt dat de kans op een fout per taak voor een weinig getrainde dijkwacht ~1/10 - 1/20 is. Voor piping (onafhankelijke waarneming) is dit voor een afstand van 10 km ~1/2 – 3/4.

Om het vaststellen van schade te bevorderen is het beter om dijkwachters te voet in te zetten in plaats van met een auto. Op deze manier is er een grotere kans op waarnemen van het schadebeeld.

f. Is er materiaal en materieel beschikbaar en is er samenwerking met een aannemer?

Elke dijkpost van Waterschap Rivierenland heeft met 3-5 aannemers een waakvlamcontract. De aannemers zijn een mix van plaatselijke kleinere aannemers en grotere aannemers. De aannemers zorgen voor het materiaal en materieel. De contracten zijn gebaseerd op preventieve noodmaatregelen. Dit materiaal en materieel zullen niet voldoende zijn voor het dichten of remmen van een bres.

g. Wordt een aannemer van te voren ingelicht over een hoogwater en hoeveel tijd kost het hem om zijn materieel en materiaal te mobiliseren?

De aannemers worden van te voren gewaarschuwd bij een dreigend hoogwater. Het kost een aannemer 12 uur om zijn materiaal en materieel te mobiliseren. Daarna is de tijd afhankelijk per faalmechanisme en wat voor materiaal of materieel daarvoor nodig is.

h. Wordt er vooruit gedacht vanuit een preventieve maatregel naar een curatieve?

Er wordt op dit moment vanuit gegaan dat een preventieve maatregel afdoende is, daarom wordt er niet vooruit gedacht naar een curatieve maatregel. Dit kan wenselijk zijn, afhankelijk van de maatregel en moet niet ten koste gaan van het uitvoeren van de preventieve maatregel. Er zal dus los van het uitvoeren van de preventieve maatregel iemand bezig moeten zijn met een eventuele curatieve maatregel.

2. Voor zo ver u dat kunt inschatten, is het protocol voor alle Waterschappen hetzelfde, zo nee, wat zijn de verschillen en waarom zijn die er?

Protocollen voor een curatieve maatregel zijn voor zover bekend nergens aanwezig. De protocollen voor preventieve maatregelen verschillen per waterschap. Een generieke basis voor beide protocollen wordt gezien als een grote vooruitgang. Een generieke basis met specificaties van type water en type dijk zou voor structuur zorgen. Op deze manier wordt voorkomen dat er 'zomaar wat gedaan' wordt en dat het gedane werk gecontroleerd kan worden.

3. Ziet u verbeterpunten in het protocol van Waterschap Rivierenland?

Ook voor Waterschap Rivierenland geldt dat er meer moet worden gestandaardiseerd en opgenomen in protocollen. Voor bijvoorbeeld het opkisten van wellen zijn werkinstructies. Dit moet echter uitgebreid worden naar draaiboeken voor het handelen tijdens het complete hoogwater. Het verbeterpunt hierin is dat werk gecontroleerd kan worden.

4. Wat ziet u als grootste knelpunt bij deze procedure?

De losse organisaties worden gezien als de grootste belemmering. De samenwerking tussen waterschappen moet geïntensiveerd worden. Op deze manier kan er professioneler gehandeld worden en wordt voorkomen dat er 'maar wat gedaan wordt'.

5. Heeft u zelf ervaring met noodmaatregelen en wat zijn uw bevindingen?

Ervaring met het opkisten van wellen tijdens hoogwater en ervaring met het hoogwater van 1993 en 1995. Tijdens deze situaties is gebleken dat het oefenen en het bestaan van werkinstructies erg belangrijk zijn. Zonder werkinstructies blijft men zich afvragen of er juist gehandeld wordt of wie er verantwoordelijk is. Het geeft houvast en zekerheid voor de mensen op de dijken. Daarnaast is er sprake van paniek en stress tijdens een hoogwater. Echter wordt er door de mensen van het waterschap wel met veel enthousiasme gewerkt.

6. Hoe kijkt u aan tegen Decision Support Systems bij dijkdoorbraken?

Positief, indien ze snel uitgevoerd kunnen worden. Het DSS zal in ieder geval gegevens over de grond, het type dijk en het type water moeten bevatten. Brestypologie formulieren met daarin het type bres en de fase waaruit de verwachtte groei afgeleid kan worden kunnen een toevoeging zijn. Op deze manier kunnen na een eerste schade eventueel al adviezen uitgebracht worden.

7. Waar zou de grootste kracht van een Decision Support System liggen?

De kracht ligt in de tijdwinst die geboekt kan worden. Het materiaal en materieel dat eerder ter plaatse kan zijn kan een grote kracht zijn.

8. Waar kan de meeste tijd op gewonnen worden?

De koppeling tussen preventieve en curatieve maatregelen en het vooruit denken vanuit de preventieve maatregel.