

M.Sc. Thesis

Vulnerability of Structural Transitions in Flood Defences

Erosion of Grass Covers due to Wave Overtopping

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Preface

This Master of Science thesis is the final part of the Master Hydraulic Engineering at the faculty of Civil Engineering and Geosciences of the Delft University of Technology. It contains a research developed in cooperation with the institute for applied research, Deltares. The thesis focusses on the vulnerability of structural transitions in flood defences, with respect to the erosion of grass covers due to wave overtopping.

I would like to thank the members of my graduation committee Prof. dr. ir. S.N. Jonkman, Dr. ir. J. Dijkstra, ir. A. van Hoven, ir. H.J. Verheij and ing. M.Z. Voorendt for their guidance and constructive criticisms. These were essential elements during the entire process of this thesis. Also, I want to express my gratitude to Deltares for sharing their knowledge and facilities.

Last but not least, I would like to thank my fellow graduates for the pleasant conversations and discussions and my colleagues for their feedback and support.

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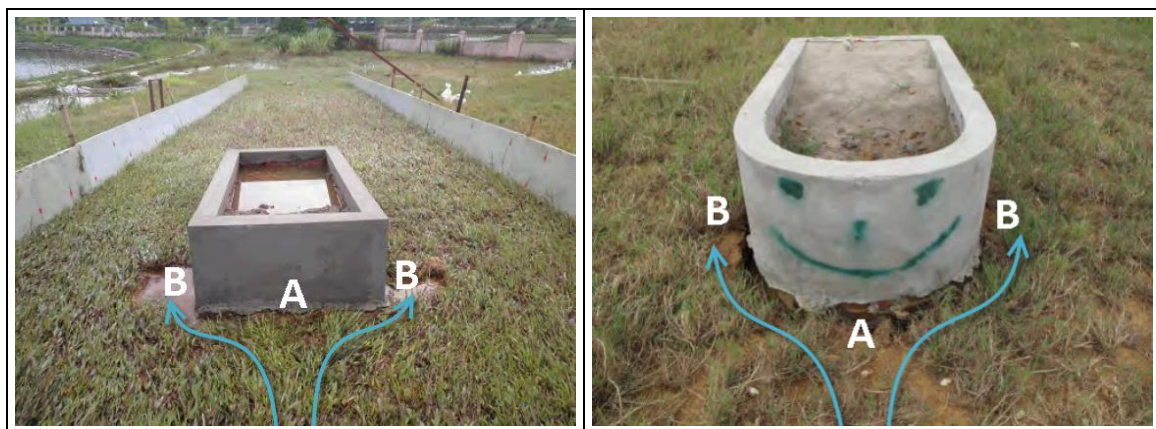
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Abstract

Hardly any flood defence system is a continuous dike system, since it is often interrupted by non-water retaining structures, as for example stairs, buildings, roads, fences and trees. Also larger hydraulic structures as floodgates and sluices are sometimes situated in the dike alignment. The transition between these different objects and the dikes with grass covers is still an area that receives limited attention in current methods for design and safety assessment in the Netherlands and other countries. The problem at hand is that during flood disasters in New Orleans, Thailand and France, these transitions appeared to be vulnerable spots in the flood defence system. The erosion process of the grass cover developed fast around objects or structures which were placed on the inner slope and suffered from wave overtopping.

The objective of this thesis is to gain insight into the vulnerability of transitions between dikes with grass covers and (non-)water retaining objects during overtopping of flood defences. This is achieved by testing the applicability of current erosion models for the structural transition zone. Some models are expanded with an extra load and/or strength parameter to describe the effects at the transition.

First of all, the problem is addressed by defining the exact location where the initiation of erosion takes place, which is shown in the figure below for a generalized situation with a square and round object.



Generalized situation for erosion zones around objects (Le, 2012)

Based on earlier field experiments and observations after dike failure, two erosion zones are defined in the figure above:

- Zone A: This is the area in front of the object on the upstream side. Here erosion is not caused by the flow velocity in the direction parallel to the slope, because it reduces to zero. However, it is assumed the dynamic pressures exerted on the grass cover increase significantly due to the impact.
- Zone B: This is the area next to the object, just after the corner (square object). Here erosion can be caused by the flow velocity in the direction parallel to the slope, because it is present and assumed to increase due to the object.

The erosion models that are defined for the different erosion zones are based on existing models for erosion on the slope. These models are separated in a load and strength side, which are paraphrased or extended to fit the purpose for erosion due to the presence of an object. In zone A is chosen for two models that use the pressure load of the wave as input, because the velocity is (nearly) zero at this point. The difference between both models is most importantly the crack between soil and structure that is required in one of the models. In zone B is chosen for a flow erosion model, because the velocity is assumed to be an important mechanism to initiate erosion. The models are expanded with a load and strength parameter to define the influence of an object on the inner dike slope. These parameters are determined by numerical modelling and field experiments.

Numerical analyses are performed on the load side of the erosion process by the program ComFLOW. This model should determine the pressures and velocities that occur around an object. This is validated with a field experiment that used a similar set-up. It appears the model calculates the velocities on the slope accurately, but due to the lack of friction in the model this velocity accelerates too much on the transition between the slope and the horizontal part. This higher velocity also causes differences in the impact pressure that is found in front of the object. However, it does show that the velocity next to the object has no peak acceleration, but even slightly decreases along the slope next to the object. Therefore the earlier assumption that a velocity difference influences the load on the grass cover next to an object seems not plausible.

For the field experiment with the overtopping simulator, two pressure gauges were installed in erosion zone A of the object and on the slope. These showed with respect to measurements on the slope, besides a difference in the hydrostatic pressures, a large difference in the dynamic pressures. These differences are compared for several wave volumes, which led to a definition of the load parameter in the pressure load model. Conclusions with respect to the actual erosion around the object could not be drawn, because due to gaps in the soil in front of the object; this area was filled with concrete.

Since the earlier performed field experiment with respect to the grass tensile strength did not give values that correspond with the theoretical values, a second experiment on the strength side of the erosion process was performed. This time the displacement of the grass cover is used as a starting point of the experiment. The cover layer was loaded until a certain displacement for several times and the subsequent force was measured. It appeared the effect of fatigue was present in the cover layer as assumed, since the force needed to reach a certain displacement decreased after each test. In other words, after one loading cycle some (weak) roots break, but quite some others only get partly pulled out or relocate. This way the force can be redistributed to other roots so the soil keeps some strength. Nevertheless, the pulse load that should correspond with overtopping waves still shows values larger than the theoretical values. Also the influence of the sides of a square grass turf on the total tensile strength is investigated. This leads to an assumption for the strength parameter that can be used for the influence of an object, because a grass turf next to a structure has one side less that contributes to the strength. The influence of one side appears to be slightly less than theoretically would be assumed.

The numerical modelling and field experiments led to erosion models for zone A and B, including the load and strength parameters for the influence of a structure. As a first validation of these

models, the field experiment at the Vechtdijk is used. This experiment consisted of a slope with a tree close to the toe. For the erosion in zone A, it appeared the cumulative load model predicted the time needed to reach the different erosion states quite well. However, the grass quality was assumed as average, which was not investigated during the experiment. The impact load model also gave a reasonable, but very indicative, prediction.

For further investigations it is recommended to look into the exact location of the transition between the defined erosion zones A and B. This should be performed to say something about the relation between the zones with respect to erosion in zone A possibly leading to erosion in zone B. Furthermore the field experiments with both the overtopping simulator and the turf-tensile apparatus could be continued. Now the load on the grass cover for several wave volumes by dynamic pressures is determined, the grass could be loaded multiple times by the tensile apparatus with these forces. This would give insight in how long a grass cover will be able to withstand several wave volumes. For the overtopping test, measurements could be performed in zone B and the effects of an object on the slope could be determined. Also the influence of objects with different shapes could be investigated and off course the test could be performed without the concrete replacement. However, it has to be stated that all these investigations could possibly also be performed by ComFLOW. Before this is possible, the model needs some improvements. Most importantly, it lacks roughness of the model boundaries (bottom) and objects to calculate the velocities of the overtopping wave correct. Finally, the defined model for zone A is based on the dynamic pressures of the overtopping waves. These values are not easy to measure, but it is the easiest way to describe the model. If the relation between the dynamic pressure and the impact velocity of the wave could be derived; the erosion in front of the object could also be described by the easier applicable impact flow velocity. Another option is to determine the relation between the wave volume and the dynamic pressure to make the input of the model more convenient.

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1 Introduction

All over the world, dikes prevent deltaic regions and river basins from flooding. These dikes do not only prevent the regions against high water levels, but also against incoming waves. They hit the outer slope, run up and might even overtop the dike. The dike must be protected against the forces these waves exert on the cover layer. Fortunately, the effects of these wave forces are understood better each day. Therefore, it is easier to deal with the forces and its effect on the failure probability of a constructed fragment of a flood defence. The problem is that hardly any flood defence is a continual structure. It is often interrupted by non-water retaining structures, as for example stairs, buildings, roads and trees. Also larger hydraulic structures as floodgates and sluices are sometimes situated in the dike alignment. The transition between these different objects and the dike is still an area that receives limited attention in current methods for design and safety assessment in the Netherlands and other countries.

During recent flood events in New Orleans, France and Thailand, quite some dikes failed at the transition between hydraulic structures and earthen dikes. For example a storm surge barrier in New Orleans prevented the water from entering, but the attached dike was completely eroded. More examples and details on these events can be found in appendix A. These events clearly show that a transition is a proven weak spot and some design guidance seems important to guarantee the reliability of these connections.

In the Netherlands this topic is investigated within the ComCoast and SBW programs (this means in Dutch 'Sterkte en Belastingen Waterkeren'). Both research programs investigate the effects of overtopping, but they have a different focus. ComCoast looked into strengthening of the dike to increase the overtopping resistance. Where later and on-going, SBW focusses on investigating the failure mechanism due to overtopping on an inner grass slope. This should lead to practical assessment and design formulae for erosion and deep or superficial shearing. In other words an update of the current Dutch guideline VTV (2006), which stands for (in Dutch) 'Voorschrift Toetsen op Veiligheid'.

1.1 Problem definition

Many (non-)water retaining objects are part of flood defence systems. The transitions between dikes with grass covers and these objects appear to be weak spots in the flood defence systems. The word 'transition' is often used in combination with flood defences, however many times it refers to different aspects. The most common ones involve the following transitions:

- The transition between one type of revetment and another. This refers to for example the transition between block mats and rip-rap that are placed on the outer slope.
- The transition between the slope of a dike and the horizontal area. This can also be referred to as the geometrical transition at for example the toe or crest of the dike.
- The transition between a hydraulic structure, as for example a sluice or a storm surge barrier, and the adjacent dike with grass cover.
- The transition between non-water retaining objects and the adjacent dike with grass cover. These objects can be divided into the following:

- Trees
- Cables and pipes
- Buildings
- Other structures (stairs, roads, fences, etc.)

The main focus of this thesis is on the third and fourth type of transition. These two could also be called structural transitions, because they focus on the connection between (hard) structures and dikes with grass covers. Within the fourth transition type, also cables and pipes are included. These are not further elaborated in this thesis, because they are mostly placed within the dike body.

The reason for this focus is the fact that recent flood events in New Orleans, France and Thailand (see appendix A) showed how vulnerable these kind of transitions actually are and how little is known about the processes that occur. It appears erosion processes develop fast around objects or structures which are placed on the inner slope of a dike and are subject to overtopping.

There is still little research done on this topic, thus the effect these objects have on erosion cannot be fully quantified yet. Several models are discussed in this thesis, which show possibilities to model the problem at hand by applying a lower strength term and a higher load term for the transition zone. Nevertheless, other modelling approaches are also possible, for instance impact models. For now it remains a weak spot in the design and safety assessment of flood defence systems and some design guidance seems important to guarantee the safety of these connections. Therefore, it is important to gain more insight into the mechanisms that occur at such transitions; referring mainly to the increased load due to flow concentration and turbulence.

1.2 Objective

The objective of this thesis is to gain insight into the vulnerability of transitions between dikes with grass covers and (non-)water retaining objects during overtopping of flood defences. This will be achieved by testing the applicability of current erosion models for the transition zone; with or without the addition of an extra load and/or strength parameter.

This will be achieved by answering the following questions:

- What are the current theories and models for grass cover erosion during overtopping and what is known from practice about erosion due to structural transitions?
- Which grass erosion models are applicable for the transition with (non-)water retaining objects; with or without the addition of an extra load and/or strength parameter?
- How could the load and strength parameters be determined from numerical modelling and field experiments?
- How should this research be continued to develop a generally applicable guideline to ensure the safety of transitions between grass covers and structures?

1.3 Structure of the report

Chapter 2 describes the literature that is found on the problem described in this chapter. It includes the load on a dike, strength of a dike and the defined erosion models for grass covers on dikes. Next, chapter 3 elaborates on the models that can be used to describe the erosion phenomena in the vicinity of a structure. Different parameters, which are used in these models to make them viable for erosion close to structures, are investigated in chapter 4. This includes a numerical model research and field experiments with respect to overtopping and grass tensile strength. The results of the numerical model and the field experiments are analysed and combined with the defined models in chapter 5. Also the resulting models are evaluated with respect to earlier field experiments. Finally, chapter 6 and chapter 7 describe respectively the conclusions of this thesis and the recommendations for further research.

2 Literature research

This chapter elaborates on the current knowledge of the safety assessment of flood defences and the subsequent knowledge on the erosion of grass covers. As stated in the introduction, it would be very useful to have some design guidance in case of structural transitions. Therefore insight has to be gained in the mechanisms that occur at such transitions. First the main probabilistic design methods in the Netherlands are described shortly in paragraph 2.1. Then the load and strength related to the inner slope of a dike are summarized in paragraph 2.2 and 2.3. Finally the currently developed and possibly applicable erosion models for grass covers are discussed in paragraph 2.4.

2.1 (semi-)Probabilistic design of flood defences

In this paragraph the safety assessment, which is nowadays used in the Netherlands, is briefly discussed. This is done because the loading part of the dike, described in paragraph 2.2, should be related to the resistance part, described in paragraph 2.3. The design assessment is described by the failure mechanisms in paragraph 2.1.1, the safety assessment in paragraph 2.1.2 and the current design methods on transitions in paragraph 2.1.3.

2.1.1 Failure mechanisms

In general the most relevant failure mechanisms of a dike can be summarized in twelve possibilities. These are shown in Figure 2-1. The goal of describing all the possible failure mechanisms is to make the behaviour of the structure accessible for analysis. The distinction between some of the failure modes is not always that obvious, but it is expected that all the ways of failure are covered. The failure mechanism, which is analysed in this report, is a consequence of wave overtopping. The reason behind the loss of stability due to overtopping should be analysed to understand the failure of a cover layer.

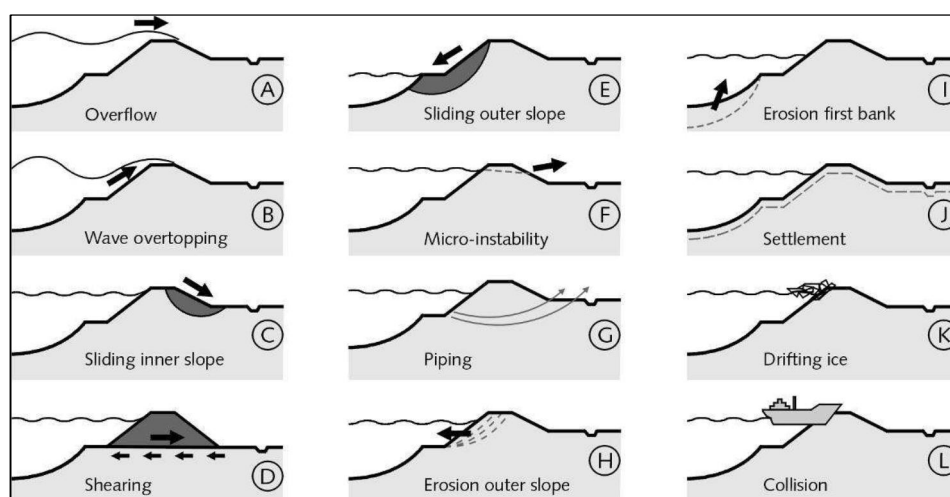


Figure 2-1: Different failure modes of a dike structure (Weijers et al., 2009)

These failure mechanisms usually occur separately from each other. This is also shown in a flow chart, with on top the overall failure or collapse of a flood defence structure, see Figure 2-2. This flow chart includes the failure of a hydraulic structure, which is left out in the dike failures.

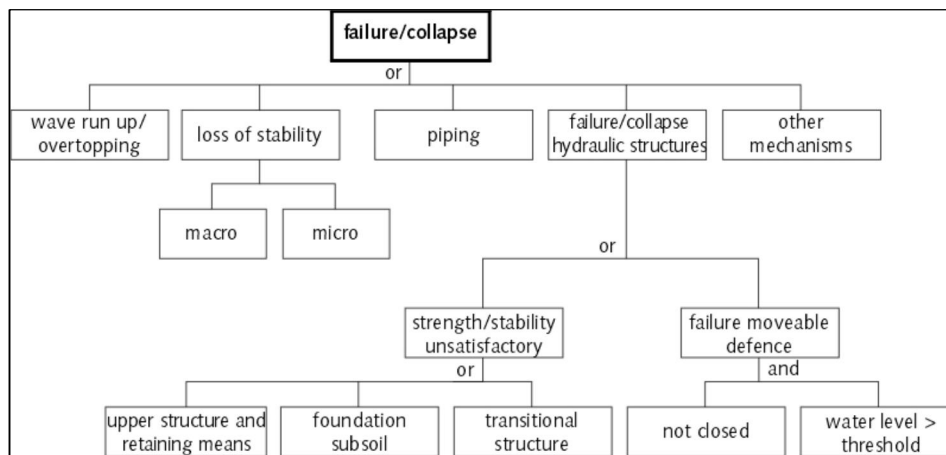


Figure 2-2: Fault tree for dike failure (Weijers et al., 2009)

By assigning a probability of failure to all the components of the flow chart, an overall probability of failure can be found for a dike structure. This information can be used in the safety assessment of a dike which is discussed in paragraph 2.1.2.

2.1.2 Safety

The original idea of safety of a dike was only based on the highest recorded water level. The crest was constructed on this level plus a little extra. In the thirties of the last century they changed this to a more mathematical approach, which includes the risk of a flood defence. This technique was then combined with an economical analysis of the investment cost, so it was possible to determine the economical optimal safety level.

The risk is based on the probability of flooding and the economical consequences. The probability of flooding is determined from the probability of exceedance of the water level and the probability this will cause a breach. These values are based on an analysis of the loads on flood defences and the strength (resistance) of those flood defences, separated in different failure modes, as is described in paragraph 2.1.1. Their combination yields the probability of flooding. This strength and load relationship can be visualized in a simple formula:

$$Z = \text{Strength} - \text{Load}$$

Where $Z < 0$ means a failure of the dike function.

In case of erosion at transitions, the load mostly originates from the water overtopping the structure and the strength originates from the erosion resistance of the grass cover.

2.1.3 Existing design methods for structural transitions

The design methods used in the Netherlands for structural transitions are the 'Voorschrift Toetsen op Veiligheid' (VTV, 2006) and 'TAW Waterkerende grondconstructies' (TAW, 2001).

Both methods state that in case of a transition between a hydraulic structure and an earthen dike, an additional load at the transition should be taken into account. The exact additional load though, is not determined. It also states that the effects of piping at transition zones can be counteracted by installing seepage barriers.

On non-water retaining objects the guidelines are saying the influence of a failing object should only reduce the quality of a dike till a level it is still sufficient to fulfil its water retaining function. It is also mentioned that with an overtopping discharge of 0.1 [l/s/m], no object causes any reduction in the safety level. Every higher discharge value should be counteracted by raising the crest level or reinforcing the soil around the structure.

The international manual EurOtop (2007) describes wave overtopping discharges at multiple coastal structures, including the distribution of overtopping wave volumes. The overtopping discharge is simply the total volume of overtopped water (per unit length) in a certain duration. There will be a number of overtopping waves that produce a distribution of overtopping wave volumes. The distribution is characterized by many small overtopping waves and a few much larger ones.

Another, not yet published, international manual is the International Levee Handbook (ILH, 2013). This is written to make a single, comprehensive handbook collating international practices and knowledge. This handbook has the goal to help to spread consistent standards and approaches widely. On the subject of transition zones, the handbook points it out as a weak spot and suggests additional armouring of the area. It states guidance is lacking and most likely either physical modelling or sophisticated numerical simulations will be required to establish flow velocities due to surge overtopping in the vicinity of dike/floodwall transitions.

2.2 Loads on inner dike slopes

Pressure fluctuations due to a water flow are the main load on the inner slope of a dike. This water load is generated by incoming waves that run-up the outer dike slope, see paragraph 2.2.1. If these waves carry enough energy they will overtop the crest, see paragraph 2.2.2. The flow velocity and depth that reaches the inner slope due to this overtopping wave is discussed in paragraph 2.2.3. Finally the occurring turbulence and subsequent pressure gradients are defined in paragraph 2.2.4.

2.2.1 Wave run-up

If the wave run-up is higher than the crest freeboard a wave will overtop. It is important to know something about the flow depth, velocity and discharge, because this determines the loads on the crest and inner slope. Therefore some insight into the amount of run-up is also trivial. The wave run-up depends on the breaker parameter (surf similarity parameter), which is a non-dimensional value defined by the steepness of the outer slope divided by the root mean square of the wave steepness. The value of the breaker parameter gives information on how a wave will break on the outer slope. Waves with the lowest values, $\xi_0 < 0.5$, are usually found along flat beaches and are called spilling waves. The plunging waves can usually be found along sea dikes and correspond to higher values of the breaker parameter, $0.5 < \xi_0 < 3.3$. The collapsing and

surging waves correspond to the highest values of the parameter and almost have half of there wave energy reflected back into the sea, $\xi_0 > 5$ (Battjes, 2001). The breaker parameter value can be found using Equation 2-1 (TAW, 2002).

$$\xi_0 = \frac{\tan \alpha}{\sqrt{s_{m-1.0}}} \quad 2-1$$

With:

$$s_{m-1.0} = \frac{H_{m0}}{L_0} \quad \text{and} \quad L_0 = \frac{g \cdot T_{m-1.0}^2}{2 \cdot \pi}$$

ξ_0	= breaker or surf parameter	[-]
α	= angle of the outer slope	[-]
$s_{m-1.0}$	= wave steepness	[-]
H_{m0}	= spectral wave height	[m]
L_0	= deep water wave length	[m]
g	= acceleration due to gravity	[m/s ²]
$T_{m-1.0}$	= spectral wave period	[s]

Another important factor of the run-up is the surface type of the flood defence. A rough slope like rock will cause less run-up than a smooth slope like grass or asphalt. Also a berm in the defence structure will reduce the run-up. Just like the incoming angel of the waves has an effect on the run-up. The wave run-up level is the distance measured vertically from the still water level. The level that is exceeded by 2% of the incoming waves is given by $z_{2\%}$. The formula of the wave run-up is given by Equation 2-2 (TAW, 2002).

$$\begin{aligned} \frac{z_{2\%}}{H_{m0}} &= 1.65 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_0 \quad (\xi_0 \leq 1.75) \\ \frac{z_{2\%}}{H_{m0}} &= \gamma_f \cdot \gamma_\beta \cdot \left(4.0 - \frac{1.5}{\sqrt{\xi_0}} \right) \quad (\xi_0 > 1.75) \end{aligned} \quad 2-2$$

Where:

$z_{2\%}$	= run-up level exceeded by 2% of the incoming waves	[m]
γ_b	= reduction factor for a berm	[-]
γ_f	= reduction factor for the roughness on the outer slope	[-]
γ_β	= reduction factor for angled wave attack	[-]

2.2.2 Overtopping

The discharge that overtops a flood defence structure and thus reaches the inner slope can be calculated by the TAW guidelines using Equation 2-3 (TAW, 2002). Nevertheless, these guidelines are based on a constant discharge. The effect of singular waves will probably be different. Especially in case of erosion on the inner slope the difference in for example water velocity and

turbulence between constant and varying loads might be essential. During a storm, different volumes of water will overtop a dike; this can be described by a Weibull distribution as shown in Equation 2-4 (Van der Meer et al., 2006).

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan \alpha}} \cdot \gamma_\beta \cdot \xi_0 \cdot \exp \left(-4.3 \cdot \frac{R_c}{H_{m0}} \cdot \frac{1}{\xi_0 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v} \right) \quad 2-3$$

With a maximum of:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp \left(-2.6 \cdot \frac{R_c}{H_{m0}} \cdot \frac{1}{\gamma_f \cdot \gamma_\beta} \right)$$

Where:

- q = mean wave overtopping discharge [m³/s/m]
 R_c = crest freeboard relative to SWL [m]
 γ_v = influence factor for a vertical wall on slope [-]

$$P_v = P(V \geq V) = \exp \left(- \left(\frac{V}{a} \right)^{0.75} \right) \quad 2-4$$

With:

$$a = \frac{0.84 \cdot T_m \cdot q}{P_{ov}}$$

Where:

- P_v = probability of an overtopping volume [-]
 V = volume of an overtopping event [m³/m]
 P_{ov} = probability of an overtopping event [-]
 T_m = mean wave period [s]

These formulae are often used for simulation of overtopping volumes.

2.2.3 Maximum flow velocity and depth

In the previous paragraph, the distribution and volume of overtopping waves is determined. Next the progression of the wave over the crest and inner slope should be known. Schüttrumpf et al. (2003) wrote a paper in which they combined their theories. They found a general formula to express the water height and water velocity. For the location at the transition between the outer slope and the crest, they give Equation 2-5 (Schüttrumpf et al., 2003). At the transition between the crest and the inner slope, they give Equation 2-6 (Schüttrumpf et al., 2003).

$$\frac{h_{2\%}(x_c=0)}{H_s} = c'_{h,2\%} \cdot \left(\frac{z_{2\%} - R_c}{\gamma_f \cdot H_s} \right)$$

$$\frac{u_{2\%}(x_c=0)}{\sqrt{g \cdot H_s}} = c'_{u,2\%} \cdot \left(\frac{z_{2\%} - R_c}{\gamma_f \cdot H_s} \right)^{0.5}$$
2-5

Where:

$h_{2\%}$ = maximum flow depth exceeded by 2% of the incoming waves [m]
 $u_{2\%}$ = maximum flow velocity exceeded by 2% of the incoming waves [m/s]
 x_c = x-coordinate on the crest [m]

$$\frac{h_{2\%}(x_c)}{h_{2\%}(x_c=0)} = \exp \left(-c''_{h,2\%} \cdot \frac{x_c}{B_c} \right)$$

$$\frac{u_{2\%}(x_c)}{u_{2\%}(x_c=0)} = \exp \left(-c''_{u,2\%} \cdot \frac{x_c \cdot f}{h_{2\%}(x_c)} \right)$$
2-6

Where:

B_c = width of the crest [m]
 f = friction coefficient (grass = 0.02) [-]

The coefficients $c'_{h,2\%}$, $c'_{u,2\%}$, $c''_{h,2\%}$ and $c''_{u,2\%}$ are determined in two separate physical model tests. The values are listed in Table 2-1.

Table 2-1: Coefficient values for flow depth and flow velocity

	Schüttrumpf, 2001b	Van Gent, 2002
$c'_{h,2\%}$	0.33	0.15
$c'_{u,2\%}$	1.37	1.30
$c''_{h,2\%}$	0.89	0.40
$c''_{u,2\%}$	0.50	0.50

According to Schüttrumpf and Van Gent the large difference between some of their results can be explained by the use of different models and programs. Bosman (2007) looked at this difference and found some new equations that should cover the discrepancy. The tests of Bosman (2007) are partly repeated and expanded by the Flowdike project. After their tests, they elaborated new formulae that are described by Equations 2-7 (Van der Meer et al., 2010).

$$h_{2\%}(x_c=0) = 0.20 \cdot (R_{u2\%} - R_c)$$

$$h_{2\%}(x_c) = 0.13 \cdot (R_{u2\%} - R_c)$$

$$u_{2\%}(x_c=0) = 0.35 \cdot \cot \alpha \cdot (g \cdot (R_{u2\%} - R_c))^{0.5}$$

$$\frac{u_{2\%}(x_c)}{u_{2\%}(x_c=0)} = \exp \left(-1.4 \cdot \frac{x_c}{L_{m-1.0}} \right)$$
2-7

Where:

$R_{u2\%}$	= run-up level exceeded by 2% of the incoming waves	[m]
α	= angle of the outer slope	[rad]
$L_{m-1.0}$	= spectral wave length	[m]

These formulae appear not to be consistent and are also not entirely correct. For an increasing overtopping discharge, but a constant overtopping volume; the flow depth, flow velocity and flow time increase. This is not possible, because the overtopping volume is constant. Therefore these formulae are not perfect, but do give the results that are closest to reality.

On the inner dike slope Schüttrumpf et al. (2003) preferred to use a one-dimensional shallow water equation for the maximum velocity, this is shown in Equation 2-8.

$$u_{is}(s) = \frac{b_1}{b_2} + \mu \cdot \exp(-3 \cdot b_1 \cdot b_2^2 \cdot s) \quad 2-8$$

With:

$$b_1 = \sqrt[3]{g \cdot \sin \beta} \quad b_2 = \sqrt[3]{\frac{0.5 \cdot f_L}{h_0 \cdot u_0}} \quad \mu = u_0 - \frac{b_1}{b_2}$$

Where:

$u_{is}(s)$	= velocity at the inner slope at s [m] from the inner crest line	[m/s]
s	= distance from the inner crest line	[m]
β	= inner slope angle	[°]
f_L	= influence factor due to inner slope friction	[-]
h_0	= flow depth at the inner crest line	[m]
u_0	= flow velocity at the inner crest line	[m]

This equation has a limit for the distance going to infinity, this is shown in Equation 2-9.

$$\lim_{s \rightarrow \infty} u_{is}(s) = \sqrt[3]{\frac{2 \cdot g \cdot h_0 \cdot u_0 \cdot \sin \beta}{f_L}} \quad 2-9$$

By using the continuity equation the maximum flow depth can also be determined, which is shown in Equation 2-10.

$$h_{is}(s) = \frac{h_0 \cdot u_0}{u_{is}(s)} \quad 2-10$$

Where:

$h_{is}(s)$	= flow depth at the inner slope at s [m] from the inner crest line	[m]
-------------	--	-----

2.2.4 Turbulence

The initiation of motion of a sand particle and small clayey aggregates requires relatively small forces. Whereas grass covers require a significant larger force to break up these aggregates within the ground. In both cases, the resulting force is a combination of flow velocity and turbulence. The depth-averaged relative turbulence intensity is defined in Equation 2-11 and some significant values for the turbulence intensity during several wave overtopping volumes are shown in Table 2-2.

$$r_0 = \frac{\sqrt{k_0}}{U_0} \quad 2-11$$

With:

$$k_0 = \frac{1}{h} \cdot \int_0^h \frac{1}{2} \cdot (u_{RMS'}^2 + v_{RMS'}^2 + w_{RMS'}^2) dz'$$

Where:

U_0	= depth-averaged flow velocity	[m/s]
$u_{RMS'}^2$	= standard deviation of the fluctuating flow velocities	[m/s]
$v_{RMS'}^2$	= standard deviation of the fluctuating flow velocities	[m/s]
$w_{RMS'}^2$	= standard deviation of the fluctuating flow velocities	[m/s]
z	= distance in the vertical direction	[m]

Table 2-2: Characteristics of the turbulence intensity (Van der Meer, 2007)

Wave overtopping volume [l/m]	Turbulence intensity [-]
700	0.15 to 0.19
1000	0.16 to 0.20
1500	0.18 to 0.21
2500	0.20 to 0.23
3500	0.21 to 0.25

However, the pressure fluctuation which is actually causing the onset of dislocation is investigated by Emmerling (1973). He found an equation for p_m , which represents the maximum lowering of the local pressure caused by eddies. After substituting the Chézy equation into Equation 2-11, it results in Equation 2-12.

$$p_m = 12.6 \cdot \rho \cdot (r_0 \cdot U_0)^2 \quad 2-12$$

Turbulence measurements on dike slopes showed that the maximum pressure fluctuation appears at the maximum flow depth location. Also the effects of air concentration should be included in the formula, because these fluctuate with different wave volumes.

2.3 Strength of dike covers

The strength discussed in this research focusses on dikes with grass cover on top of a clay layer. Strength of clay is first discussed in paragraph 2.3.1, followed by the grass cover in paragraph 2.3.2. This includes the grass or root strength, turf-element model and critical velocities.

2.3.1 Clay layer

Many dikes over the world make use of clay, either as a top layer or as a core material. Nowadays in the Netherlands the application of clay is mostly restricted to a top layer, because of the low availability of the material. The material is known for its good erosion resistance and low permeability. This erosion resistance originates from the cohesive behaviour of clay. Cohesion is a result of the binding forces between the very fine soil particles; these forces are large in relation to the weight of particles. Transport of water through a clay layer is extremely slow, because of the very fine pores in clay.

Another aspect of clay is the presence of a soil structure. For instance root penetration from vegetation leads to this soil structure formation. In this case it is not a bad effect, because the roots also have a positive effect on keeping the soil together. Other sources, like biological activity or climate differences mostly have negative effects on the stability of clay.

The largest influences on restriction of the development of soil structure are the way of compacting and the composition of the soil. Sand inclusions enlarge the permeability and supply of oxygen and water in the clay that increases the development of the soil structure. The coherence of the clay mainly determines the strength of the clay. If the soil structure development is less clear, the coherence and strength of the clay is higher. This can also be seen in Table 2-3, where the critical flow velocity of clay is compared to the quality.

Table 2-3: Significant values for the critical velocity for clay (Hoffmans et al., 2009)

Quality of clay (Verheij et al., 1995)	Critical velocity [m/s]
sandy	0.15
poor	0.3
structured	0.6
good	0.8
very good	1.1

2.3.2 Grass cover

Most dikes in the Netherlands use grass as a cover layer. This is because of its low price, but also because of its positive effect on the erosion resistance. A typical grass cover layer can be separated in different sections with its own characteristics, see Figure 2-3.

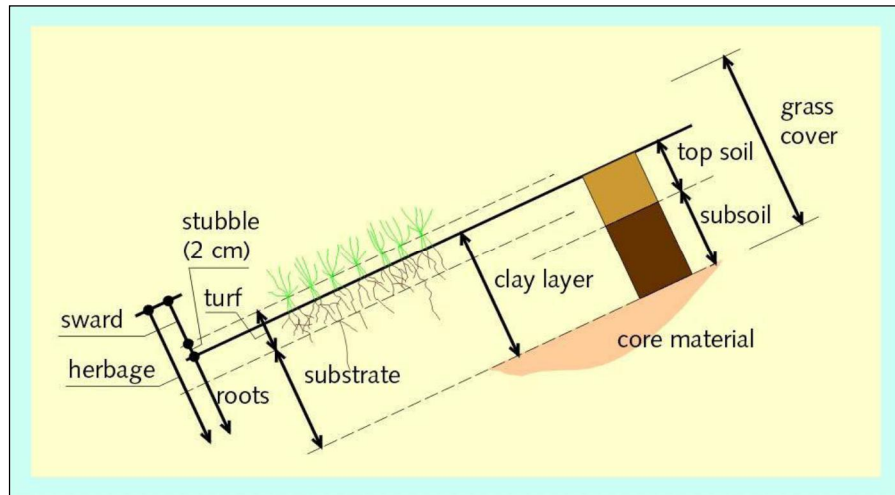


Figure 2-3: Typical grass cover layer (TAW, 1998)

The reason behind the increase of erosion resistance is that the roots of the grass keep the soil together and create a tougher clay layer. This requires clay with a high sand percentage, because the development of roots in high sand percentage clay is faster. Of course the density of the root system decreases with an increasing depth. Near the surface the strength of the grass layer dominates the root cohesion, whereas from a certain depth the cohesion of the clay dominates. The exact strength of these mechanisms is still not completely known, but a quantification on the erosion resistance is subscribed by the VTV (2006). This quantification is based on the number of roots. The measurement procedures describe counting the number of roots within a soil sample obtained with a ground drill with a diameter of 3 cm. By counting the number of roots every 2.5 cm, measured with respect to the depth direction, the root density as a function of the depth could be determined. The measured root density as function of the depth should be quantified according to Figure 2-5. This qualification of good, average, poor or very poor is used to calculate the erosion resistance of the grass mat, which is listed in Figure 2-4. The table shows respectively in columns four till seven: the minimum tensile root strength, the root cohesion, a strength parameter and the critical flow velocity.

A_r/A (mm ² /m ²)	Number of roots No./m ²	Quality grass acc. to VTV	$\sigma_{\text{roots,min}}$ (kN/m ²)	C_{roots} (kN/m ²)	C_E [m ⁻¹ s ⁻¹]	U_c [m/s]
200	15100	very poor	2.8	4.8	$0.062 \cdot 10^{-4}$	2.5
400	30150	poor	5.6	9.6	$0.033 \cdot 10^{-4}$	3.4
600	45200	averaged	8.4	14.4	$0.022 \cdot 10^{-4}$	4.2
800	60300	good	11.2	19.2	$0.016 \cdot 10^{-4}$	4.9

Figure 2-4: Indicative values for Dutch grass roots (VTV, 2006)



Figure 2-5: Grass quality as function of root density (VTV, 2006)

The initiation of motion of clay aggregates cannot be determined by the Shields equation, because stability is only based on gravity. To model the strength of a clay layer the Mohr Coulomb equation is more applicable. In this formula (c_r) represents the additional root system strength. This leads to an equation for the soil shear stress.

$$\tau_s = c_e \cdot \cos(\phi_e) + c_r + (\sigma - p_w) \cdot \sin(\phi_e) \quad 2-13$$

Where:

τ_s	= soil shear stress	[kPa]
c_e	= effective soil cohesion	[kPa]
ϕ_e	= effective internal friction angle	[°]
c_r	= root shear strength	[kPa]
σ	= soil normal stress	[kPa]
p_w	= pore water pressure	[kPa]

The root shear strength can be determined by Equation 2-14 according to Wu et al. (1979).

$$c_r = 1.2 \cdot \sigma_r \cdot \frac{A_r}{A} \quad 2-14$$

Where:

σ_r	= mean root tensile stress	[kPa]
$\frac{A_r}{A}$	= RAR or root area ratio	[-]

The critical mean grass normal stress is then determined by Equation 2-15, which is at maximum near the surface.

$$\sigma_{grass}(0) = \frac{A_r(0)}{A} \cdot \sigma_{r,c} \quad 2-15$$

Later, a so called turf-element model is introduced by Hoffmans et al. (2009) to investigate the forces acting on a cubic turf. This is shown in Figure 2-6 and the acting forces are described in Equation 2-16

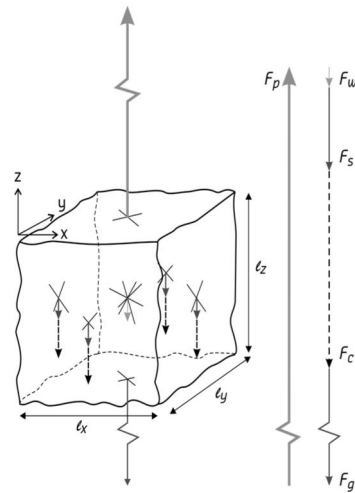


Figure 2-6: Turf-element model including forces

$$F_p \geq F_w + 4F_c + 4F_s + F_g$$

2-16

With:

$$\begin{aligned} 4F_c &= 2 \cdot (1-n) \cdot (C_{clay,c} + \tau_{grass,c}) \cdot (l_x + l_y) \cdot l_z && \text{= frictional force on the four side walls} \\ F_p &= p_m \cdot l_x \cdot l_y && \text{= maximum lift force} \\ F_g &= (1-n) \cdot [C_{clay,c} + \sigma_{grass,c}(z = -l_z)] \cdot l_x \cdot l_y && \text{= critical mean tensile force on the bottom} \\ F_w &= (1-n) \cdot (\rho_s - \rho) \cdot g \cdot l_x \cdot l_y \cdot l_z && \text{= submerged weight of the soil} \\ 4F_s &= (1-n) \cdot (\rho_s + \rho) \cdot \tan(\phi) \cdot g \cdot (l_x + l_y) \cdot l_z^2 && \text{= shear force on the four side walls} \end{aligned}$$

Where:

n	= porosity	[-]
$C_{clay,c}$	= critical clay cohesion	[kPa]
$\tau_{grass,c}$	= critical grass shear stress	[kPa]
$\sigma_{grass,c}$	= critical grass normal stress	[kPa]
$\tan\phi$	= friction factor	[-]
ρ_s	= material density	[kg/m ³]
ρ	= water density	[kg/m ³]

The formulations of Equation 2-16 can be rewritten to a formula that describes the incipient motion. The condition for moving turf is reached when the mean shear stress equals the critical mean shear stress. If both the submerged weight and the strength of clay are neglected with respect to the strength of grass, then it reduces to Equation 2-17 (Hoffmans, 2012).

$$\tau_0 \geq \tau_c = \psi_c \cdot [4\tau_{grass,c} + \sigma_{grass,c}(-\lambda_{ref})]$$

2-17

Where:

ψ_c	= critical Shields parameter ≈ 0.03	[-]
$\tau_{grass,c}$	= critical grass shear stress	[kPa]
$\sigma_{grass,c}$	= critical grass normal stress	[kPa]
λ_{ref}	= reference height for the initiation of erosion	[m]

This leads to a critical depth-averaged velocity, given the condition $U_0 = U_c$. Also the effect of suction pressures in unsaturated soils is included in Equation 2-18 (Hoffmans, 2012).

$$U_c = \frac{\alpha_{grass,U}}{r_0} \cdot \sqrt{\frac{\psi_c \cdot (\sigma_{grass,c}(0) - p_w)}{\rho}} \quad 2-18$$

Where:

U_c	= critical depth averaged velocity	[m/s]
$\alpha_{grass,U}$	= $\alpha_0 \cdot \sqrt{1 + 3 \cdot \alpha_{grass}} = 2.0$	[-]
p_w	= suction pressure in the roots	[kPa]

In these formulae the critical shear stress and the critical flow velocity are a function of the critical grass strength. It assumes the submerged weight and clay cohesion are negligible compared to this grass strength. The number of grass roots though, decreases with respect to the depth, but the cohesion obtained by the clay increases with respect to the depth. Therefore the formulae only apply to a small layer and the application is thus restricted to a length scale with a maximum of about 10-15 [cm].

2.4 Erosion models

The initiation of erosion can be approached by erosion models. This paragraph describes several erosion types and subsequent models that appear or can be applied for dike slopes. These might have potential to describe erosion around objects on the inner slope of a dike. These types are:

- Surface erosion: Erosion of small particles from the top layer.
- Block erosion: Erosion of a grass turf with a thickness of several centimetres.
- Wave impact erosion: Erosion due to the impact of a wave
- Geometrical transition erosion: Erosion at the geometrical transition between slopes and horizontal parts.
- Erosion around (non-)water retaining objects.

Finally a cumulative effective load model is discussed. The applicability of these models with respect to the transition between the earthen dike with grass cover and a structure is described in chapter 3.

2.4.1 Surface erosion

This type of erosion can be described as superficial erosion of the top grass layer. This erosion usually starts at a weaker spot in the grass cover. From there on, it continues to erode in a

downward direction. To model this erosion behaviour of a grass cover on the inner slope Verheij et al. (2010b) give Equation 2-19.

$$y_m = \frac{\sum_{i=1}^N ((0.7 \cdot \alpha \cdot U_{m,i} - U_c)^2) \cdot t_m}{E_{soil}} \quad 2-19$$

With:

$$E_{soil} = 6.15 \cdot 10^4 \cdot \frac{U_c^2}{\sqrt{g \cdot d_a}}$$

Where:

y_m	= maximum scour depth	[m]
$U_{m,i}$	= representative depth-averaged velocity at overtopping	[m/s]
U_c	= critical depth-averaged velocity	[m/s]
t_m	= representative wave time	[s]
E_{soil}	= erosion parameter	[m/s]
N	= number of waves	[-]
d_a	= 0.004	[m]
α	= turbulence constant ($= 1.5 + 5 \cdot r_0$)	[-]

The original model was called the EPM model by Van den Bos (2006). It is based on an infinite slope and it defines a bare spot as the location on the slope where erosion will be initiated. Later Hoffmans et al. (2009) expanded this model to not only link initiation of motion to a particular place but over the total slope. Still the EPM/Hoffmans model based his initiation of motion on irregularities on the surface of the grass layer. Therefore the load model, based on existence of holes, has been taken over from the EPM model.

Nevertheless, it appears surface erosion is not the most common reason for failure of the inner slope. In recent experiments this type of erosion did not occur often. Another point is that this model is based on a bare spot as location for the initiation of motion, but the location itself is not predicted. It is assumed that bare spots are always present somewhere in a grass cover.

2.4.2 Block erosion

The block erosion model is described with the use of the turf-element model of paragraph 2.3. Block erosion can be defined in a similar way as the model of the previous paragraph. Only in this case not small aggregates erode until a sudden depth is reached, but this entire 'block' erodes in one motion. To initiate this sudden erosion some form of fatigue is expected. The influence of fatigue on the critical flow velocity can be determined by Equation 2-20.

$$U_{0,c} = K_{v,u} \cdot U_c \quad 2-20$$

With:

$$K_{v,u} = \frac{1}{1.5 + 5 \cdot r_0} \left(\sqrt{\frac{\alpha_{soil} \cdot \lambda_{ref}}{t \cdot \sqrt{g \cdot d_a}}} + 1 \right)$$

Where:

$U_{0,c}$	= critical depth-averaged velocity	[m/s]
$K_{v,u}$	= fatigue factor	[-]
α_{soil}	= empirical constant = $6.15 \cdot 10^{-6}$	[m ² /s]
r_0	= relative depth-averaged turbulence intensity	[-]
t	= loading time	[s]
λ_{ref}	= reference height for the initiation of erosion	[m]

The sudden erosion process can be investigated with the turf-element model, which is discussed in paragraph 2.3.2. If these mechanisms occur the underlying soil could erode very fast by the subsequent erosion processes described in paragraph 2.4.

2.4.3 Wave impact erosion

Stanczak (2007) suggests that clay with and without cracks can fail due to shear stress failure. To verify this, experiments were performed on situations with and without a grass cover including an artificially induced crack. This was already theoretically investigated by Führböter in 1966, which resulted in the shear failure model of Figure 2-7. The tests from Stanczak showed a partly verification of the conceptual model. Thus he improved the model by including the dead weight of the soil and shear strength at both sides of the block, which eventually leads to the limit state Equation 2-21.

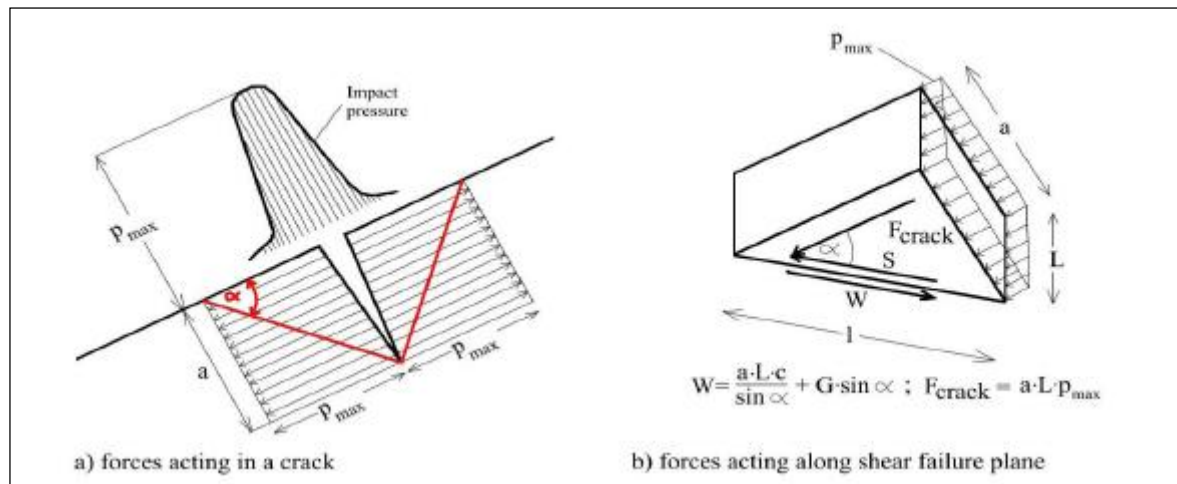


Figure 2-7: Shear failure model by Führböter (Stanczak, 2007)

$$\frac{a \cdot L_c \cdot c}{\sin(\alpha)} + \frac{0.5 \cdot a^2 \cdot L_c}{\tan(\alpha)} \cdot \rho \cdot g \cdot \sin(\alpha) + \frac{a^2 \cdot c}{\tan(\alpha)} = a \cdot L_c \cdot p_{max} \cdot \cos(\alpha) \quad 2-21$$

The above model considers all relevant resisting and mobilizing forces, but yet a failure prediction is impossible due to the fact that all resisting forces depend on the shape of the mobilized soil block.

The interesting thing about this model in relation to objects at the inner slope is the fact that at the connection between an object and the soil often a crack appears. An overtopping wave might force a water pressure into this crack at the upstream side of the object and thus makes this shear failure a probability.

Another model for impact erosion was determined by Mous (2010), this is based on uplift pressures and the critical pressures for initiation of erosion. Mous (2010) determined his model from the erosion model by Hoffmans et al. (2009). The main equation (3-1) of the model, was derived for wave impacts on the outer slope. The derivation of this formula is shown below.

$$y_m = \sum_{i=1}^N \frac{(p_{up,i}(z) - p_c(z)) \cdot t_{imp}}{E_p(z)} \quad 2-22$$

Where:

y_m	= maximum scour depth	[m]
$p_{up,i}(z)$	= representative uplift pressure	[N/m ²]
$p_c(z)$	= critical uplift pressure	[N/m ²]
t_{imp}	= representative wave impact time	[s]
$E_p(z)$	= erosion parameter	[m/s]
N	= number of waves	[-]

In Equation 3-1 the load term is given by $p_{up}(z)$, which represents the uplift pressure generated by the pressure gradient in the soil due to the wave impact. If this uplift pressure exceeds the critical uplift pressure of the soil, erosion will occur. This critical uplift pressure is given by $p_c(z)$ and derived from the turf element model of Hoffmans et al. (2009), which is shown in Equation 2-16. The model further contains a time factor (t_m) for the average wave impact time and an erosion parameter (E_p). This model is finally summed for every wave to determine a summed erosion depth.

The tensile grass strength is depth dependent and determined by Valk (2009) and Hoffmans (2012). In this case the equation of Valk (2009) is used by Mous (2010) to determine the critical uplift pressure, because it was assumed the strength reduces faster in comparison with the depth than the equation of Hoffmans does. This seems to be more in line with the fact that once the top layer is eroded, the underlying layer erodes even faster. In Equation 2-23 the critical normal strength of grass is determined for depth (z) based on the root normal strength and the amount of roots at a certain depth.

$$\begin{aligned} \sigma_{grass,c}(z) &= \sigma_{root,c} \cdot RAR_z \cdot e^{-\beta \cdot z} \\ \tau_{grass,c}(z) &= 1.2 \cdot \sigma_{root,c} \cdot RAR_z \cdot e^{-\beta \cdot z} \end{aligned} \quad 2-23$$

Where:

$\sigma_{root,c}$	= critical root normal stress	[Pa]
RAR_z	= root area ratio at depth z	[-]
β	= 22.32 (Valk, 2009)	[-]
z	= depth	[m]

If the parameters (l_x) and (l_y) in the turf element model are changed for a certain width (d) of a turf aggregate and (l_z) is changed for a depth (z), Equation 2-24 is determined. It shows a minimum value for the uplift pressure to initiate erosion. Thus the right hand side can be seen as ($P_c(z)$). In this equation the clay cohesion is neglected, because in the top 10 [cm] of the grass layer it is negligible.

$$p_{up} \geq (1-n) \cdot \left[(\rho_s - \rho_w) \cdot g \cdot z + \frac{\frac{1}{2} \cdot n_s \cdot \tan(\phi) \cdot (\rho_s - \rho_w) \cdot g \cdot z^2}{d} + \frac{n_s \cdot z \cdot \int_0^z \sigma_{grass,c}(z)}{d} + \sigma_{grass,c}(z) \right] \quad 2-24$$

In paragraph 2.4.1 the EPM/Hoffmans erosion model was discussed, including the erosion parameter (E_{soil}). This parameter depends on the strength of clay and grass, defined in the critical flow velocity, see Equation 2-25.

$$E_{soil} = \alpha_{soil} \cdot \frac{U_c^2}{\sqrt{g \cdot d_a}} \quad 2-25$$

Where:

α_{soil}	= $77 \cdot 10^3$	[-]
U_c	= critical depth-averaged velocity	[m/s]
d_a	= 0.004	[m]

The erosion model that is described in this paragraph is based on the critical uplift pressure (p_c). Therefore the parameter is rewritten into a formulation which includes (p_c). In Equation 2-26 the definition of (U_c) is given according to Hoffmans (2012). After entering the parameter values for (α_0), (α_τ) and the value of 0.15 for the turbulence intensity; the equation is rewritten so it only contains the critical uplift and suction pressures. In this formula the critical uplift pressure is taken equal to the soil strength.

$$\begin{aligned} U_c^2 &= \left(\frac{\alpha_0^2}{r_0^2} \right) \cdot \Psi_c \cdot (\sigma_s(0) - p_w) / \rho_w \\ &= \left(\frac{\alpha_0^2}{r_0^2} \right) \cdot \left(\frac{1}{\alpha_\tau \cdot \rho_w} \right) \cdot (p_c - (1-n) \cdot p_w) \\ &= \left(\frac{1.2^2}{0.15^2} \right) \cdot \left(\frac{1}{18 \cdot \rho_w} \right) \cdot (p_c - (1-n) \cdot p_w) \\ &= 3.56 \cdot \frac{(p_c - (1-n) \cdot p_w)}{\rho_w} \end{aligned} \quad 2-26$$

Where:

$$\begin{aligned}
 \alpha_0 &\approx 1.2 & [-] \\
 r_0 &= \text{relative turbulence intensity} & [-] \\
 \alpha_\tau &\approx 18 & [-] \\
 p_c(z) &= \text{critical uplift pressure} & [\text{Pa}] \\
 p_w &= \text{suction pressure in the roots} & [\text{Pa}]
 \end{aligned}$$

Equation 2-26 is inserted into Equation 2-25, from which Equation 2-27 follows. This final formulation for the erosion parameter includes a factor (ρ_s) to make the dimensions of the erosion model lead to an erosion rate in [m/s] again.

$$E_p = \alpha_{soil} \cdot \frac{\rho_s}{\rho_w} \cdot \frac{(\rho_c - (1-n) \cdot p_w)}{\sqrt{\Delta \cdot g \cdot d_a}} \quad 2-27$$

The coefficient (α_{soil}) in the erosion parameter is a calibration coefficient with a value of $5.5 \cdot 10^3$ [-].

2.4.4 Geometrical transition erosion

The transition from the inner dike slope to a horizontal area is often the area where scour holes appear. The water flow can be compared with a jet. Many people investigated jet scour holes and the two most appropriate formulae for the transition from a slope to a horizontal area are from Stein (Equation 2-28) and Hoffmans (Equation 2-29).

$$y_{m,e} = \frac{C_d^2 \cdot C_f \cdot \rho_w \cdot U_m^2 \cdot h_t}{\tau_c} \cdot \sin(S) \quad 2-28$$

$$y_{m,e} + h_t = U_{DL} \cdot \sqrt{\sin(s) \cdot \frac{q \cdot U_m}{g}} \quad 2-29$$

With:

$$U_{DL} = \frac{23}{\left(U_c \cdot \left(\frac{\Delta}{\nu \cdot g} \right)^{1/3} \right)^{1/2}}$$

Where:

$$\begin{aligned}
 y_{m,e} &= \text{maximum scour depth in the equilibrium phase} & [\text{m}] \\
 U_{DL} &= \text{characteristic parameter for soil strength} & [-] \\
 h_t &= \text{overtopping water depth} & [\text{m}] \\
 S &= \text{inner slope gradient} & [-] \\
 C_d &= \text{diffusion parameter} & [-] \\
 C_f &= \text{friction parameter} & [-] \\
 \Delta &= \text{relative density} & [-]
 \end{aligned}$$

Given these two formulae, the one from Stein is the most conservative one. Both formulae give an equilibrium scour depth though. The question remains if this equilibrium is reached in case of overtopping waves during a storm.

Quite recently Valk (2009) derived a model that showed the scour depth at time 't', see Equation 2-30.

$$\Delta y_m = \frac{\omega^2 \cdot \tau_0(z) - \tau_c(z)}{E_{soil}(z)} \cdot \Delta t \quad 2-30$$

With:

$$\omega = 1.5 + 5 \cdot r_0$$

The depth depended load follows from Equation 2-31.

$$\tau_0(z) = \tau_0 \cdot \exp(-0.25 \cdot z) \quad 2-31$$

With:

$$\tau_0 = 0.016 \cdot \frac{1}{2} \cdot \rho_w \cdot \left(\frac{1}{1 + \varepsilon} \cdot U_0 \right)^2$$

Where:

$$\varepsilon = \text{air concentration} \quad [-]$$

The critical strength of the soil follows from Equation 2-32.

$$\tau_{s,c} = 0.056 \cdot [(\rho_s - \rho) \cdot g \cdot z + \tau_{c,c}(z) + \tau_{g,c}(z)] \quad 2-32$$

With:

$$\tau_{c,c} = 0.021 \cdot \tau_{c,0} \cdot (1 + \alpha_{cs} \cdot z)$$

$$\tau_{g,c} = \tau_{g,0} \cdot \exp(-\beta \cdot z) = 0.7 \cdot RAR \cdot t_r$$

Where:

$$\tau_{c,c} = \text{depth dependent critical clay strength} \quad [\text{N/m}^2]$$

$$\tau_{g,c} = \text{depth dependent critical grass strength} \quad [\text{N/m}^2]$$

$$\tau_{c,0} = \text{critical clay strength at the surface} \quad [\text{N/m}^2]$$

$$\tau_{g,0} = \text{critical grass strength at the surface} \quad [\text{N/m}^2]$$

These formulae should give an indication for the scour depth at a certain time. The results show that the scour depth values at time 't', often appear to be larger than at the equilibrium depth found by Stein and Hoffmans.

2.4.5 Non-water retaining objects

As explained before, there are many kinds of non-water retaining objects. Maybe the most common objects on dike slopes are round poles. This can either be a bridge pier, tree, wooden pole or something else. In case of scour holes around bridge piers, some equations are determined. These can be expanded to trees, which lead to Equation 2-33.

$$\begin{aligned}\frac{y_{m,e}}{b} &= 2.0 \quad \text{in case of } b/h \ll 1 \\ \frac{y_{m,e}}{h_0} &= 1.9 \quad \text{in case of } b/h \cong 1\end{aligned}\tag{2-33}$$

Where:

b = diameter tree [m]

h_0 = water depth [m]

Another method to determine the scour depth around objects could be based on the turf-element model and the subsequent formulation for surface erosion, which is already discussed in paragraphs 2.3.2 and 2.4.1. In this case an increased value for the load introduced by the object should be used. This additional load is caused by flow concentration and turbulence. Also the strength of the grass cover should be changed in this equation. In this case it should be lower than the original strength, because there is less friction along for example wood or concrete and the grass quality is less due to a lack of sunlight.

2.4.6 Damage criteria

Van der Meer et al. (2010) discussed the cumulative effective load, which is a measure for the erosion on the inner slope. This is shown in Equation 2-34. The first term is a measure for the load; the second term is a measure for the strength. The damage number is determined by considering the number of waves and the flow velocity of the largest wave volumes and from observations after the hydraulic measurements. The definition of the three erosion states is shown in Figure 2-8 and can be described in the following way. First of all 'Failure' is the easiest definition: the sand core underneath the soil layer becomes free and damage develops fast. Secondly, 'Initial damage' is the first small hole in the grass cover, however, this depends heavily on the existence or non-existence of a weak spot on the slope surface. Finally, 'Damage at various locations' means that it does not depend solely on one weak spot, but it is certain damage occurs anyway at various locations on the slope. After this state, the various damaged locations expand until it reaches 'Failure'. This state is not referred to in the model and therefore leaves a gap, because it is a continuous changing situation which cannot be captured at one certain moment during the erosion process.

$$\begin{array}{ll}
 \text{Initial damage} & \sum_{i=1}^{i=N_t} (U_{m,i}^2 - U_c^2) \approx 500 \quad [\text{m}^2 / \text{s}^2] \\
 \text{Damage at various locations} & 500 \quad [\text{m}^2 / \text{s}^2] < \sum_{i=1}^{i=N_t} (U_{m,i}^2 - U_c^2) < 1500 \quad [\text{m}^2 / \text{s}^2] \quad 2-34 \\
 \text{Failure} & \sum_{i=1}^{i=N_t} (U_{m,i}^2 - U_c^2) > 3500 \quad [\text{m}^2 / \text{s}^2]
 \end{array}$$

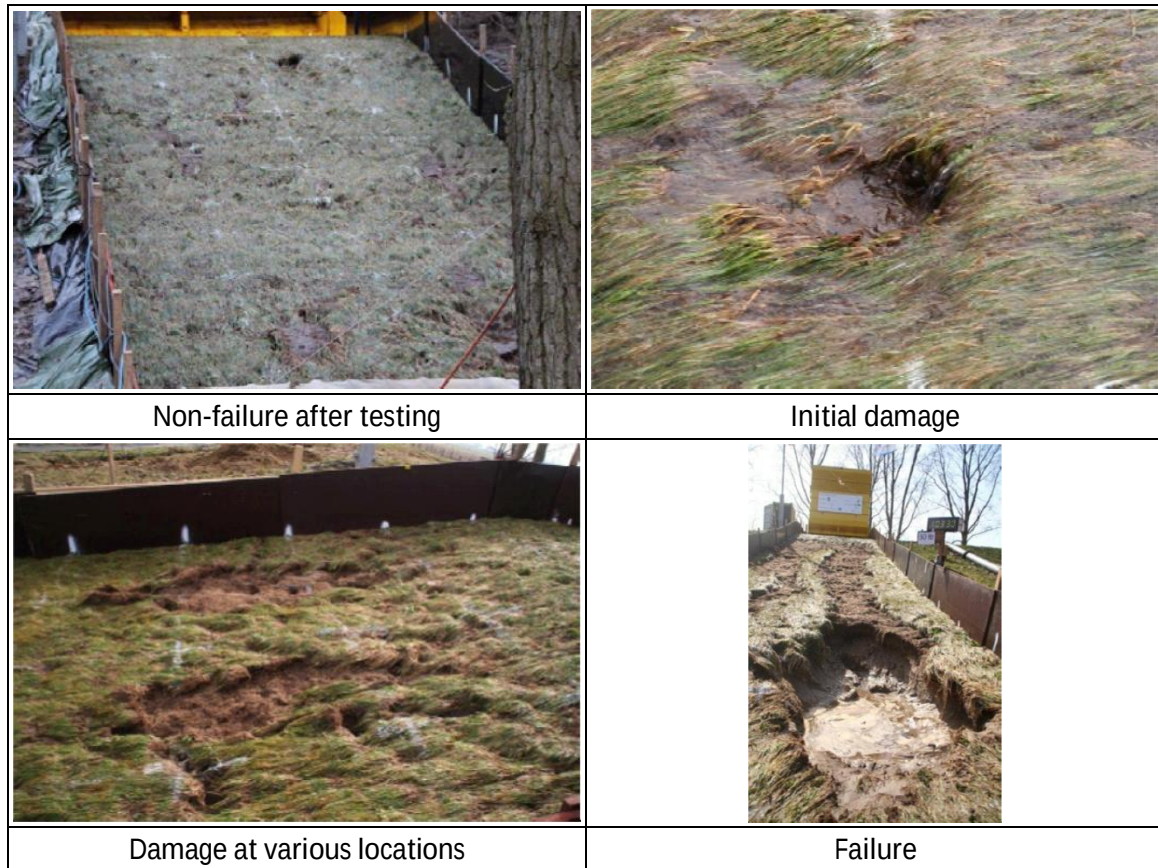


Figure 2-8: Illustrations of four states of erosion

3 Erosion modelling

The wide variety of erosion models described in the previous chapter shows the complexity of grass erosion in general. Close to (non-)water retaining objects, there are even more mechanisms that might influence the erosion rate. These mechanisms follow from the literature research in chapter 2 and are mainly the following:

1. Flow velocity
2. Water pressure
3. Flow depth
4. Turbulence
5. Water-air ratio
6. Grass quality
7. Root strength
8. Soil cohesion
9. Shear strength
10. Suction pressure

Variables one to five generally represent the load and variables six to ten the strength. To determine the most applicable model to describe the erosion process around objects, the location and governing mechanisms have to be defined; this is described in paragraph 3.1. Next, the best suitable erosion models can be determined from the models described in paragraph 2.4. The models defined for the two erosion zones are respectively described in paragraphs 3.2 and 3.3 and briefly summarized in paragraph 3.4.

3.1 Erosion location

Two generalized objects for erosion problems on inner slopes due to overtopping are shown in Figure 3-1. The location of the object can be either on the crest, on the slope or just after the slope and the dimension of the object can be either round (tree) or square (house). In addition, it should be noted that for the location after the transition between slope and horizontal it is assumed that the flow velocity just upstream of the object will be less compared to a location on the slope due to the presence of water on the horizontal.

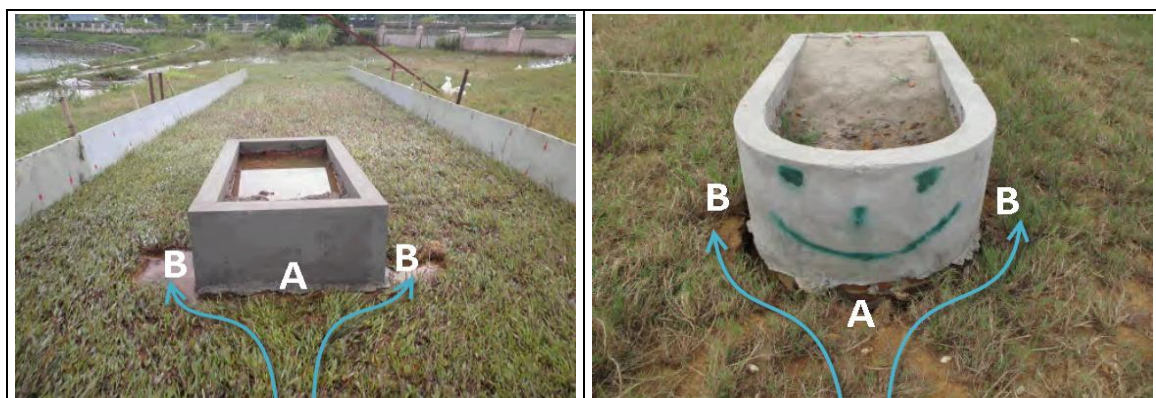


Figure 3-1: Generalized situation for erosion zones around objects (Le, 2012)

In Figure 3-1, two erosion zones are defined; zone A and zone B. These zones are determined from field experiments in the past (see appendix B), where it appeared that the first erosion occurred immediately in front of the object at zone A (see Figure 3-1 and Figure 3-2). Also the area next to the object, but mostly close to the upstream corner shows erosion in an early stage (see Figure 3-1); this is defined as zone B. The transition between both zones is still a vague area. It can be assumed that for the square object it will change close to the corner of the object, but for the round one it is still uncertain and depends on the diameter. This is further analysed in chapter 5.



Figure 3-2: Erosion pit in front of a tree during the field experiment at the Vechtdijk (Bakker et al., 2010)

3.2 Pressure erosion model

Zone A is the location in front of the object, where the flow velocity reduces to zero in the flow direction parallel to the slope. Besides this effect, the pressure on the bottom will significantly increase due to the wave impact and the velocity in the direction perpendicular to the slope will also increase. Therefore, the assumed important initiators on the load side of the problem are the turbulence and the water pressure (gradient). This is assumed to be similar for both the square and the round object, only it is likely the width of the area defined as zone A will be smaller for the round object.

Since the location in front of the object is loaded by a very limited flow velocity, but a large pressure; the erosion models of Mous (Mous, 2010) and Stanczak (Stanczak, 2007) from paragraph 2.4.3 seem most relevant to describe the erosion process. This is assumed, because the models define the impact pressure as the load on the grass cover and not the flow velocity. Nevertheless, both models were determined for wave impact erosion on an outer slope, so it has to be modified to fulfil the required aspects of a model for erosion at the impact zone of an object on the inner dike slope.

The differences between both models are mainly the boundary conditions. First of all, the model of Stanczak defines a crack in the cover layer where the impact pressure can penetrate easily.

The model of Mous does not, because it is based on the EPM/Hoffmans (Hoffmans et al., 2009) model of paragraph 2.4.1; which is actually a flow erosion model. This also leads to the second difference between both models, because the EPM/Hoffmans model defines the erosion depth as a continuous process for each overtopping wave. Oppositely, Stanczak defines the erosion depth in his model as a block with a given depth that gets torn out of the soil at one certain moment. Previously performed field experiments showed, besides the erosion location, also that erosion is most likely to occur in the form of larger grass turfs with dimensions up to 15x15x5 [cm³]. This was not only observed in front of objects, but also on the slope and at the transition between the slope and the toe. Therefore the EPM/Hoffmans and the therefrom derived Mous model do not describe the correct erosion process. This could be solved by using the cumulative load model of Van der Meer (Van der Meer et al., 2010a), which is described in paragraph 2.4.6. The model has many similarities with both the Mous and EPM/Hoffmans model, only does not describe the erosion depth as a continuous process for each overtopping wave. Instead it sums the load of all overtopping waves and gives criteria for several states of erosion. The two defined erosion models for zone A are described below and the subsequent load and strength term in these models are respectively elaborated in paragraphs 3.2.1 and 3.2.2.

Cumulative pressure load model

If no crack is present between soil and structure, which is likely during winter, the damage criteria model of Van der Meer seems the best option to describe the erosion process. As is stated above, the model has similarities with the Mous and EPM/Hoffmans model that are described in paragraph 2.4.3 and 2.4.6. However the load and strength of this model are given in velocities. These are not present in zone A, therefore the model is rewritten to a pressure erosion model as is shown in Equation 3-1 and Figure 3-3. It includes the summation of all waves that exert an uplift pressure on the soil that is larger than the critical pressure. The outcome value can be returned to three significant states of erosion: initial damage, damage at various locations and failure (see paragraph 2.4.6). First of all, 'Initial damage' is the first small hole in the grass cover, however, this depends heavily on the existence or non-existence of a weak spot on the slope surface. Secondly, 'Damage at various locations' means that it does not depend solely on one weak spot, but it is certain damage occurs anyway at various locations on the slope. Finally, 'Failure' is defined as the situation where the sand core underneath the soil layer becomes free and damage develops fast. The transition between the various damaged locations and the failure of the cover layer is not referred to in the model. This gap is left there, because it is a continuous changing situation which cannot be captured as one certain moment during the erosion process. Since the load and strength in front of the object differ from a location on the slope, additional load (α_L) and strength (α_s) parameters are added.

$$\begin{aligned}
 \text{Initial damage} & \sum_{i=1}^{i=N} (\alpha_{L,A} \cdot p_{u,i} - \alpha_s \cdot p_c) \approx R_1 \quad [\text{kPa}] \\
 \text{Damage at various locations} \quad R_1 \quad [\text{kPa}] & < \sum_{i=1}^{i=N} (\alpha_{L,A} \cdot p_{u,i} - \alpha_s \cdot p_c) < R_2 \quad [\text{kPa}] \\
 \text{Failure} & \sum_{i=1}^{i=N} (\alpha_{L,A} \cdot p_{u,i} - \alpha_s \cdot p_c) > R_3 \quad [\text{kPa}]
 \end{aligned} \tag{3-1}$$

Where:

$\alpha_{I,A}$	= load parameter in zone A	[-]
$p_{u,i}$	= uplift pressure of wave i	[kPa]
α_s	= strength parameter	[-]
p_c	= critical pressure	[kPa]
N	= number of waves	[-]
R	= damage criteria value	[kPa]

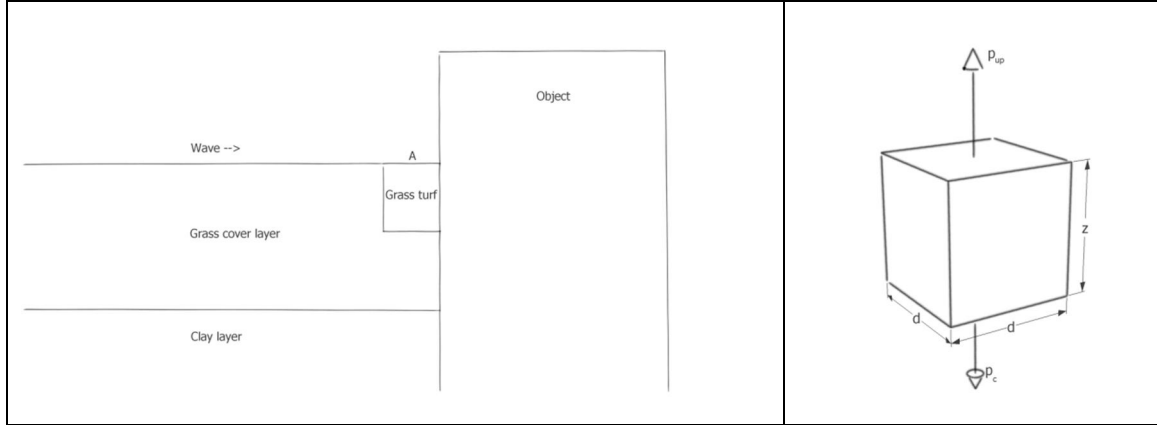


Figure 3-3: Left: Schematic side view at location 'A' (see Figure 3-1) Right: Schematic grass turf

Equation 3-1 includes the parameters (R_1), (R_2) and (R_3). The values for these parameters are found by Van der Meer for the velocity model. However, these can be determined for the pressure model with the relation shown in Equation 3-2.

$$r_0 = 1.2 \cdot \sqrt{\frac{1}{18} \cdot \frac{p_u}{\rho \cdot U^2}}$$

$$p_u = \frac{18 \cdot r_0^2 \cdot \rho}{1.2^2} \cdot U^2$$

$$p_u = 0.5 \cdot U^2$$
3-2

Where:

p_u	= uplift pressure	[kPa]
U	= loading velocity	[m/s]
r_0	= turbulence intensity = 0.2	[-]
ρ	= water density = 1000	[kg/m ³]

Applying this result for the parameter values found by Van der Meer, leads to Equation 3-3.

Initial damage	$\sum_{i=1}^{i=N} (\alpha_{I,A} \cdot p_{u,i} - \alpha_s \cdot p_c) \approx 250$	[kPa]	
Damage at various locations	250	[kPa]	$< \sum_{i=1}^{i=N} (\alpha_{I,A} \cdot p_{u,i} - \alpha_s \cdot p_c) < 750$
Failure	$\sum_{i=1}^{i=N} (\alpha_{I,A} \cdot p_{u,i} - \alpha_s \cdot p_c) > 1750$	[kPa]	

3-3

Where:

$\alpha_{I,A}$	= load parameter in zone A	[-]
$p_{u,i}$	= uplift pressure of wave i	[kPa]
α_s	= strength parameter	[-]
p_c	= critical pressure	[kPa]
N	= number of waves	[-]

Impact load model

If a crack is present between soil and structure, the model of Stanczak could be applied. However, this model is originally defined for wave impacts on the outer slope. In case it would be used to model erosion in front of a structure it looks like Figure 3-4. The modelled grass turf is loaded from the side by an impact pressure due to the presence of a crack. This presence is plausible, because the coherence between soil and for instance concrete is not very high. The impact pressure of the overtopping wave will penetrate into the crack and cause a pressure force on the grass turf. This pressure is resisted by the critical shear force of the soil until the initiation of erosion. In formulas these load and resistance would become as is shown in Equation 3-4.

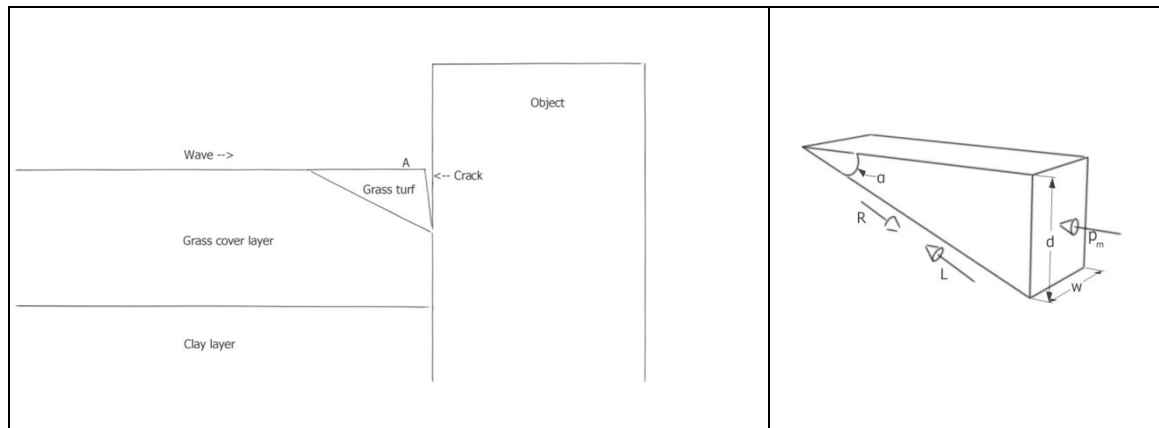


Figure 3-4: Left: Schematic side view at location 'A' (see Figure 3-1) Right: Schematic grass turf

$$L = d \cdot w \cdot p_m \cdot \cos(\alpha)$$

$$R = \frac{d^2 \cdot c}{\tan(\alpha)} + \frac{d \cdot w \cdot c}{\sin(\alpha)} + \frac{0.5 \cdot d^2 \cdot w}{\tan(\alpha)} \cdot \rho \cdot g \cdot \sin(\alpha) \quad 3-4$$

Where:

L	= loading shear force	[N]	d	= depth of eroding element	[m]
R	= resisting shear force	[N]	α	= angle of eroding element	[°]
p_m	= maximum impact pressure	[Pa]	c	= frictional strength	[Pa]
w	= width of eroding element	[m]	ρ	= soil density	[kg/m ³]

3.2.1 Load

In both of the models described above, the load is defined as a pressure. Nevertheless, there is a significant difference. The model of Stanczak is based on the maximum impact pressure, which in theory might be equal to the steady-state solution of Equation 3-5. However, this does not include any sonic effects due to the impact that are likely to influence the pressure. This is further elaborated in chapter 4.

$$p_m = \frac{1}{2} \cdot \rho \cdot v^2 \quad 3-5$$

Where:

p_m = hydrostatic pressure [Pa]
 ρ = water density [kg/m³]
 v = water velocity [m/s]

The model based on Van der Meer is based on upward directed pressure gradients, which are generated by the overtopping wave. These waves generate over and under pressures relative to the hydrostatic pressure, where the pressure gradient is defined as the amplitude of this dynamic pressure. This is schematically shown in Figure 3-5.

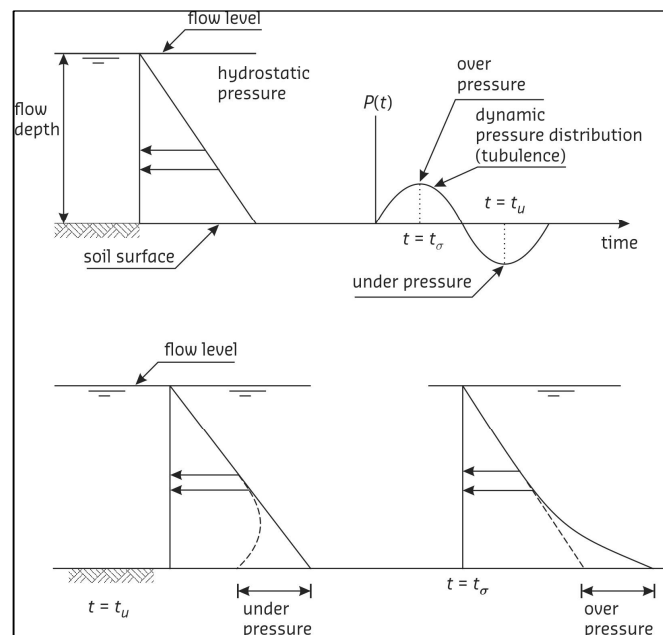


Figure 3-5: Dynamic and hydrostatic pressures during wave overtopping (Hoffmans, 2013)

3.2.2 Strength

In the original models of Mous and Van der Meer, the critical pressure is determined from the turf element model of Hoffmans. This model is discussed in paragraph 2.3.2 and the main equation is shown again in Equation 3-6. This term still has the different components acting on the grass turf, with from left to right: soil weight, shear force, frictional force and tensile

strength. This formulation already neglects the cohesion of the clay, because this is marginal in the top layer of the soil.

$$p_u \geq p_c$$

$$p_u \geq (1-n) \cdot \left[(\rho_s - \rho_w) \cdot g \cdot d + \frac{\frac{1}{2} \cdot n_s \cdot \tan(\phi) \cdot (\rho_s - \rho_w) \cdot g \cdot d^2}{w} + \frac{n_s \cdot z \cdot \int_0^d \sigma_{grass,c}(z) dz}{w} + \sigma_{grass,c}(z) \right] \quad 3-6$$

To determine the importance of each factor, some standard values for each variable and turf dimensions of 15x15x4 [cm] are entered, displayed in Table 3-1.

Table 3-1: Applied standard values

Variable	Value	Variable	Value
n	0.4 [-]	n_s	4 [-]
ρ_s	2000 [kg/m ³]	ϕ	35 [°]
ρ_w	1000 [kg/m ³]	$\int_0^d \sigma_{grass,c}(z) dz$	$8 \cdot 10^3$ [Pa]
g	9.81 [m/s ²]	$\sigma_{grass,c}(z)$	$5 \cdot 10^3$ [Pa]

After calculating, the following contribution for each factor appears:

- 3% = submerged weight of the soil
- 1% = shear force on the four side walls
- 61% = frictional force on the four side walls
- 35% = tensile force on the bottom

Therefore, it seems reasonable to reduce Equation 3-6 to Equation 3-7. This formulation, for now, still depends on the dimensions of the eroding grass turf.

$$p_c = (1-n) \cdot \left[\frac{n_s \cdot d}{w} \cdot \int_0^d \sigma_{grass,c}(z) dz + \sigma_{grass,c}(z) \right] \quad 3-7$$

Where:

- n = porosity [-]
- $\int_0^d \sigma_{grass,c}(z) dz$ = critical grass shear stress [Pa]
- $\sigma_{grass,c}(z)$ = critical grass normal stress at depth z [Pa]
- n_s = number of collaborating side walls [-]
- d = depth of the grass turf [m]
- w = width of the grass turf [m]
- z = depth [m]

The values for the critical uplift pressure, for these standard values, are shown in Table 3-2.

Table 3-2: Theoretical values for the critical uplift pressure

Number of side planes contributing [-]	Critical uplift pressure [kPa]
0	2.9
2	5.5
4	8.0

These results can be directly applied for the cumulative load model based on pressures, once the grass quality and its corresponding critical grass shear and normal stress are known. However, the Stanczak model (Equation 3-4) describes three terms within the resistance part. The first term is the frictional strength of the two sides, the second term is the frictional strength of the inclined plane and the third term is the weight of the soil. The factor (c) in these terms can be described as this frictional strength of the grass roots. This is determined by using Equation 3-8, which is the critical grass shear strength from the turf-element model.

$$c = \int_0^d \sigma_{grass,c}(z) dz = \int_0^d \sigma_{root,c} \cdot RAR_0 \cdot e^{-\beta \cdot z} dz \quad 3-8$$

Where:

- $\sigma_{root,c}$ = critical root normal stress [Pa]
 RAR_0 = root area ratio at surface [-]
 β = 22.32 (Valk, 2009) [-]
 d = depth of eroding element [m]

The applicability and validity of these formulae is described in chapter 4 and further analysed in chapter 5.

3.3 Flow erosion model

Zone B is as earlier described the area next to the object and near the upstream corner. In contrast to zone A, this location has a velocity component in the direction parallel to the slope. Therefore it is possible to apply the original model of Van der Meer (Van der Meer et al., 2010a) for this location. Furthermore it is assumed the initiation of erosion in this area is caused by the velocity differences between the slope and next to the object. Therefore also this model is expanded with a load and strength parameter. The model is shown in Equation 3-9.

$$\begin{aligned}
 \text{Initial damage} & \sum_{i=1}^{i=N_1} (\alpha_{l,B} \cdot U_{m,i}^2 - \alpha_s \cdot U_c^2) \approx 500 \quad [\text{m}^2/\text{s}^2] \\
 \text{Damage at various locations} \quad 500 \quad [\text{m}^2/\text{s}^2] & < \sum_{i=1}^{i=N_1} (\alpha_{l,B} \cdot U_{m,i}^2 - \alpha_s \cdot U_c^2) < 1500 \quad [\text{m}^2/\text{s}^2] \quad 3-9 \\
 \text{Failure} & \sum_{i=1}^{i=N_1} (\alpha_{l,B} \cdot U_{m,i}^2 - \alpha_s \cdot U_c^2) > 3500 \quad [\text{m}^2/\text{s}^2]
 \end{aligned}$$

Where:

$\alpha_{I,B}$	= load parameter in zone B	[-]
$U_{m,i}$	= maximum velocity of wave i	[m/s]
α_s	= strength parameter	[-]
U_c	= critical velocity	[m/s]
N	= number of waves	[-]

3.3.1 Load

The load term in the flow erosion model is defined as the maximum velocity of an overtopping wave. This value is multiplied by a load factor in case of the presence of a structure. It is assumed the velocity will increase near the corner of the object, which is determined with a numerical model in paragraph 4.1.

3.3.2 Strength

The strength term consists of the critical velocity, which is deduced from the turf-element model (Hoffmans, 2012). This is previously described in paragraph 2.3.2 and the final formula is shown again in Equation 3-10. The critical velocity is multiplied by a strength factor in case of the presence of a structure. This factor should be similar to the factor introduced in paragraph 3.2.2, however, if it is placed in front of the critical velocity it should be paraphrased.

$$U_c = \frac{\alpha_{grass,U}}{r_0} \cdot \sqrt{\frac{\psi_c \cdot (\rho_c - \rho_w)}{\rho}} \quad 3-10$$

Where:

U_c	= critical depth averaged velocity	[m/s]
$\alpha_{grass,U}$	= $\alpha_0 \cdot \sqrt{1 + 3 \cdot \alpha_{grass}} = 2.0$	[-]
r_0	= turbulence intensity	[-]
ψ_c	= critical Shields parameter ≈ 0.03	[-]
ρ_c	= critical pressure	[Pa]
ρ_w	= suction pressure in the roots	[Pa]
ρ	= liquid density	[kg/m ³]

3.4 Summary

A summary of the models and its parameters that are determined in this chapter for zone A and zone B, are shown in Figure 3-6, Figure 3-7 and Figure 3-8.

Cumulative pressure load model			
Initial damage		$\sum_{i=1}^{i=N_1} (\alpha_{I,A} \cdot p_{u,i} - \alpha_s \cdot p_c) \approx 250$	[kPa]
Damage at various locations	250	$\sum_{i=1}^{i=N_1} (\alpha_{I,A} \cdot p_{u,i} - \alpha_s \cdot p_c) < 750$	[kPa]
Failure		$\sum_{i=1}^{i=N_1} (\alpha_{I,A} \cdot p_{u,i} - \alpha_s \cdot p_c) > 1750$	[kPa]
Strength term :		Boundary condition :	
$p_c = (1-n) \cdot \left[\frac{n_s \cdot d}{w} \cdot \int_0^d \sigma_{grass,c}(z) dz + \sigma_{grass,c}(z) \right]$ $\sigma_{grass,c}(z) = \sigma_{root,c} \cdot RAR_0 \cdot e^{-\beta \cdot z}$		$p_{u,i} > p_c$	
Where:			
$\alpha_{I,A}$	= load parameter in zone A	[-]	p_c = critical pressure [kPa]
$p_{u,i}$	= uplift pressure of wave i	[kPa]	N = no. of overtopping waves [-]
α_s	= strength parameter	[-]	V = wave volume [m ³ /m]
$\sigma_{grass,c}(z)$	= critical grass normal stress	[Pa]	n = porosity [-]
$\sigma_{root,c}$	= critical root normal stress	[Pa]	d = depth of the grass turf [m]
RAR_0	= root area ratio at surface	[-]	w = width of the grass turf [m]
n_s	= number of side walls	[-]	z = depth [m]
β	= 22.32 (Valk, 2009)	[-]	

Figure 3-6: Zone A - Pressure erosion model

Impact load model

$$L = d \cdot w \cdot p_m \cdot \cos(\alpha)$$

$$R = \frac{d^2 \cdot c}{\tan(\alpha)} + \frac{d \cdot w \cdot c}{\sin(\alpha)} + \frac{0.5 \cdot d^2 \cdot w}{\tan(\alpha)} \cdot \rho \cdot g \cdot \sin(\alpha)$$

With:

$$c = \int_0^d \sigma_{grass,c}(z) dz = \int_0^d \sigma_{root,c} \cdot RAR_0 \cdot e^{-\beta \cdot z} dz$$

Where:

L	= loading shear force	[N]	α	= angle of eroding element	[°]
R	= resisting shear force	[N]	ρ	= soil density	[kg/m ³]
p_m	= maximum impact pressure	[Pa]	w	= width of eroding element	[Pa]
c	= soil cohesion	[Pa]	d	= depth of eroding element	[-]
$\sigma_{root,c}$	= critical root normal stress	[m]	β	= coefficient (Valk, 2009)	[-]
RAR_0	= root area ratio at surface	[m]	z	= depth	[m]

Figure 3-7: Zone A - Pressure erosion model

Cumulative velocity load model		
Initial damage		$\sum_{i=1}^{i=N_t} (\alpha_{l,B} \cdot U_{m,i}^2 - \alpha_s \cdot U_c^2) \approx 500 \quad [\text{m}^2/\text{s}^2]$
Damage at various locations	500 $[\text{m}^2/\text{s}^2]$	$< \sum_{i=1}^{i=N_t} (\alpha_{l,B} \cdot U_{m,i}^2 - \alpha_s \cdot U_c^2) < 1500 \quad [\text{m}^2/\text{s}^2]$
Failure		$\sum_{i=1}^{i=N_t} (\alpha_{l,B} \cdot U_{m,i}^2 - \alpha_s \cdot U_c^2) > 3500 \quad [\text{m}^2/\text{s}^2]$
Strength term:		Boundary condition :
$U_c = \frac{\alpha_{grass,U}}{r_0} \sqrt{\frac{\psi_c (p_c - p_w)}{\rho}}$		$U_{m,i} > U_c$
Where:		
$\alpha_{l,B}$	= load parameter in zone B	[-]
$U_{m,i}$	= maximum velocity of wave i	[m/s]
U_c	= critical depth averaged velocity	[m/s]
$\alpha_{grass,U}$	= $\alpha_0 \sqrt{1 + 3\alpha_{grass}}$	= 2.0
r_0	= turbulence intensity	[-]
ψ_c	= critical Shields parameter	[-]
α_s	= strength parameter	[-]
U_c	= critical velocity	[m/s]
p_c	= critical pressure	[Pa]
p_w	= suction pressure roots	[Pa]
ρ	= liquid density	[kg/m ³]

Figure 3-8: Zone B - Flow erosion model

4 Numerical and experimental results

Several field experiments are performed and a numerical model is used to determine whether the assumptions and erosion models of the previous chapter are plausible. This chapter holds the results that are obtained from these experiments on the area of both the load and the strength. Paragraph 4.1 focusses on the results of the numerical model, which is mainly about the loading side. Paragraph 4.2 focusses on the field experiments, which are about both the strength as the load side. These numerical and experimental results are analysed with respect to each other and to the erosion models of chapter 3. This is described in the analysis of chapter 5.

4.1 Numerical model

As stated in chapter 3, the load in zone A consists of either the impact pressure or the pressure gradient and in zone B it consists mainly of the flow velocity. To gain some insight into the pressures and velocities that are loading the grass cover around the object a numerical model is used. The pressure research, focussed on the area in front of the object (zone A), is discussed in paragraph 4.1.1. The effects of the width of a certain object on the exerted pressures are described in paragraph 4.1.2 and finally the velocity effects around the object and on the slope are treated in paragraph 4.1.3.

Initially two models were reviewed to fit this purpose: ComFLOW V3.0 and SWASH. SWASH (an acronym of Simulating Waves till Shore) is a non-hydrostatic wave-flow model and is intended to be used for predicting transformation of surface waves from offshore to the beach for studying the surf zone and swash zone dynamics, wave propagation and agitation in ports and harbours, and rapidly varied shallow water flows in coastal waters. ComFLOW on the other hand is a Volume-of-Fluid (VOF) model developed by Luppés (Luppés et al., 2010). It solves the incompressible Navier-Stokes equations and the free-surface motion. In other words, it is a simulation method for free-surface flow in terrestrial and micro-gravity environments. It models viscous incompressible flow in and around arbitrary geometries. At the free surface, continuity of stresses is imposed; effects of capillarity are included. Also liquid-solid body interaction is included in ComFLOW. To model an overtopping wave on an inner slope, the model ComFLOW seemed to be the better choice and is therefore applied. The focus of SWASH is mainly on waves near the beach, where an overtopping wave is something entirely different. ComFLOW however, focusses more on the water flow itself. Therefore it seems to have possibilities to produce a simplified situation of an inner dike slope including an object, which would give some insight into the pressures and velocities.

The layout of the numerical model is based on the one used for the field experiment, which is schematically shown in Figure 4-1. A three-dimensional impression of the model and a simulated wave seen from the side are shown in Figure 4-2. The slope is 1:3 and the object has a width of 1.5 [m] of the 4.0 [m] total model width. The grid cells that are used for the calculations have dimensions of 10x10x10 [cm³].

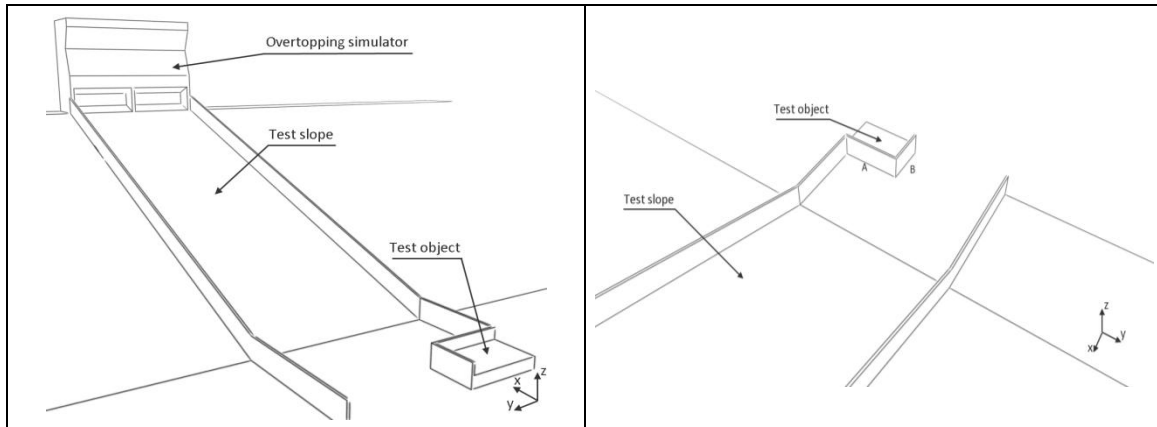


Figure 4-1: Schematic impression of the field experiment, including erosion zone A and B

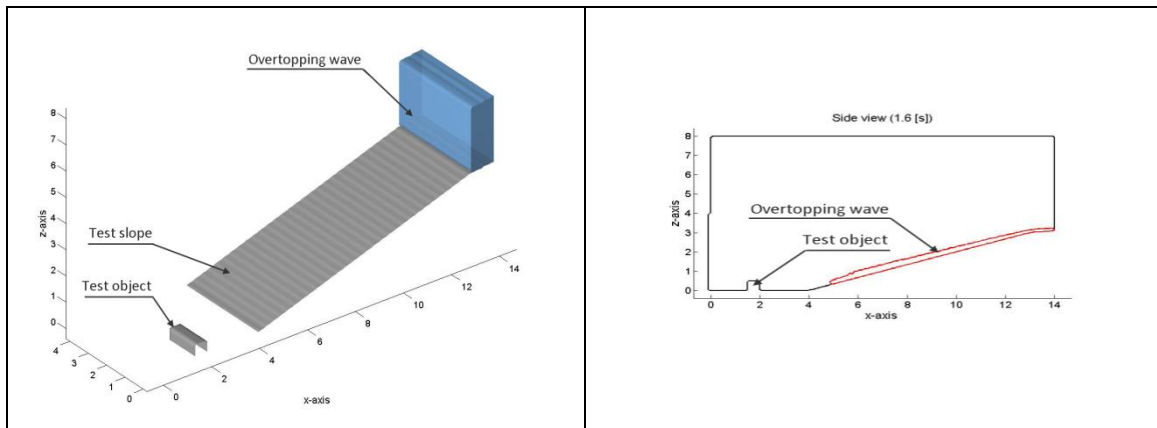


Figure 4-2: 3D and 2D impression of the ComFLOW model

It was said before that the cumulative load erosion model shown in chapter 3 is a summation of the 'relevant' waves overtopping a dike during a storm. The tested average discharges in the field experiment of Nijmegen are 1, 10, 50, 100 and 200 [l/m/s]. The waves used to simulate these discharges have volumes from about 20 [l/m] up to 4500 [l/m]. Several of these are modelled in ComFLOW to determine several flow aspects that load the grass cover just in front of an object. Before the results are shown the applicability of the model is checked. It is determined whether the velocities and flow depths found on the inner slope within ComFLOW are similar to the values found in earlier field experiments (Bakker et al., 2010). An overview of these results is shown in Table 4-1 and a visual presentation of a modelled wave is shown in Figure 4-3.

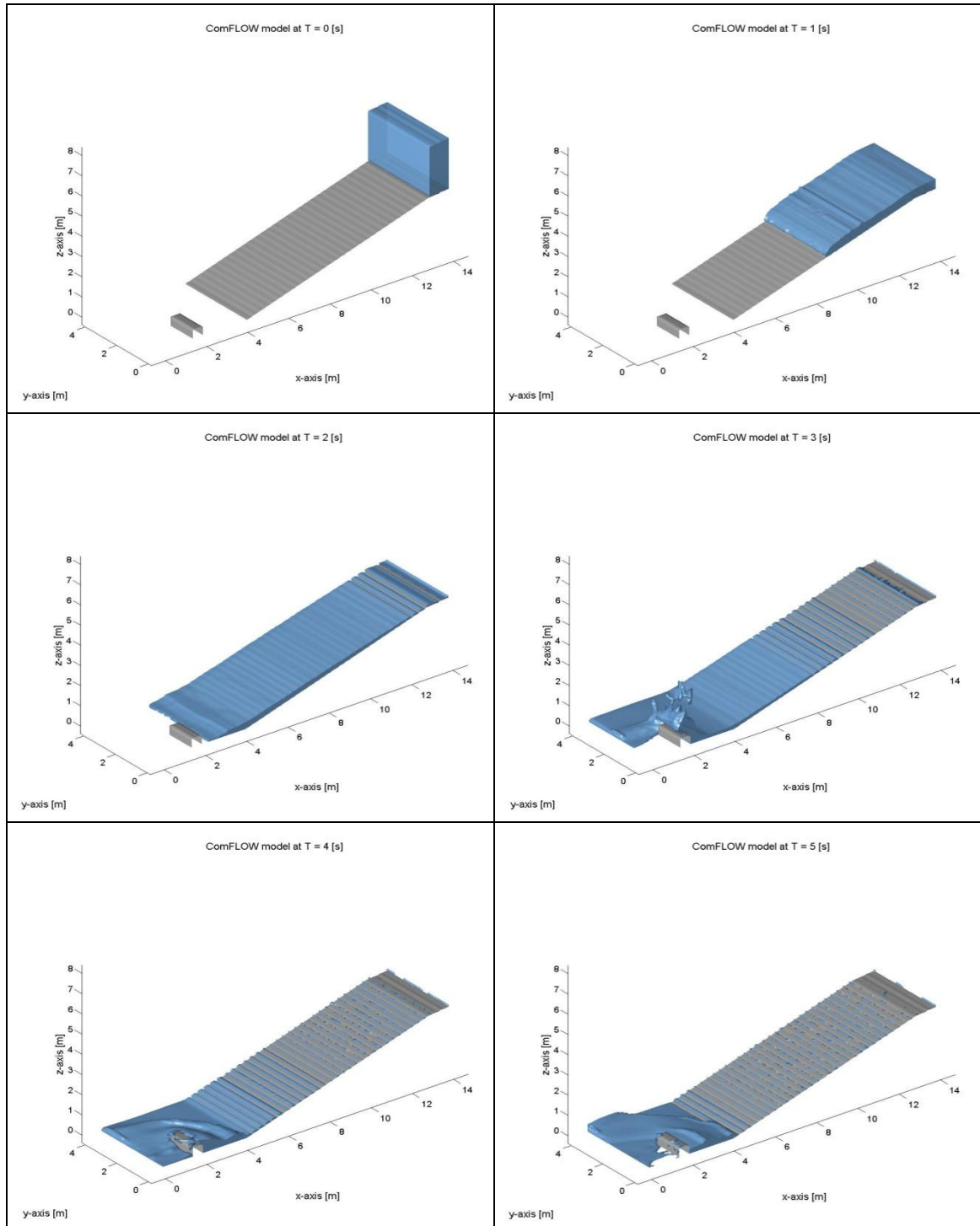


Figure 4-3: 3D Overview of the ComFLOW model at several time steps for a wave of 2500 [l/m]

Table 4-1: Characteristic wave values of field experiments (Bakker et al., 2010) and ComFLOW modelling

	Deltares measurements		ComFLOW results	
Vol. [l/m]	Wave height [m]	Wave velocity [m/s]	Wave height [m]	Wave velocity [m/s]
750	0.08	4.1	-	-
1500	0.14	5.1	0.16	4.0
2500	0.20	5.9	0.23	6.3
3500	0.26	6.7	0.36	7.0
4500	0.31	7.1	0.42	7.5

First of all, the ComFLOW results show no value for the wave volume of 750 [m/s]. The reason for this is that the small water volume got mostly 'stuck' on the slope and very little water actually reached the object. The velocities and wave heights shown are measured on the lower part of the slope (see Figure 4-4) for both Deltares' field measurements and the results from ComFLOW. According to the field measurements, the velocity should be slightly higher for a 1500 [l/m] wave. This is most likely caused by the fact that some water remains on the slope due to its cascading shape. The shape should give a representative situation for a rough grass cover under larger wave loads.

Table 4-1 also shows this; the velocities correspond well. However, the wave heights found in the model are quite larger for the larger wave volumes. This is probably influenced by the fact the modelled water volume has no restriction at his initial position; like the overtopping simulator has an outflow opening. It can be concluded the ComFLOW model does not show a correct reflection of the reality. However, the differences are accepted and the model is used to perform several analyses.

The following paragraphs show pressures and velocities found in the ComFLOW model. To clarify the applied measurement locations and cross-sections in the model, these are shown in Figure 4-4.

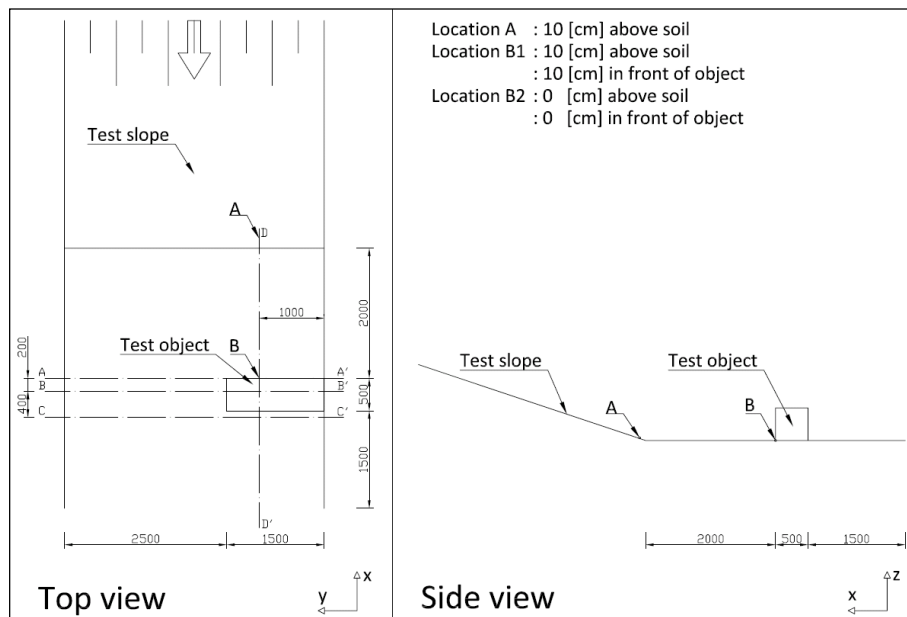


Figure 4-4: Measurement points and cross-sections for ComFLOW analyses

4.1.1 Pressure in front of object

According to the literature research and the defined models of the previous chapters, the occurring pressures in front of the object are of importance with respect to the erosion. The pressure distributions found in the numerical model are shown in Figure 4-5 for measurement location B2.

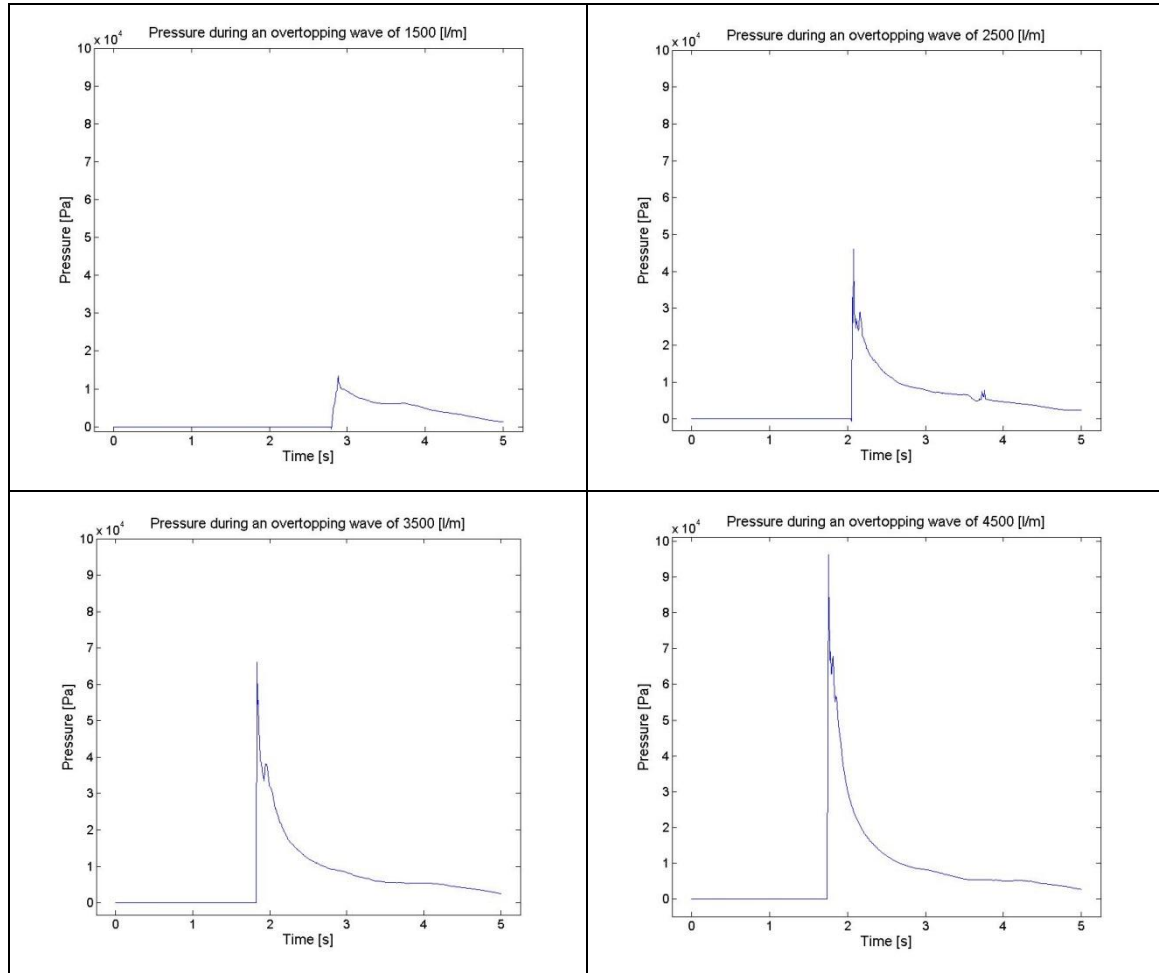


Figure 4-5: Pressure distribution from measurement location B2 for several overtopping volumes

Especially for large waves it is clearly visible the wave consists of a very short sonic peak (time frame in milliseconds), a broader peak (time frame in tenths of seconds) and a reduction towards zero (time frame in multiple seconds). This broader peak or hydrostatic pressure should correspond with a steady state situation calculated with Equation 4-1. This appears not to be right and the difference might be in the actual velocity in front of the object, instead of the bottom of the slope. The results, including these actual velocities, are shown in Table 4-2.

$$p = \frac{1}{2} \cdot \rho \cdot v^2$$

4-1

Where:

p = hydrostatic pressure [Pa]
 ρ = water density [kg/m³]
 v = water velocity [m/s]

Table 4-2: Velocity differences on and off slope by ComFLOW

Wave volume [l/m]	Velocity at bottom slope (measurement A) [m/s]	Velocity in front of object (measurement B1) [m/s]	Theoretical hydrostatic pressure (measurement B1) [kPa]
1500	4.0	4.0	8.0
2500	6.3	7.0	24.5
3500	7.0	9.5	45.0
4500	7.5	12.0	72.0

The theoretical values correspond quite well with the hydrostatic part of the ComFLOW pressure distributions in Figure 4-5. Nevertheless, these velocities are not realistic for a field location, because the lack of friction on the flat area shoots the velocities to high values.

The field experiments, which are discussed in paragraph 4.2, actually show a velocity reduction on the horizontal area. This was not only the effect of friction, but also due to the water that remained relatively permanent in the flat area. Therefore, this pool of water is also introduced in the model, which gave the results shown in Figure 4-6 and changed the values of Table 4-2 into Table 4-3.

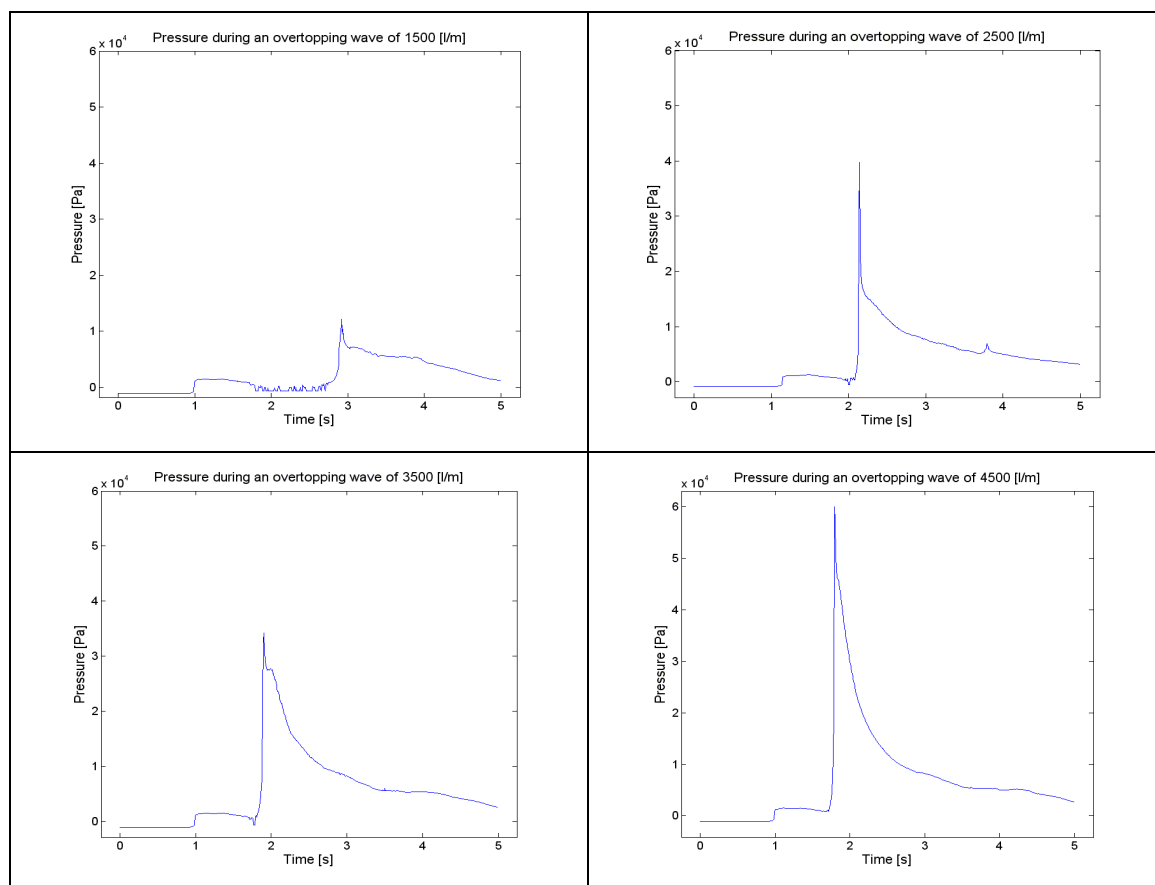


Figure 4-6: Pressure distribution from measurement location B2 including water in front of the object

Table 4-3: Velocity differences on and off slope including water in front of the object by ComFLOW

Wave volume [l/m]	Velocity at bottom slope (measurement A) [m/s]	Velocity in front of object (measurement B1) [m/s]	Theoretical hydrostatic pressure (measurement B1) [kPa]
1500	4.0	3.5	6.0
2500	6.3	5.8	17.0
3500	7.0	7.1	25.0
4500	7.5	9.2	43.0

Now the velocities found in the model and in previous field experiments (see Table 4-1) correspond reasonably. Only the largest wave still shows a significant too large increase in velocity on the horizontal part. This was not seen in the latest field experiments, where it remained more or less equal. This is further discussed with respect to the latest field experiment in paragraph 5.1. Also the theoretical hydrostatic pressure and the values of the part just after the sonic pressure peak are corresponding quite well. It also appears this sonic peak is significantly reduced in case there is still water in front of the object.

It was assumed in chapter 3 that the pressure would be high close to the object, with a fast reduction over the z-direction (see Figure 4-2). This should be seen also in the ComFLOW model, which is shown in Figure 4-7. Clearly, the reduction of the sonic peak is within several decimetres. Due to the grid size of the model, this is a quite rough result. A finer and significantly longer model could be run for further analysis.

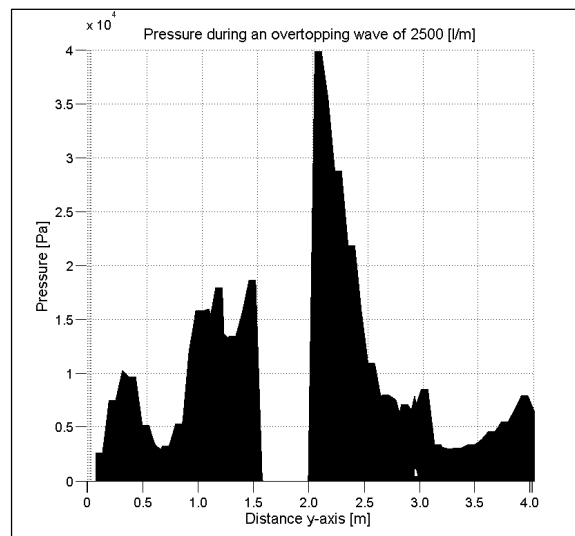


Figure 4-7: Pressure distribution in cross section D-D'

4.1.2 Effect of the object width on the pressure

Over the width of the structure the pressure is distributed quite evenly, with a reduction of the occurring pressure near the corner; located at 1.5 [m] from the right side. This is displayed in Figure 4-8. It can be observed the reduction starts at about 20 [cm] from the corner of the object, which means the maximum pressure is never reached when an object is smaller than this value. It can be concluded that small objects will not have a big influence on the surrounding

grass stability. However, this heavily depends on the occurring wave load. A large wave could still cause enough load to damage the grass cover, although it is less load than it would be in case of a larger object.

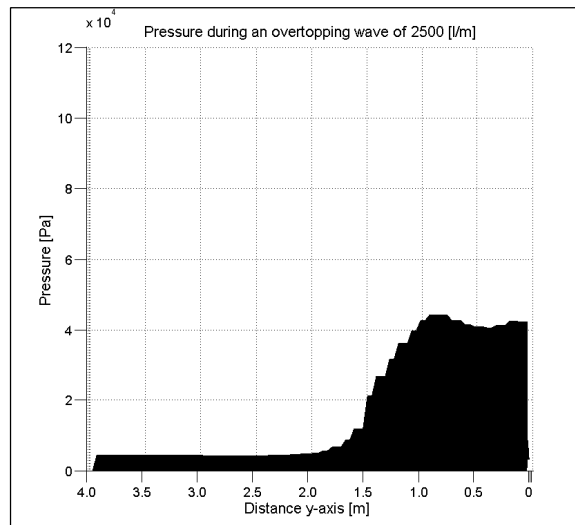


Figure 4-8: Pressure distribution in front of the object (cross-section A-A')

The effects of the dimensions of the object are generalized in Table 4-4. These results are based on pressure plots of multiple wave volumes and several model runs, see appendix D. Here the pressure at a certain distance from the corner of an object is divided by the maximum pressure in front of an object.

Table 4-4: Pressure load reduction factor for object dimensions

Object dimension [m]	Reduction factor [-]
0.1	0.3
0.3	0.4
0.5	0.9
>1.0	1.0

4.1.3 Effect of the object on the velocity

The potential erosion locations around objects are stated as in front of the object and around the corners. It is assumed the increase in flow velocity next to the object is of influence on the erosion at these locations. To verify this assumption the velocity is investigated in the model, ComFLOW. The results for several cross-sections are shown in Figure 4-9. Multiple plots for other wave volumes are shown in appendix D. The results show hardly any difference in the local flow velocity at the sides of the object; right after the impact. It is even possible to state that the velocity reduces along and after the object. Nevertheless, these local decelerations found in the ComFLOW model are quite small.

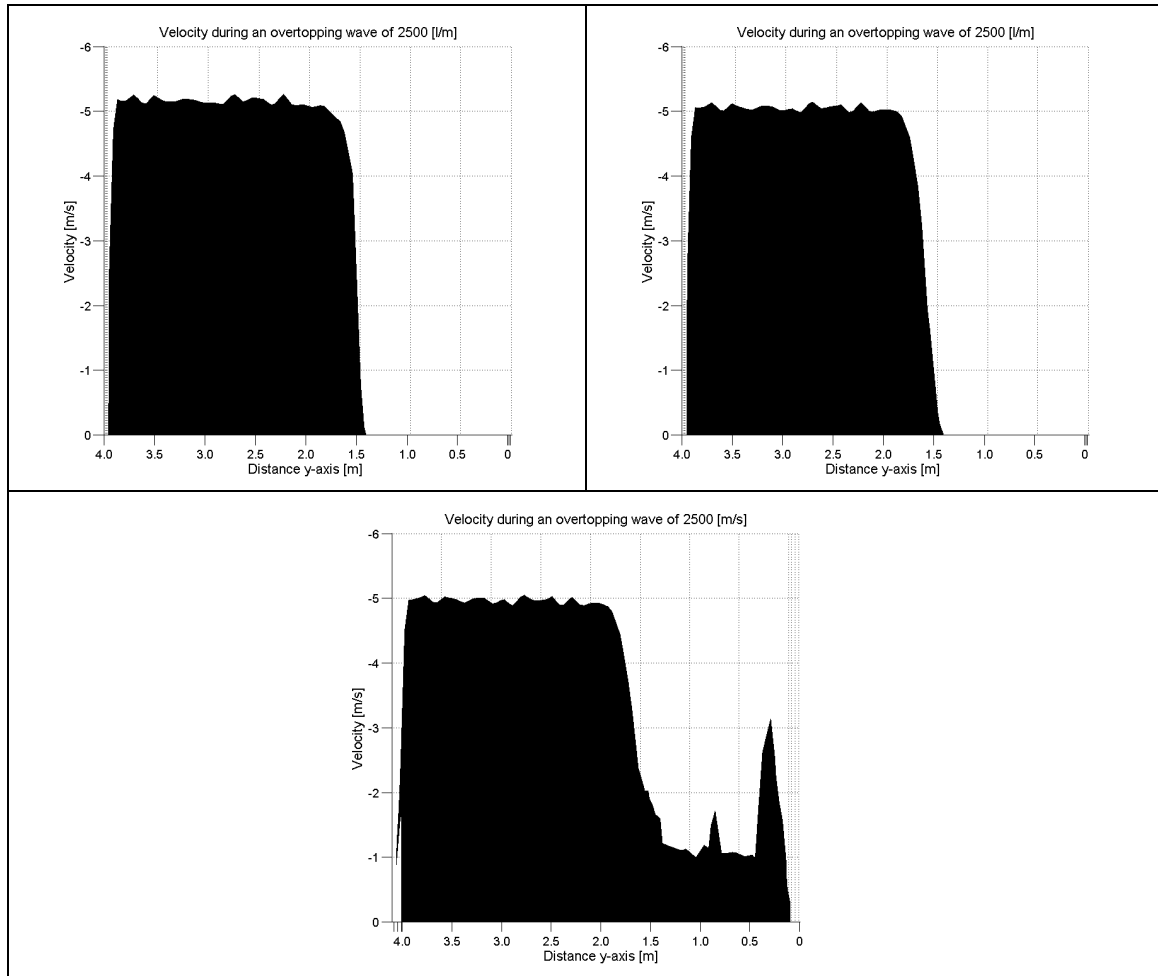


Figure 4-9: Velocities in cross-sections A-A', B-B' and C-C'

4.2 Field experiments

This paragraph describes the results of two field experiments that are performed from January till March of 2013. First an overtopping experiment in paragraph 4.2.1, followed by a grass tensile strength test in paragraph 4.2.2.

4.2.1 Overtopping

In appendix B, some previously performed field experiments are discussed. These were followed by another experiment in January and February of 2013 near Nijmegen. The main goal of these experiments was to gain more insight into the strength of grass covers during several overtopping conditions. Two different slopes were tested at this location. Strip one was a slope (1:2) next to a concrete object and strip two was a slope (1:3) with an object just after the toe (see Figure 4-11). This second strip is similar to the one that is used for the numerical model and is shown again in Figure 4-10. The visual results of the erosion on the slope of test strips one and two are described in appendix E, along with some pictures of the experiment.

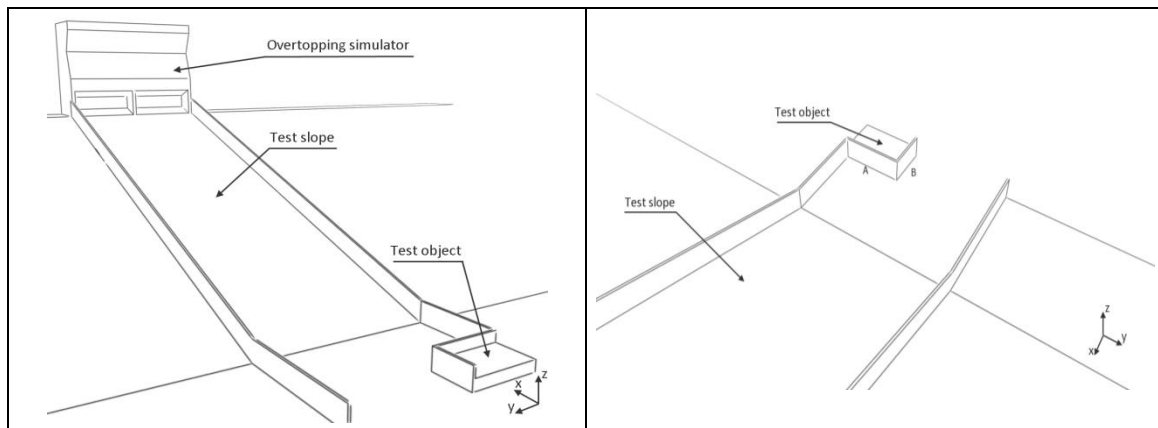


Figure 4-10: Schematic overview of the field experiment with erosion zone A and B



Figure 4-11: Overview field experiments on test strip one and two

The main goal of the experiment with respect to this thesis is to gain more insight into the erosion around the object in the second test strip. However, it appeared there was already a fairly large crack present between the soil and the object (10 [cm]). Therefore the first 15 [cm] in front of the object was replaced with concrete. This made it impossible to draw any inferences to the erosion rate around the object, but did create other possibilities. To gain some insight into the load exerting on the grass cover in front of the object; two pressure gauges are placed on the positions 1 and 2 shown in Figure 4-12. These field measurements could verify the numerical model and give information on the quantification of the pressure forces, gradients and thus the modelled uplift force. Besides measurements, also visual observations of the flow movement could contribute to the theoretical model.

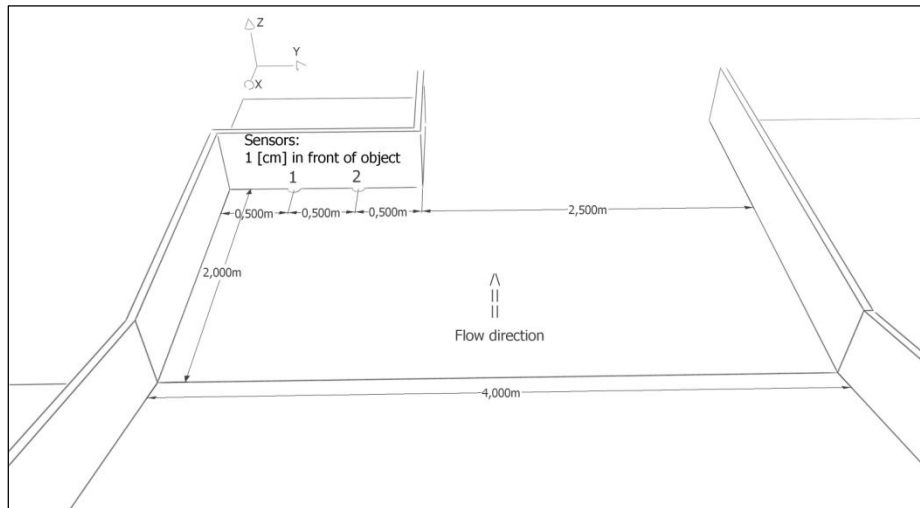


Figure 4-12: Detail of the locations of sensor 1 and 2

Before the results of the pressure gauges are given, several waves of (almost) the same volume are compared of which two are plotted in Figure 4-13 to see if they show similarities. The values of sensor 1 appear to be quite similar and even the peaks show some similarities. Sensor 2 shows some more differences, especially towards the tail. The reason of these differences can probably be found in the location of the sensor. Due to the corner with the dead end in the test strip, the effects of the previous wave can be seen in the measurements of the current wave. It seems the higher initial water level reduces the measured turbulence effects.

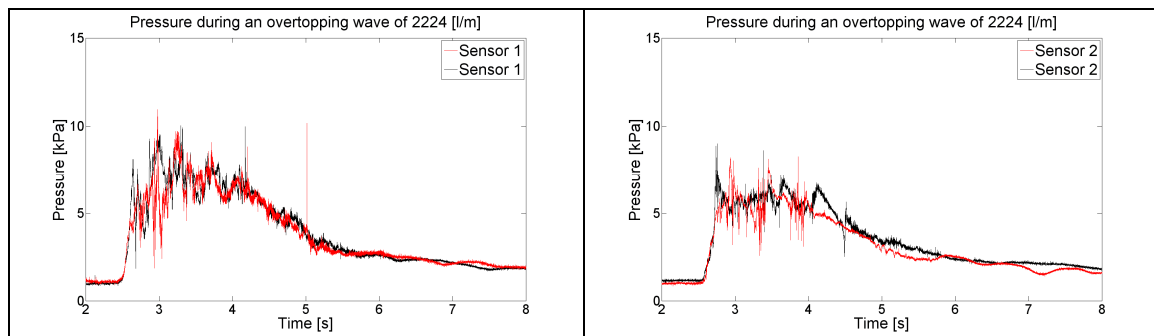
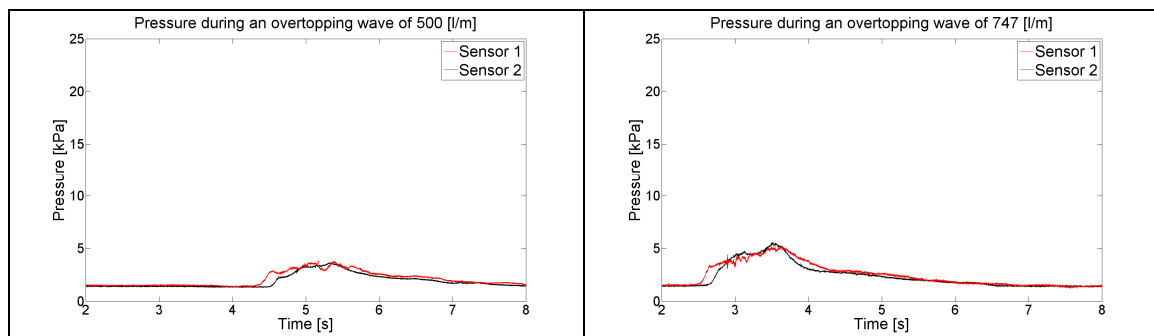


Figure 4-13: Calibration waves of (almost) the same volume (left and right)

In Figure 4-14 the pressure results are shown for waves from 500 [l/m] to 4500 [l/m] with steps of 500 [l/m]. The frequency used for the experiment is 5000 [s⁻¹], which is significantly smaller than the fluctuations in the measurement signal.



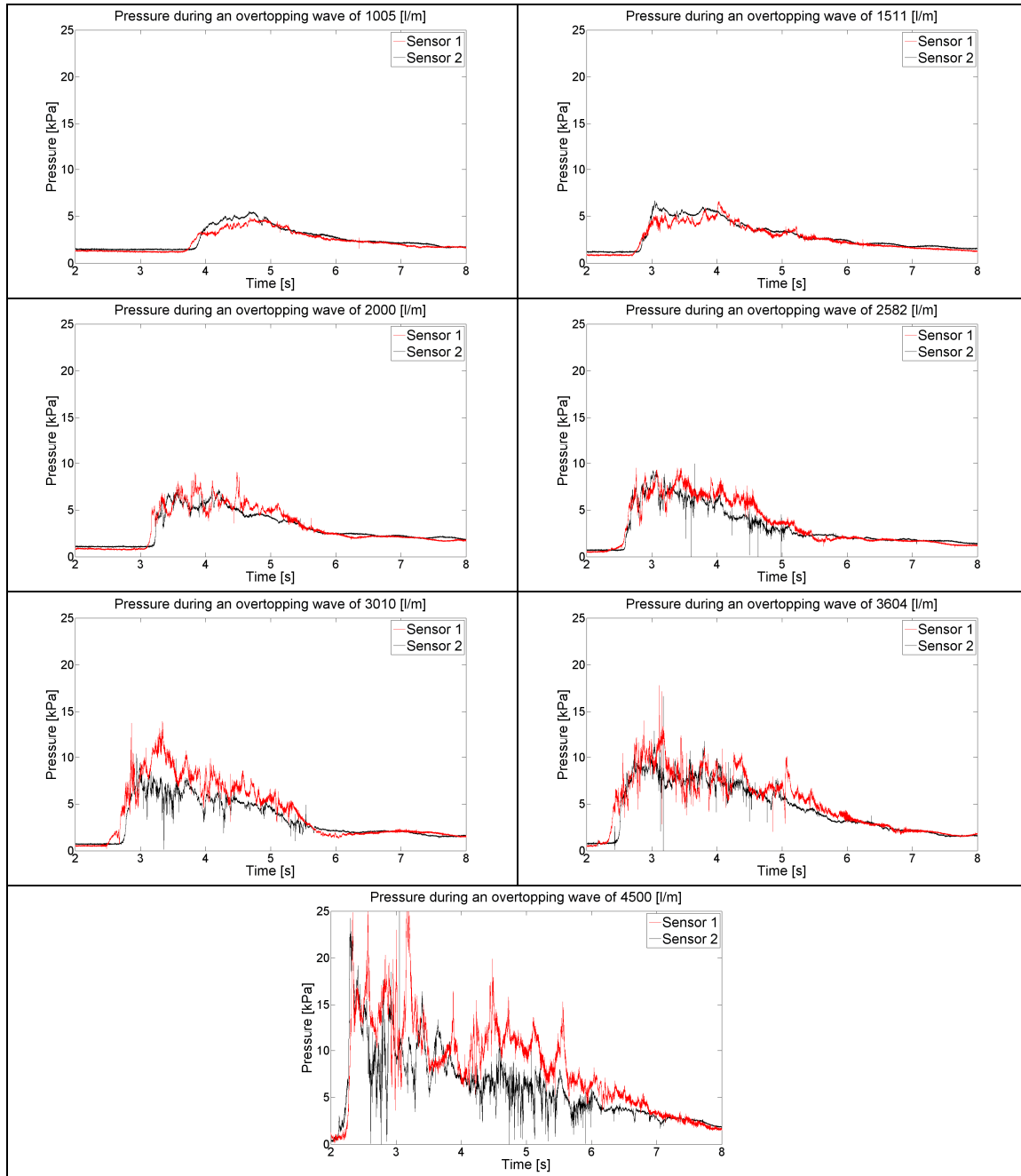


Figure 4-14: Pressure measurements in front of the object for waves from 500 [l/m] to 4500 [l/m]

Besides the measurements in front of the object, the pressure gauges are also installed on another dike slope without an object for a different experiment. These results are used to compare the pressure gradients that are measured on the slope and just in front of the object. The difference should contribute to the determination of the load parameter in the erosion model, see chapter 5 for the analysis.

This experiment used a dike slope with the same angle. The first pressure gauge is installed at 9.0 [m] from the crest and the second pressure gauge is installed at 13.65 [m] from the crest. Both devices are placed with their sensors at the surface. In Figure 4-15 the pressure results are shown for waves from 500 [l/m] to 5000 [l/m]. The analysis of these results with respect to the cumulative load model of paragraph 3.1 is described in paragraph 5.2.

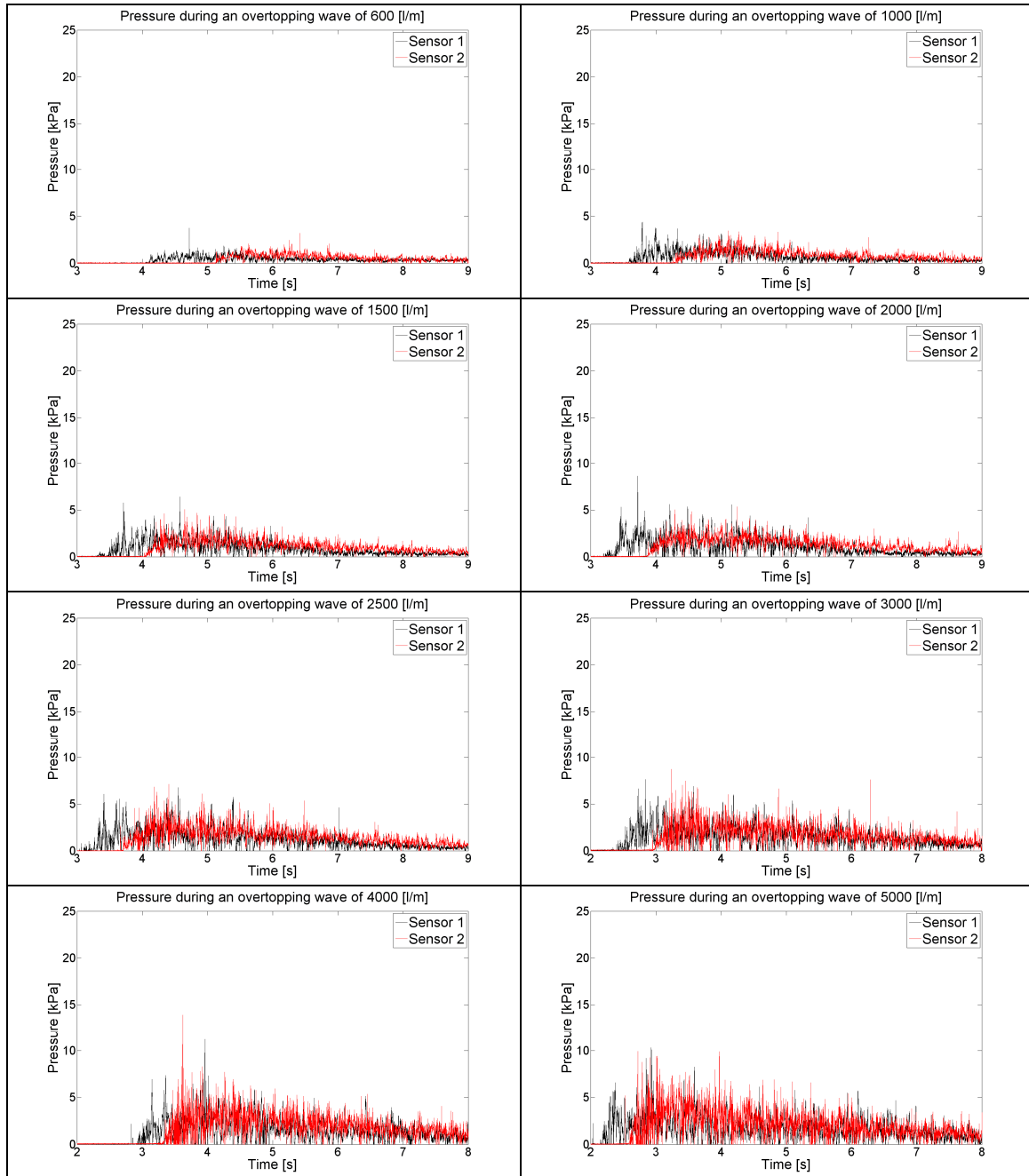


Figure 4-15: Pressure measurements on the slope for waves from 600 [l/m] to 5000 [l/m]

4.2.2 Grass tensile strength

In 2012 within the WTI 2017 framework a device to determine the tensile strength of a certain turf was developed. This turf-tensile apparatus is shown in Figure 4-16. The apparatus consists of a tripod to hold the hydraulic pump to pull out a grass turf. The turf is connected to the hydraulic pump by three or four pins that are penetrated through the grass turf at a depth of 3 [cm]. The force needed to pull out the grass turf is registered by an electronic sensor.



Figure 4-16: Turf-tensile apparatus

Experiment one

During the first experiment with the turf-tensile apparatus, the maximum tensile strength of the grass turf was measured while the turf was pulled out in one motion. The tests were performed in wet and dry conditions, with either two or four sides that were cut loose. The complete results for the first tests at three different locations are shown in appendix C. The visual review that was performed on the grass at the test locations resulted in the following:

- Millingen aan de Rijn: High and well developed vegetation.
- Nijmegen: High overgrown vegetation including herbs.
- Zwolle: 'Open' vegetation on a non-coherent and dry sand soil, but a well developed root system.

This test led to the average grass strengths that are shown in Table 4-5; including only the wet conditions, because of its relevance with overtopping. The fourth column with zero sides cut is an extrapolated value from the other measurements.

Table 4-5: Tensile grass strength test results

Location	Tensile force [kN] (4 sides cut)	Tensile force [kN] (2 sides cut)	Tensile force* [kN] (0 sides cut)
Millingen a.d. Rijn	0.43	0.90	1.37
Nijmegen	0.50**	0.43**	0.36**
Zwolle	0.30	0.49	0.68

* = extrapolated values
 ** = measurement error, see text below

These results are extrapolated to values for the tensile grass strength in $[\text{kN}/\text{m}^2]$, which is displayed in Table 4-6. This calculation is based on the turf aggregate dimensions that are given in appendix C.

Table 4-6: Tensile grass strength test results

Location	Tensile stress [kN/m ²] (4 sides cut)	Tensile stress [kN/m ²] (2 sides cut)	Tensile stress* [kN/m ²] (0 sides cut)
Millingen a.d. Rijn	19.1	26.1	33.1
Nijmegen	22.2**	19.1**	16.0**
Zwolle	13.3	21.8	30.2

* = extrapolated values
 ** = measurement error, see text below

From Table 4-6 can be concluded that the measurement in Nijmegen seems to be incorrect. The values decrease when less sides of the turf are cut loose, which is not possible. This was probably caused by one very strong root from another species than grass, which resulted in a high value for the four sides cut test.

According to Hoffmans (2012), the tensile strength of grass should be distributed as shown in Table 4-7 (final column). This table is based on the root characteristics given in VTV (2006).

Table 4-7: Root properties of Dutch dike grassland (Hoffmans, 2012)

Grass quality VTV-2006	No(0) per VTV area	$A_{\text{root}}(0)/A_1$ [-]	$A_1/\text{No}(0)$ [mm ²]	$A_1/\text{No}(0)^{1/2}$ [mm]	$\sigma_{\text{grass,c}}(0)$ [kN/m ²]
Very poor	< 18	< 0.0002	> 39	> 6	< 3.0
Poor	18 - 32	0.0002 - 0.0004	22 - 39	5 - 6	3.0 - 5.3
Average	32 - 45	0.0004 - 0.0005	16 - 22	4 - 5	5.3 - 7.5
good	> 45	> 0.0005	< 16	< 4	> 7.5

It appears the tensile strength found with the field measurement device is much larger than the tensile strength that is theoretically assumed in the VTV (2006). The same conclusion applies for the theoretically determined grass strength in Table 3-2. One explanation for these large values is the fact that the tests were performed in summer. Generally the strength of grass in summer is quite a bit higher, due to the fast growth of the roots. The theories are mostly based on winter conditions, because most storm events appear in winter. Another reason is the elasto-plastic behaviour of the roots (grass cover). During the test, the grass cover is pulled up until failure. This resulted in large forces, but also in large deformations. These deformations are never seen at a grass cover under overtopping conditions. At these overtopping events the applied load is based on pulse forces. It seems more plausible that a few roots will rip off during a wave load and some more at the next. This could be seen as if the grass cover suffers from fatigue.

A different result that appeared from the test was the tear depth of the turf. This was found to be mostly around 4.0 [cm]. If the theoretical grass normal strength (Equation 3-6) is plotted against the thickness of the turf (given a turf of 15x15 [cm]); the graph in Figure 4-17 appears. This also shows the lowest grass strength is reached at a depth of around 4.0 [cm].

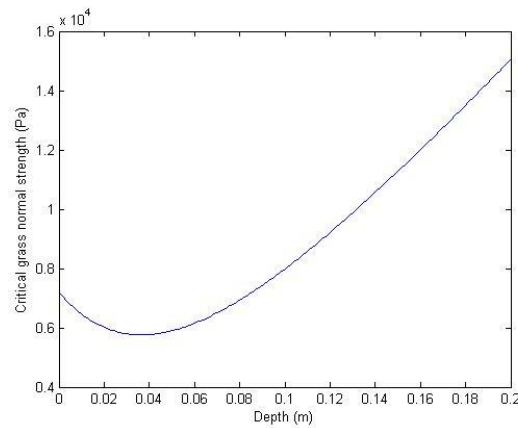


Figure 4-17: Critical grass normal strength for every turf thickness

Experiment two

The goal of the second test is to gain some insight into the assumptions about the fatigue of grass covers. To determine this, the strain of the top layer is taken as starting point and the tensile strength needed to reach this strain is determined. This is done for the strain distances of: 0.5, 1.0, 2.0, 3.0, 4.0, and 5.0 [cm] or until failure. Each strain value is tested five times to see whether the needed tensile strength reduces. A summary of the performed test can be found in Table 4-8, where tests one till four are conducted in the same way as the previous test and tests five till ten in the way described above.

Table 4-8: Overview of experimental results of the turf-tensile tests (Hoffmans, 2013)

Test	No. of sides cut	Slope orientation	Max. stress	Thickness turf (mean)	Thickness turf (max.)	Deformation (max)	Turf area	Comment
[-]	[-]	[-]	[kN/m ²]	[cm]	[cm]	[cm]	[cm ²]	
1	2-side	North	24.5	-	-			horz. berm
2	2-side	North	28.9	4	7.5	> 5	25 x 15	on slope (upper)
3	4-side	North	22.1	5	8	-	15 x 15	halfway on slope
4	2-side	North	42.6	8	12	6		halfway on slope
5	2-side	North	40.9	5	11	6		halfway on slope
6	4-side	North	23.8	4		5	15 x 15	halfway on slope
7	2-side	North	18.7	3		5	15 x 15	moss
8	2-side	North	16.3	4	4	6	15 x 15	moles
9	2-side	South	38.5	6.5	9.5	6	23 x 15	halfway on slope
10	4-side	South	25.5	6	7.5	5	15 x 15	halfway on slope

The graphical presentation of the results of test 5, 6 and 7 are graphically shown in Figure 4-18, Figure 4-19 and Figure 4-20. The complete results for all tests can be found in appendix C. During the tests, the pressure is measured, which can be rewritten towards a force by multiplying it by the area of the hydraulic pump ($7.66 \text{ [cm}^2\text{]}$). Furthermore, after each pressure measurement for a certain displacement, the grass turf was unloaded and manually returned to zero displacement. This was necessary, because the hydraulic pump would stay in its current position after releasing the pressure. However, it appeared that after some loading and unloading it took quite some pressure to return the grass turf to its original location. This could be seen by the tripod, which was getting pushed upward. Once this happened, the grass turf was not pushed further down and the deformation was assumed as plastic. This is also stated in appendix C.

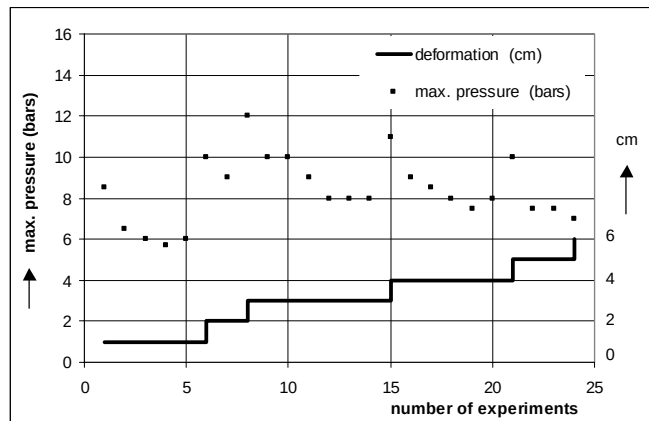


Figure 4-18: Results of turf-tensile test 5 (Hoffmans, 2013)

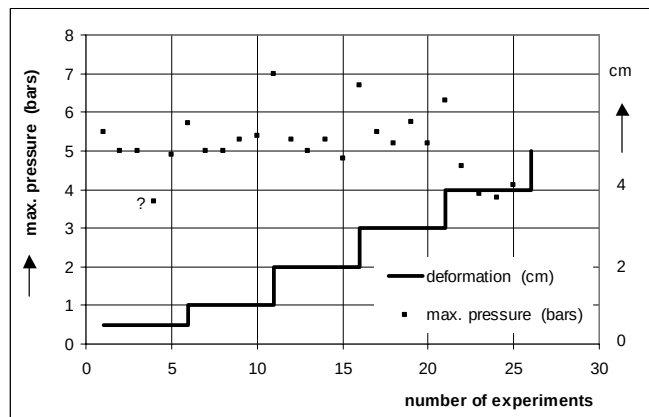


Figure 4-19: Results of turf-tensile test 6 (Hoffmans, 2013)

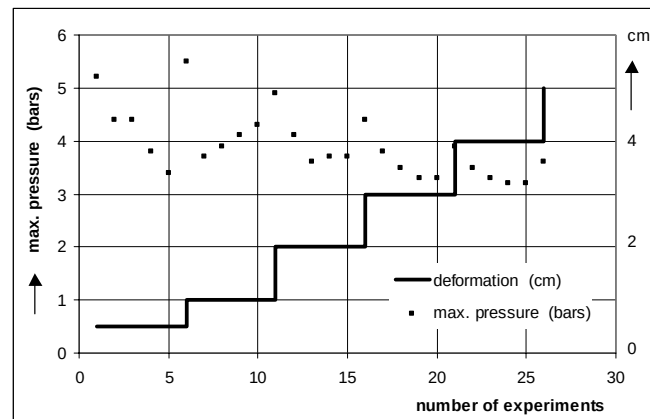


Figure 4-20: Results of turf-tensile test 7 (Hoffmans, 2013)

Each test shows that after the first loading cycle on the grass turf, the second time takes significantly less force. This is seen best in tests 5 and 7. In other words, after one load some weak roots broke or they were oriented in such a way that some had to resist the entire load. However, if the displacement is increased, the turf can take another large load because of redistribution of the forces to other roots. An exception seems the 1.0 [cm] displacement of test 7. Here the force actually increases, which is probably a measurement failure due to the application of a slightly larger displacement. Nevertheless, the peak values and the subsequent tail values are still higher than any theoretical value found in paragraph 3.3.2. It seems though, that the effect of fatigue is measured and can be seen in the results, only with these quite high values.

Still the differences between two and four sides cut and between turf and moss can be determined. If you cut two sides more, the reduction of the peak value is 43% on the north side and 33% on the south side. If the reduction is compared for each displacement, the results of Table 4-9 appear. It has to be noted that during test five the displacement of 0.5 [cm] has not been tested. This probably initiated the large strength of the grass cover that is measured during the test with a displacement of 1.0 [cm].

Table 4-9: Comparison between two and four sides cut during the test

Experiment 5 and 6			Experiment 9 and 10		
Displacement [cm]	Reduction peak [%]	Reduction tail [%]	Displacement [cm]	Reduction peak [%]	Reduction tail [%]
0.5	-	-	0.5	10	10
1.0	33	15	1.0	16	20
2.0	30	-	2.0	23	31
3.0	44	34	3.0	43	53
4.0	43	50	4.0	57	63
5.0	-	-	5.0	-	-

This clearly shows that the reduction increases along with the displacements and that the reduction of the tail is faster than the reduction of the peak. It can be concluded that the influence of the frictional grass strength on the side walls has a larger influence when the displacement of the grass turf is larger. Another conclusion is that the influence of the side

planes is larger for the tail than the peak loads. This is actually also shown by the pattern of the reduction in Figure 4-18 and Figure 4-19. When all sides are cut, the tail immediately reduces to a constant lower level. In the other case, the pressure reduces gradually over the following tests. A third way this effect is shown, is the displacement that corresponds to the peak pressure. In case of four cut side planes this peak is reached at a smaller displacement than for only two cut side planes. This means in short that (some) roots on the bottom of the grass turf break during the first impact; the roots in the side planes are mostly pulled to a different location, but still remain (a part of) their strength.

The same comparison as above is done for tests on an area with moss and a mole corridor, which is shown in Table 4-10.

Table 4-10: Comparison between turf and moss/mole corridor

Experiment 9 and 7			Experiment 9 and 8		
Displacement [cm]	Reduction peak [%]	Reduction tail [%]	Displacement [cm]	Reduction peak [%]	Reduction tail [%]
0.5	10	20	0.5	34	34
1.0	29	37	1.0	45	41
2.0	50	52	2.0	51	50
3.0	60	61	3.0	59	58
4.0	65	64	4.0	-	-
5.0	-	-	5.0	-	-

The reduction of the applied force for moss appears to be really fast for the smaller deformations. This means that the roots for moss on the bottom and side planes break and/or are pulled out earlier. An expected result, since the roots of moss do not reach as deep as the roots of grass.

Interesting to see is the initial force that is needed for the mole corridor, which is significantly lower than for moss or grass turfs with four sides cut. Most likely this is caused by the low presence of vegetation. Since the grass turf with four sides cut, two sides cut and the moss give quite the same values, it is concluded that for the first small displacement the coherence of the soil absorbs a large chunk of the applied force.

If this test is reflected with respect to overtopping situations, the soil would be saturated. This could reduce the soil coherence, so the force goes directly to the roots and the equal result for the first displacement would not have been measured. Another remark of this experiment with respect to overtopping situations is the loading of the grass turf. Where during wave loading the pressure gradients that load the turf occur in tens of milliseconds, this occurs during the experiment in several seconds. This could very likely be the most important reason that this high strength is measured, because the longer loading time allows the roots to get (partly) pulled out and/or relocated instead of breaking them off. Further investigation is therefore needed to determine the actual critical soil strength in the field.

5 Analysis

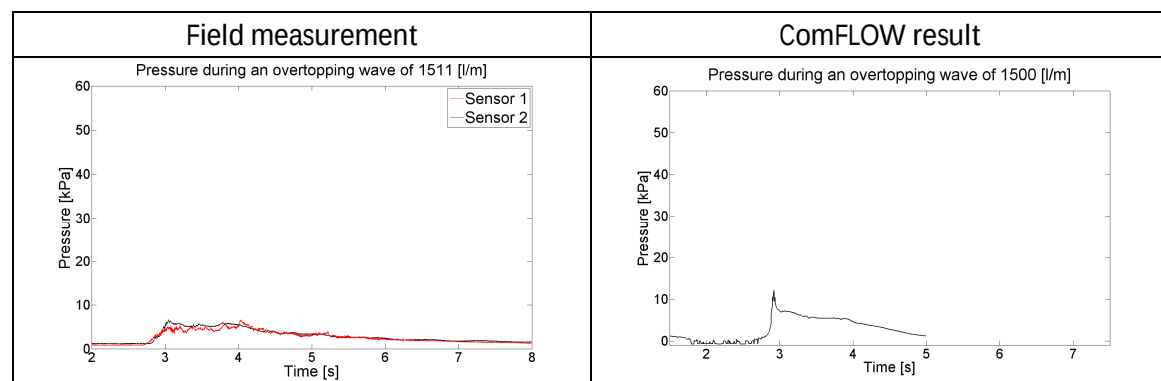
In chapter 3 the erosion models for zone A and B are defined and in chapter 4 these definitions are subjected to experiments and numerical analyses. Now it is possible to determine whether the assumptions and defined models are correct or plausible. First the numerical model is validated with respect to the experimental results in paragraph 5.1. This is followed by an analysis of the pressure erosion models of zone A in paragraph 5.2 and the flow erosion model of zone B in paragraph 5.3. The methodology of how these analyses are related to the previous chapters is summarized in Table 5-1. Finally the models are validated with respect to an earlier field experiment in paragraph 5.4.

Table 5-1: Methodology of the analyses performed in this chapter

	Zone A		Zone B
Model	Cum. pressure load model	Impact load model	Cum. velocity load model
Load	Theory (§3.2.1) + Field experiment: Overtopping (§4.2.1) ↓ Load parameter $\alpha_{l,A}$ (§5.2)	Theory (§3.2.1) + Field experiment: Overtopping (§4.2.1) ↓ Load analysis (§5.2)	Theory (§3.3.1) + Numerical model: ComFLOW (§4.1) ↓ Load parameter $\alpha_{l,B}$ (§5.3)
Strength	Theory (§3.2.2) + Field experiment: Turf tensile test (§4.2.2) ↓ Strength parameter α_s (§5.2)	Theory (§3.2.2) ↓ Strength analysis (§5.2)	Theory (§3.3.2) + Field experiment: Turf tensile test (§4.2.2) ↓ Strength parameter α_s (§5.3)

5.1 Comparison numerical and experimental results

To clarify the comparison of the numerical ComFLOW results and the experimental results, they are shown together in Figure 5-1. The results of the pressure measurements in front of the object for wave volumes of 1500, 2500, 3500 and 4500 [l/m] are given.



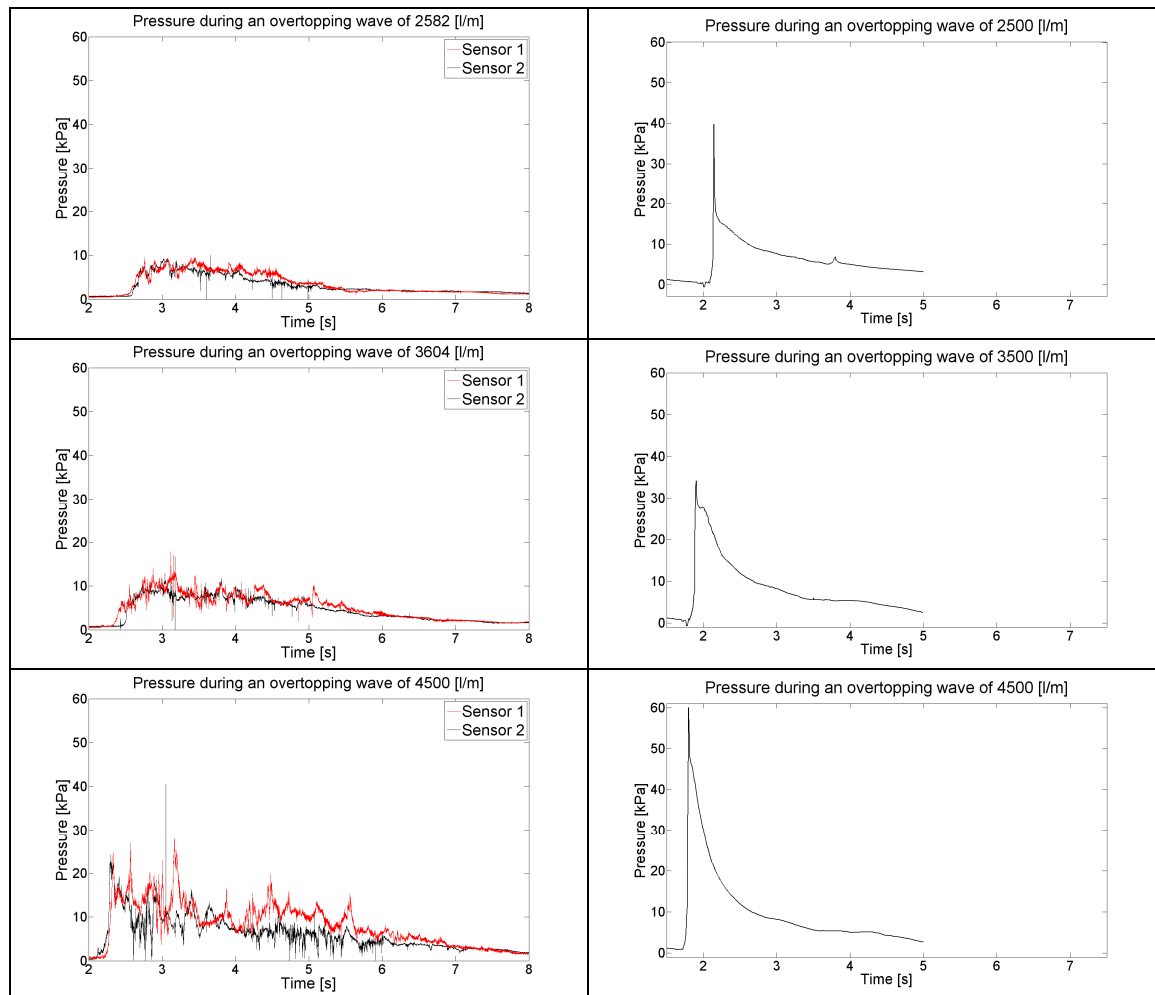


Figure 5-1: Comparison pressure in front of object for field measurements and ComFLOW results

It appears the peak pressure found in the field experiments is significantly lower than the numerical results. Varying from a factor 1.2 for waves of 1500 [l/m] till a factor 2 for waves of 4500 [l/m]. However, the tail of the wave shows a quite similar shape.

A first reason for this could still be the water that was constantly present at the toe of the dike during the experiment and most likely slowed the waves. This could have caused a velocity difference between the model and the experiment. During one of the later experiments, the velocity was measured on the slope. These values are at the moment of the writing of this report not available, therefore this could be validated at a later stage. To get a first insight, video footage is used to estimate the velocity in front of the object. This value is an average over the last 3 [m] in front of the object. A value of 3.5 [m/s] is found for a wave of approximately 1500 [l/m], which is the same as in ComFLOW. Nevertheless, because it is an average and the velocity decreases over the horizontal part; it is expected the actual impact velocity is slightly lower. Hence, the velocities and slope friction in the ComFLOW model should be further researched with respect to field values in future research to improve the contribution of the model.

A second reason for the high ComFLOW peak could be the grid size of the model. The velocity is taken as an average over an area of 10x10x10 [cm], which might have increased the actual velocity that is present several centimetres in front of the object. During the measurements at

the field experiment the pressure is only based on the velocity at about 1 [cm] in front of the object. This is most likely lower than the average velocity over the 10 [cm] in front of the object.

Furthermore, the sonic pressure peak is still present in the ComFLOW model after the water in front of the object is introduced. This is not seen in the field experiments, thus it seems the water in front of the object completely absorbs the sonic wave impact.

The field measurements do show a lot of peaks over the entire wave length, this can be seen as the turbulence in the water. The turbulence in the water cannot be seen in the numerical model results, because it is not implemented in the program. The amount of turbulence increases fast from low to high wave volumes. This way the turbulence may have more impact on the erosion than initially expected. It can also be seen that the grey signal (closest to the 'open' corner) shows more turbulence than the black signal (closest to the 'closed' corner). This is probably caused by the direction of the wave which was slightly towards the dead end corner in the slope. Subsequently, the wave turned towards a standing wave in the closed corner and only sensor 1 got the complete impact. Off course this effect reduced when the waves got larger.

5.2 Pressure erosion model

In paragraph 3.2, two erosion models are defined for zone A. Both are based on the pressures that are exerted on the grass cover in front of the object by an overtopping wave. The difference was mainly defined as the presence of a crack between the soil and the structure, which is necessary for the impact load model. The cumulative load model, however, is not bounded to the presence of a crack.

Cumulative pressure load model

The defined cumulative load model for erosion in zone A is shown again in Equation 5-1.

$$\begin{array}{ll}
 \text{Initial damage} & \sum_{i=1}^{i=N_t} (\alpha_{I,A} \cdot p_{u,i} - \alpha_s \cdot p_c) \approx 250 \quad [\text{kPa}] \\
 \text{Damage at various locations} & 250 \quad [\text{kPa}] < \sum_{i=1}^{i=N_t} (\alpha_{I,A} \cdot p_{u,i} - \alpha_s \cdot p_c) < 750 \quad [\text{kPa}] \\
 \text{Failure} & \sum_{i=1}^{i=N_t} (\alpha_{I,A} \cdot p_{u,i} - \alpha_s \cdot p_c) > 1750 \quad [\text{kPa}]
 \end{array} \quad 5-1$$

Where:

$$\begin{array}{ll}
 \alpha_{I,A} & = \text{load parameter} \quad [-] \\
 p_{u,i} & = \text{uplift pressure of wave } i \quad [\text{kPa}] \\
 \alpha_s & = \text{strength parameter} \quad [-] \\
 p_c & = \text{critical pressure} \quad [\text{kPa}]
 \end{array}$$

As is stated before, during the field experiments pressure gauges were installed on the slope and in front of the object. The results showed both the hydrostatic and the dynamic pressure that the wave causes. If the dynamic pressure moves from an over to an under pressure, an upward directed pressure gradient will appear. The quantification of this pressure gradient is defined as

the amplitude of the dynamic pressure. In Table 5-2 the upward directed pressure gradients are determined from the experiment for the sensor on the slope and in front of the object. To determine these values, the four largest gradients are taken and an average of these four is shown in Table 5-2.

Table 5-2: Pressure gradients on slope and in front of object

Wave volume [l/m]	Pressure gradient slope [kPa]	Pressure gradient object [kPa]	Ratio α_L [-]
±500	0.8	0.8	1.00
1000	1.3	0.9	0.69
1500	1.8	1.2	0.67
2000	2.0	2.4	1.20
2500	2.5	3.3	1.32
3000	3.0	4.7	1.57
±4000	3.6	8.5	2.36

Table 5-2 also shows the ratio between the two measurements. It seems the load actually increases slower near the object compared to the slope for lower wave volumes. This is most likely the result of the water layer that was present in front of the object, which significantly absorbed the dynamic pressure. In Figure 5-2 the results are plotted against the corresponding wave volumes and a trendline for the load parameter is plotted from these results. For the plot of the trendline, the results for the wave volumes of 500, 1000 and 1500 [l/m] are ignored. It is assumed unlikely that an object will decrease the load and therefore a boundary condition is added. This leads to Equation 5-2 for the load parameter. Remarks on this parameter are discussed below.

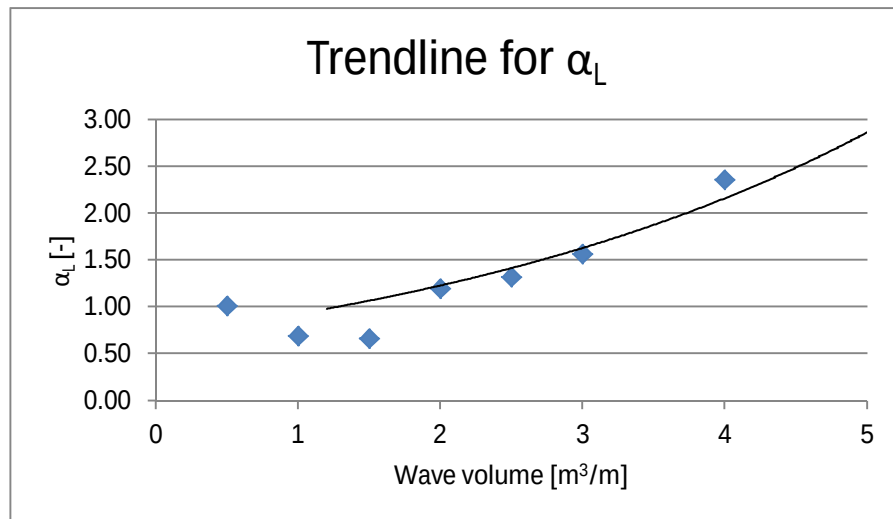


Figure 5-2: Trendline plot for the load parameter

$$\alpha_{l,A} = 0.7 \cdot e^{0.28 \cdot V}$$

5-2

With:

$$\alpha_{l,A} = 1 \quad \text{for} \quad V \leq 1.2$$

Where:

$$\begin{aligned}\alpha_{I,A} &= \text{load parameter in zone A} \quad [-] \\ V &= \text{wave volume} \quad [\text{m}^3/\text{m}]\end{aligned}$$

Remarks on the determined equation are first of all the assumption that has been made with respect to the velocity at the different measurement locations. It is assumed that the wave velocity at the measurement location on the slope has reached an equilibrium velocity. Therefore this equation only holds for slopes with an angle and length where the water reaches an equilibrium velocity. This holds for slopes of 1:3 and gentler, according to Steendam et al. (2012). Furthermore, the location of the object to be able to apply this formula is also restricted to several metres after the toe of the dike. If the object is on the bottom side of the slope (equilibrium velocity), there might be less water in front of the object to absorb dynamic pressures. The same holds for the top side of the slope, given that also the velocities will be lower.

The definition of the critical pressure was defined in paragraph 3.2.2 and is shown again in Equation 5-3.

$$p_c = (1-n) \cdot \left[\frac{n_s \cdot d}{w} \cdot \int_0^d \sigma_{grass,c}(z) dz + \sigma_{grass,c}(z) \right] \quad 5-3$$

Where:

$$\begin{aligned}n &= \text{porosity} \quad [-] \\ \int_0^d \sigma_{grass,c}(z) dz &= \text{critical grass shear stress} \quad [\text{Pa}] \\ \sigma_{grass,c}(z) &= \text{critical grass normal stress at depth } z \quad [\text{Pa}] \\ n_s &= \text{number of collaborating side walls} \quad [-] \\ d &= \text{depth of the grass turf} \quad [\text{m}] \\ w &= \text{width of the grass turf} \quad [\text{m}] \\ z &= \text{depth} \quad [\text{m}]\end{aligned}$$

With:

$$\sigma_{grass,c}(z) = \sigma_{root,c} \cdot RAR_0 \cdot e^{-\beta \cdot z} \quad 5-4$$

Where:

$$\begin{aligned}\sigma_{root,c} &= \text{critical root normal stress} \quad [\text{Pa}] \\ RAR_0 &= \text{root area ratio at surface} \quad [-] \\ \beta &= 22.32 \text{ (Valk, 2009)} \quad [-] \\ z &= \text{depth} \quad [\text{m}]\end{aligned}$$

It is assumed that next to a structure only three sides of a grass turf are contributing to the strength. Furthermore, the root area ratio is expected to be lower around objects due to the lack

of sunlight. The parameters are summed in Table 5-3 and the results for the critical pressure with respect to the grass quality are shown in Table 5-4.

Table 5-3: Applied values for soil next to structure

Variable	Value	Variable	Value
d	0.05 [m]	w	0.15 [m]
n	0.4 [-]	n_s	3 [-]
ρ_s	2000 [kg/m ³]	ϕ	35 [°]
ρ_w	1000 [kg/m ³]	$\sigma_{\text{root,c}}$	$15 \cdot 10^6$ [Pa]
g	9.81 [m/s ²]	β	22.32 [-]

Table 5-4: Critical pressure for several grass qualities

Grass quality VTV-2006	RAR_0 [-]	Critical pressure next to object [kN/m ²]	Critical pressure [kN/m ²]
Very poor	< 0.0002	< 2.0	< 2.5
Poor	0.0002 - 0.0004	2.0 - 3.7	2.5 - 4.5
Average	0.0004 - 0.0005	3.7 - 4.6	4.5 - 5.5
good	> 0.0005	> 4.6	> 5.5

According to Table 5-4, the critical pressure would reduce with about 20 [%] for any grass quality if it is located next to a structure. This reduction should also be seen in the field experiments of paragraph 4.2.2. Therefore the relevant values of the experiment are shown again in Table 5-5.

Table 5-5: Comparison between two and four sides cut during the test

Experiment 5 and 6			Experiment 9 and 10		
Displacement [cm]	Reduction ratio peak [-]	Reduction ratio tail [-]	Displacement [cm]	Reduction ratio peak [-]	Reduction ratio tail [-]
0.5	-	-	0.5	0.90	0.90
1.0	0.67	0.85	1.0	0.84	0.80
2.0	0.70	-	2.0	0.77	0.69
3.0	0.56	0.66	3.0	0.57	0.47
4.0	0.57	0.50	4.0	0.43	0.27
5.0	-	-	5.0	-	-

It appears from Table 5-4 and Table 5-5 that the field experiments showed an increase in the reduction ratio (between two and four contributing sides) along with an increase in the displacement. The theoretical value, however, shows a constant value of 0.8 [-] for one side cut. This means two sides cut would give a 0.6 [-] reduction ratio, which corresponds to displacements of the soil of around 3-4 [cm]. So it can be concluded, the influence of the side planes is largest for high displacement. In the first 0.5-1.0 [cm], the influence is significantly smaller. Therefore it would be safer to determine the reduction factor from the smaller displacement regions, since a higher factor would decrease the initial strength too much. This means a ratio of 0.90 [%] could be used for two sides not contributing and a reduction ratio of $\alpha_s = 0.95$ for one side that is not contributing in case of an object. Nevertheless, this is based on

two tests with a high uncertainty on especially the low displacements. More tests are needed to verify this value.

Impact load model

The defined model for erosion in zone A, given that a crack is present between soil and structure, is shown again in Equation 5-5 and Figure 5-3.

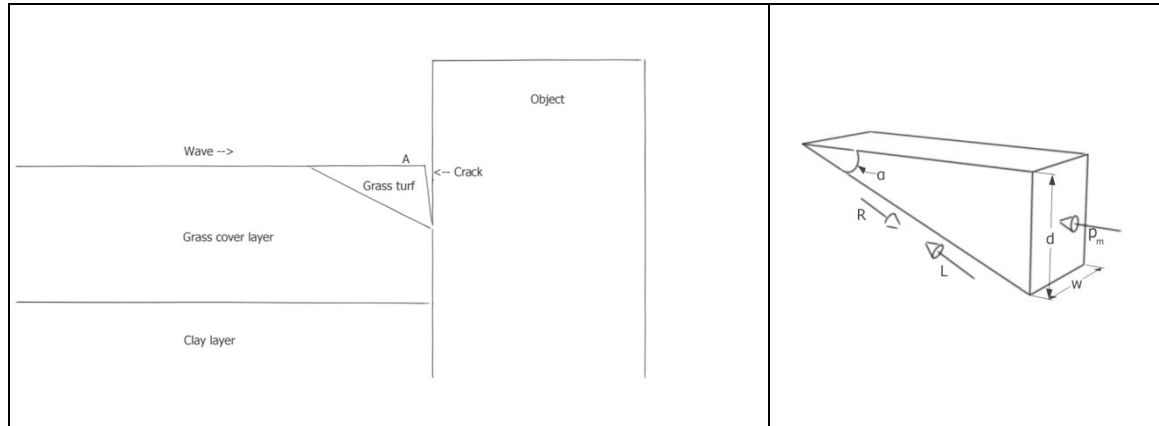


Figure 5-3: Left: Schematic side view of erosion zone A, Right: Schematic grass turf

$$L = d \cdot w \cdot p_m \cdot \cos(\alpha)$$

$$R = \frac{d^2 \cdot c}{\tan(\alpha)} + \frac{d \cdot w \cdot c}{\sin(\alpha)} + \frac{0.5 \cdot d^2 \cdot w}{\tan(\alpha)} \cdot \rho \cdot g \cdot \sin(\alpha) \quad 5-5$$

Where:

L	= loading shear force	[N]	d	= depth of eroding element	[m]
R	= resisting shear force	[N]	α	= angle of eroding element	[°]
p_m	= maximum impact pressure	[Pa]	c	= frictional strength	[Pa]
w	= width of eroding element	[m]	ρ	= soil density	[kg/m ³]

It is assumed erosion occurs when the load exceeds the resistance ($L > R$). Before anything can be said about the wave volume that would initiate erosion, assumptions have to be made for the other variables. The value for the depth of the turf is assumed at 0.04 [m], which is based on multiple observed eroding grass turfs during field experiments in the past. The width of the element is initially assumed at 0.15 [m], which seems quite large. The reason is that if this type of erosion would occur over the entire width of a large structure, the influence of the side planes would be marginal. If the structure would be small (pole), the influence would be larger. The exact influence of the width is not known yet, but is studied further on in this paragraph. Furthermore, the assumptions of the frictional strength and the other variables are shown below, where the root strength and RAR_0 are taken from (VTV, 2006) for poor to average grass. This grass quality seems legitimate since the close to structures the grass is not very good due to a lack of sunlight.

$$c = \int_0^d \sigma_{grass,c}(z) dz = \int_0^d \sigma_{root,c} \cdot RAR_0 \cdot e^{-\beta \cdot z} dz$$

Where:

$\sigma_{root,c}$	= critical root normal stress	= $15 \cdot 10^6$	[Pa]
RAR_0	= root area ratio at surface	= 0.0004	[-]
β	= coefficient	= 22.32 (Valk, 2009)	[-]
d	= depth of eroding element	= 0.04	[m]

Variables:

w	= width of eroding element	= 0.15	[m]
α	= angle of eroding element	= 45	[°]
c	= frictional strength	= $4 \cdot 10^3$	[Pa]
ρ	= soil density	= 2000	[kg/m ³]

Equation 5-5 is solved for the maximum impact pressure by entering the values that are given above. To determine at which width the side planes could be neglected, the pressure is plotted against the width of the eroding element in Figure 5-4. The same principle is applied for the angle of the eroding element in Figure 5-5.

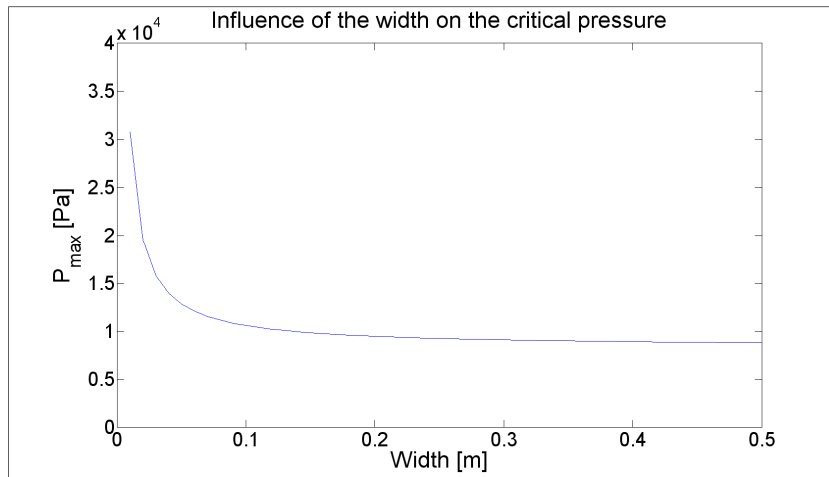


Figure 5-4: The influence of the width on the critical pressure

Figure 5-4 shows that only for objects smaller than roughly 0.04 [m], it has a significant influence on the maximum pressure. This also means that erosion modelled by this impact pressure model would most likely not occur in front of for instance poles that have a diameter smaller than 4 [cm].

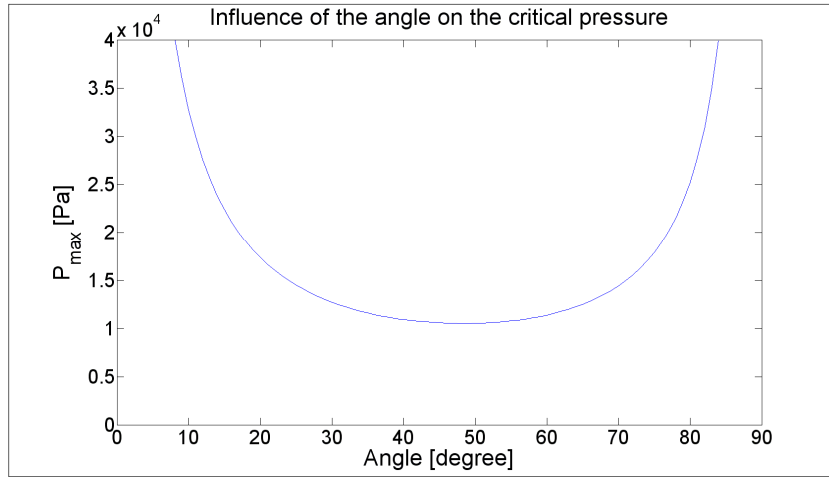


Figure 5-5: The influence of the angle (see Figure 5-3) on the critical pressure

Figure 5-5 shows that the differences in the middle area are quite small. In other words, erosion could occur in the form of grass turfs with an inclined plane between 30 and 70 [°]. Within this range of more or less equal results for the pressure, the local soil structure determines which plane is actually taken by the grass turf to erode. This would change if the erosion/crack depth differs a lot from the applied value of 0.04 [m]. Finally the applied root area ratio is taken as 0.0004 [-], which is a value for a poor to average grass quality. This is assumed, because the roots around structures develop poorly due to the lower presence of sunlight. Given these values the maximum pressure that is allowed before erosion would occur is $9.8 \cdot 10^3$ [kPa].

If this is compared to the results of the field experiments in paragraph 4.2, the following can be concluded: Waves larger than roughly 3000 [l/m] would initiate certain erosion, waves of 2000 - 3000 [l/m] have a significant chance to initiate erosion and waves smaller than 2000 [l/m] would not initiate erosion. Nevertheless, this remains heavily dependent on the local grass quality.

5.3 Flow erosion model

The defined model for erosion in zone B is shown again in Equation 5-6.

$$\begin{aligned}
 \text{Initial damage} & \sum_{i=1}^{i=N_t} (\alpha_{l,B} \cdot U_{m,i}^2 - \alpha_s \cdot U_c^2) \approx 500 \quad [\text{m}^2 / \text{s}^2] \\
 \text{Damage at various locations} & 500 \quad [\text{m}^2 / \text{s}^2] < \sum_{i=1}^{i=N_t} (\alpha_{l,B} \cdot U_{m,i}^2 - \alpha_s \cdot U_c^2) < 1500 \quad [\text{m}^2 / \text{s}^2] \quad 5-6 \\
 \text{Failure} & \sum_{i=1}^{i=N_t} (\alpha_{l,B} \cdot U_{m,i}^2 - \alpha_s \cdot U_c^2) > 3500 \quad [\text{m}^2 / \text{s}^2]
 \end{aligned}$$

Where:

$$\begin{aligned}
 \alpha_{l,B} &= \text{load parameter in zone B} & [-] \\
 U_{m,i} &= \text{maximum velocity of wave i} & [\text{m/s}] \\
 \alpha_s &= \text{strength parameter} & [-] \\
 U_c &= \text{critical velocity} & [\text{m/s}]
 \end{aligned}$$

The load (velocity) next to the object is investigated with the numerical ComFLOW model in paragraph 4.1. The results show hardly any difference in the local flow velocity at the sides of the object; right after the impact. It is even possible to state that the velocity reduces along and after the object. Nevertheless, these local decelerations found in the ComFLOW model are quite small. Therefore the assumption that a velocity difference influences the load on the grass cover seems not plausible. Due to the relatively small load increase, it seems that another mechanism should be contributing more to the initiation of the erosion near the corner of the object. Another reason could be that erosion in zone B is initiated due to earlier erosion in zone A. The erosion crack or area in zone A can cause flow concentration that increases the load on the cover in zone B.

The strength next to the object is similar to the strength in front of the object. Therefore the strength and its corresponding strength parameter can be taken from paragraph 5.2. However, since the strength is defined in the form of a critical pressure in paragraph 5.2 and as a critical velocity in this paragraph, the parameter should be paraphrased. This is shown in Equation 5-7, wherefrom it appears the strength parameter can be applied in the same way as for the pressure model.

$$U_{c*} = \frac{\alpha_{grass,U}}{r_0} \cdot \sqrt{\frac{\psi_c \cdot \alpha_s \cdot p_c}{\rho}} \quad 5-7$$

$$U_{c*}^2 = \frac{\alpha_{grass,U}^2}{r_0^2} \cdot \frac{\psi_c \cdot \alpha_s \cdot p_c}{\rho} = \alpha_s \cdot \frac{\alpha_{grass,U}^2}{r_0^2} \cdot \frac{\psi_c \cdot p_c}{\rho} = \alpha_s \cdot U_c^2$$

Where:

U_c	= critical depth averaged velocity	[m/s]
U_{c*}	= critical depth averaged velocity next to object	[m/s]
α_s	= strength parameter	[-]
$\alpha_{grass,U}$	= $\alpha_0 \cdot \sqrt{1 + 3 \cdot \alpha_{grass}} = 2.0$	[-]
r_0	= turbulence intensity	[-]
ψ_c	= critical Shields parameter ≈ 0.03	[-]
p_c	= critical pressure	[Pa]
ρ	= liquid density	[kg/m ³]

5.4 Validation of the erosion models

Due to the concrete used in front of the object in the latest field experiment; the results cannot be validated with this experiment. Therefore an earlier field experiment at the Vechtdijk is used to do a first validation of the above models. The test consisted of an overtopping test with a tree near the toe of the dike. According to the factual report of the experiment (Bakker et al., 2010), the test used average discharges from 1.0 [l/s/m] up to 30 [l/s/m]. The initial erosion occurred in the first two hours of the 10 [l/s/m] test and significant erosion was found after the completion of this test (six hours). After five hours of testing with 30 [l/s/m] the washout of sand went so fast they decided to cover the erosion hole, which can be seen as failure of the cover layer. The

different erosion states are shown in Figure 5-6 and should correspond with the failure states described in the original model of Van der Meer as can be seen in paragraph 2.4.6.

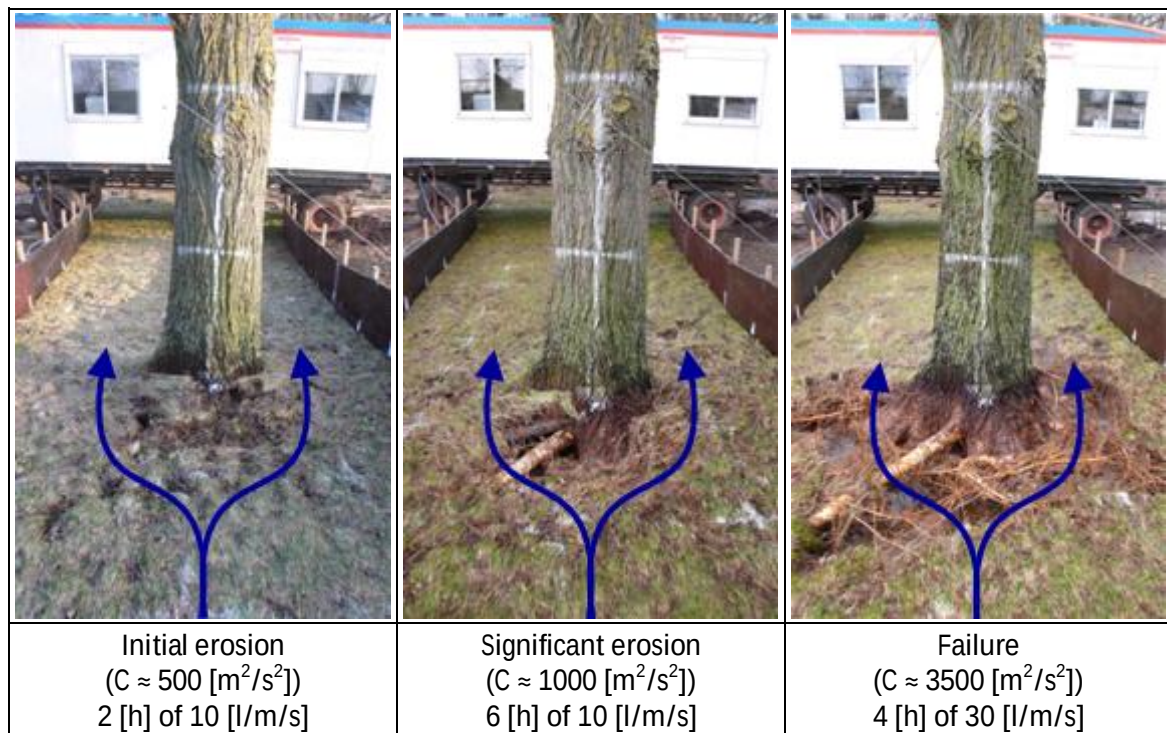


Figure 5-6: Different erosion states during the Vechtdijk experiment

To validate the cumulative pressure load model for zone A, the wave distribution of the experiment is needed, including the wave velocities just in front of the tree. These are deduced from the measurements and shown in Table 5-6. This table also shows the load term and strength term used in the model. It can be seen these terms are given as velocities and not as pressures such as described in paragraph 5.2, but since the pressures are not measured; they cannot be used. For now it is assumed the load factor can also be used for the impact velocities. However, the validity of this relation between the dynamic pressures and the impact velocity should be further investigated. Furthermore, the strength term is displayed for a critical velocity of 4 [m/s], but the calculations below are performed for several critical velocities.

Table 5-6: Characteristics Vechtdijk experiment and applied model input

1 l/s/m						
No. of wave [-]	Vol. [l/m]	Wave velocity [m/s]	α_L [-]	$\alpha_L(N(U_{m,i})^2)$ [m ² /s ²]	α_s [-]	$\alpha_s(N(U_c)^2)$ [m ² /s ²]
68	50	1.8	1.00	222	0.95	1034
31	175	2.8	1.00	237	0.95	471
18	375	3.6	1.00	231	0.95	274
8	750	4.5	1.00	164	0.95	122
1	1500	5.7	1.07	35	0.95	15

10 l/s/m						
No. of wave [-]	Vol. [l/m]	Wave velocity [m/s]	α_L [-]	$\alpha_L(N(U_{m,i})^2)$ [m ² /s ²]	α_s [-]	$\alpha_s(N(U_c)^2)$ [m ² /s ²]
261	50	1.8	1.00	851	0.95	3967
212	175	2.8	1.00	1620	0.95	3222
150	375	3.6	1.00	1925	0.95	2280
92	750	4.5	1.00	1891	0.95	1398
31	1500	5.7	1.07	1088	0.95	471
3	2500	6.8	1.41	197	0.95	46
30 l/s/m						
No. of wave [-]	Vol. [l/m]	Wave velocity [m/s]	α_L [-]	$\alpha_L(N(U_{m,i})^2)$ [m ² /s ²]	α_s [-]	$\alpha_s(N(U_c)^2)$ [m ² /s ²]
537	175	2.8	1.00	4104	0.95	8162
318	375	3.6	1.00	4080	0.95	4834
258	750	4.5	1.00	5304	0.95	3922
129	1500	5.7	1.07	4527	0.95	1961
27	2500	6.8	1.41	1774	0.95	410
6	3500	7.7	1.87	656	0.95	91

If the above load and strength terms are used, in addition of the critical velocity (Hoffmans, 2012) and failure criteria described earlier in this chapter, the times needed to reach a certain erosion state in Table 5-7 are found.

Table 5-7: Prediction for the time needed to reach a certain erosion state in zone A

Grass quality [m/s]		Damage criteria [m ² /s ²]		
		C ≈ 500	C ≈ 1000	C ≈ 3500
Very poor	$U_c = 1.5$	4 [h] of 1 [l/m/s]	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
			0.5 [h] of 10 [l/m/s]	3 [h] of 10 [l/m/s]
Poor	$U_c = 3.0$	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
Average	$U_c = 4.0$	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
Good	$U_c = 5.5$	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +

The times found for an average grass quality correspond quite well to the times found in the experiment as is shown in Figure 5-6. However, the grass quality around the tree is not tested and it is thus unknown if this matches. Also the load parameter was determined in paragraph 5.2 for a square object, while the tree had a round shape. The difference between both shapes, if there is one, should still be investigated.

To validate the impact load model, the same experiment is used. A tree and especially its large roots are assumed to have a negative influence on the quality of the grass around it. Therefore an average grass quality is used, which has a root area ratio of 0.0004 [-]. This would result in a maximum allowed pressure of $9.8 \cdot 10^3$ [kPa]. This value is approached by waves of 2500 [l/m] and exceeded by waves of 3000 [l/m], according to the field measurements (Figure 4-14). If this is compared to the flow chart of Table 5-6; it can be concluded that severe erosion might initiate during the 10 [l/m/s] test and will definitely initiate during the 30 [l/m/s] test. This model result corresponds with the experimental result; however, it is a very indicative result. Further investigation of the impact velocities that occur in front of the tree and the object should be performed to have a better insight into the results.

To validate the flow erosion model, the experiment at the Vechtdijk can also be used. However this time the load parameter is equal to one for each wave volume. This results in the times to reach a certain erosion state for several grass qualities as is shown in Table 5-8.

Table 5-8: Prediction for the time needed to reach a certain erosion state in zone B

Grass quality [m/s]		Damage criteria [m^2/s^2]		
		C $\approx$ 500	C $\approx$ 1000	C $\approx$ 3500
Very poor	$U_c = 1.5$	4 [h] of 1 [l/m/s]	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
			0.5 [h] of 10 [l/m/s]	3 [h] of 10 [l/m/s]
Poor	$U_c = 3.0$	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
		0.5 [h] of 10 [l/m/s]	3 [h] of 10 [l/m/s]	6 [h] of 10 [l/m/s] +
Average	$U_c = 4.0$	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
		2 [h] of 10 [l/m/s]	5 [h] of 10 [l/m/s]	6 [h] of 10 [l/m/s] +
Good	$U_c = 5.5$	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
		6 [h] of 10 [l/m/s] +	6 [h] of 10 [l/m/s] +	6 [h] of 10 [l/m/s] +
		2 [h] of 30 [l/m/s]	5 [h] of 30 [l/m/s]	6 [h] of 30 [l/m/s] +
				4 [h] of 50 [l/m/s]

The results do not differ that much from zone A, because the load parameter only influences the largest waves. This can be seen in the fact that only the good grass quality shows a large increase in the necessary overtopping water volumes. Therefore, if the grass quality is assumed as average (similar to the quality of zone A), only the second and third erosion state would occur

one hour later. This is not seen in the test results, where even after the 10 [l/m/s] test (see Figure 5-6) no real initiation of erosion can be distinguished in zone B. Therefore, two conclusions can be drawn for erosion zone B. Either that the model for this zone overestimates the reality, or that the round form of the object reduces the erosion effects in zone B. Also according to the factual report, the erosion initiated solely in front of the tree and it expanded towards the side of the tree. Therefore, it seems valid to assume the erosion does not initiate in zone B, but additional studies are required.

The erosion prediction if no tree was present on the test slope is shown in Table 5-9. It is assumed the grass quality on the slope is 'good' without the influence of the tree, which is reasonable since they estimated a critical velocity of 5 [m/s] for the slope during the test. Than the table shows that each damage state is reached by a higher overtopping discharge than in case of the presence of a tree. This means the influence of the tree is predicted as quite large, however (as is stated before), the actual grass quality around the tree is not determined.

Table 5-9: Prediction for the time needed to reach a certain erosion state on a slope without object

Grass quality [m/s]		Damage criteria [m ² /s ²]		
		C ≈ 500	C ≈ 1000	C ≈ 3500
Very poor	U _c = 1.5	5 [h] of 1 [l/m/s]	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
			0.5 [h] of 10 [l/m/s]	3 [h] of 10 [l/m/s]
Poor	U _c = 3.0	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
		1 [h] of 10 [l/m/s]	3 [h] of 10 [l/m/s]	6 [h] of 10 [l/m/s] +
Average	U _c = 4.0	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
		3 [h] of 10 [l/m/s]	6 [h] of 10 [l/m/s]	6 [h] of 10 [l/m/s] +
Good	U _c = 5.5	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +	6 [h] of 1 [l/m/s] +
		6 [h] of 10 [l/m/s] +	6 [h] of 10 [l/m/s] +	6 [h] of 10 [l/m/s] +
		2 [h] of 30 [l/m/s]	6 [h] of 30 [l/m/s]	6 [h] of 30 [l/m/s] +
				6 [h] of 50 [l/m/s]

6 Conclusions

Several steps have been taken to address the need for some guidance in the design and safety assessment for flood defences with respect to the vulnerability of structural transitions. First, the exact location where the initiation of erosion takes place is defined. This is shown in Figure 6-1 for a generalized situation with a square and round object.

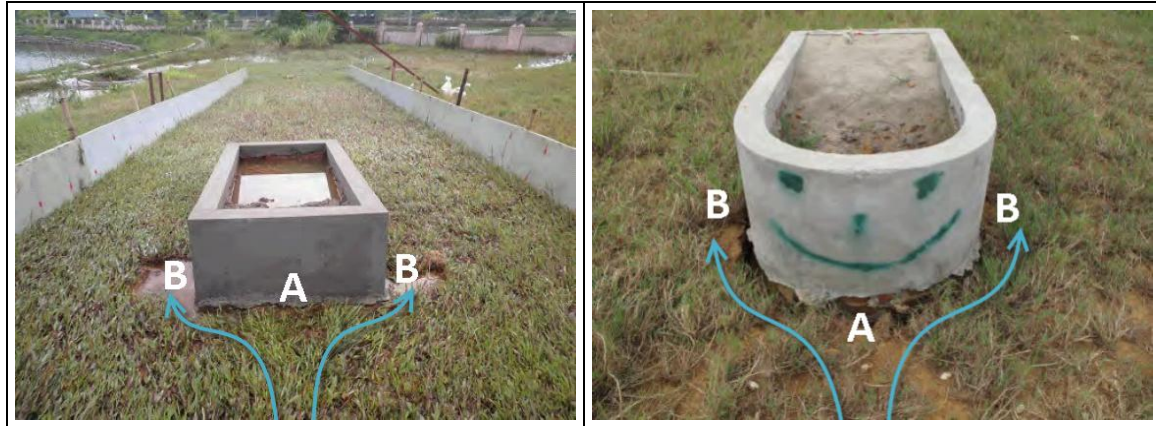


Figure 6-1: Generalized situation for erosion zones around objects (Le, 2012)

Based on earlier field experiments and observations after dike failures, two erosion zones are defined in Figure 6-1:

- Zone A: This is the area in front of the object on the upstream side. Here erosion is not caused by the flow velocity in the direction parallel to the slope, because it reduces to zero. However, it is assumed the dynamic pressures exerted on the grass cover increase significantly due to the impact.
- Zone B: This is the area next to the object, just after the corner (square object). Here erosion can be caused by the flow velocity in the direction parallel to the slope, because it is present and assumed to increase due to the object.

The defined erosion models for the different erosion zones are based on existing models for erosion on the slope. These are paraphrased or extended to fit the purpose for erosion due to the presence of an object. This is performed by adding a parameter on the load side and the strength side of the model. A summary of these models is shown in paragraph 3.4.

Numerical modelling and field experiments

The definition of the load and strength parameters for the models is amongst other things performed by numerical modelling and field experiments, where the set-up of the numerical model is practically similar to the field experiment. The following conclusions have been found:

- The numerical model calculates the velocities on the slope accurately, but due to the lack of friction in the model this velocity accelerates too much on the transition between the

slope and the horizontal part. This higher velocity also causes differences in the impact pressure that is found in front of the object.

- The numerical model shows a reduction of the pressure near the corner of the object. From this could be concluded that generally for smaller objects (below 10 [cm]), the maximum impact pressure is less than half of the impact pressure in front of larger objects (above 50 [cm]). See the table below, where the reduction factor is determined by taking the pressure at a certain distance from the corner of the object and divide it by the maximum pressure in front of the object.

Object dimension [m]	Reduction factor [-]
0.1	0.3
0.3	0.4
0.5	0.9
>1.0	1.0

- The numerical model shows the velocity next to the object has no peak acceleration, but even slightly decreases along the slope next to the object. This means the load parameter in zone B is practically equal to one. Therefore the assumption that a velocity difference influences the load on the grass cover seems not plausible.
- The overtopping experiment included the installation of pressure gauges in erosion zone A of the object and on the slope. The results show large differences for the two locations in both the hydrostatic pressures and the dynamic pressures (turbulence). This varies from almost equal for smaller waves (<1200 [l/m]) up to a factor 2.5 for large waves (4500 [l/m]) in front of the object. Conclusions with respect to the actual erosion around the object cannot be drawn, because due to gaps in the soil in front of the object; this area was replaced with concrete.
- The turf tensile experiment is performed by loading the cover layer until a certain displacement for several times and measure the subsequent force that is needed. This showed that the effect of fatigue was present as assumed, because the force needed to reach a certain grass sample displacement decreased. In other words, after one loading cycle some (weak) roots break, but quite some others get only partly pulled out. This way the force can be redistributed to other roots so the soil keeps his strength. Nevertheless, this pulse load that should correspond with overtopping waves still shows tensile strength values larger than the theoretical value.
- The turf tensile experiment also studied the influence of the sides of a square grass turf on the total tensile strength. This influence appears to be slightly less than theoretically would be assumed.

Validation

As stated above the numerical modelling and field experiments lead to assumptions for the load and strength parameters. These are combined with the erosion models that are defined for zone A and B. A summary of the models and its parameters for zone A and zone B are shown in paragraph 3.4. As a first validation of the models, the field experiment at the Vechtdijk is used. This experiment consisted of a slope with a tree close to the toe. This resulted in the following conclusions:

- For the erosion in zone A, it appears the cumulative load model predicts the amount of overtopping needed to reach the damage criteria quite well for the Vechtdijk. However, the grass quality is assumed as average, which was not investigated during the experiment. Nevertheless, it is very likely the roots of the tree decrease the quality of the grass close to it. The impact load model also gives a reasonable, but very indicative prediction.
- For the erosion in zone B, it appears the cumulative load model predicts that slightly more overtopping waves are needed to reach the damage criteria for average grass. This is not seen in the experiment results, where the real initiation of erosion in zone B can be seen after zone A already shows a large erosion pit. Therefore, two conclusions can be drawn for the model of erosion zone B. Either that the model for this zone overestimates the reality, or that the round form of the object reduces the erosion effects in zone B. Also according to the factual report, the erosion initiated solely in front of the tree and it expanded towards the side of the tree. Therefore, it seems valid to conclude zone B is not of any influence, given an object with a round shape.

7 Recommendations

In order to guide further research on the erosion of grass covers at the transition with structures due to overtopping of flood defences, the following recommendations are given:

1. The erosion zones that are used to define the problem and the models are located next to each other. However, the exact location of the transition between both models is not defined. This should be investigated to say something about the relation between the zones. For instance, whether the erosion in zone A could influence the initiation of erosion in zone B, because the velocity next to the object did not increase according to ComFLOW. It could be possible that the erosion in zone B is initiated by flow concentration through an erosion crack that appears in zone A.
2. The numerical model ComFLOW calculates the flow direction and movement of the water quite accurately, because it looks identical to observations in the field. However, the model lacks a certain roughness of the model boundaries (bottom) and objects to calculate the velocities of the overtopping wave correct over the entire slope length and toe. Furthermore, to improve the water height found in the model, an object could be placed next to the initial water column to restrict the outflow area (like the real overtopping simulator). If these improvements lead to better representations of the wave height and most importantly the wave velocities, the model could be extended to analyse for instance objects with different shapes and dikes with different slope angles.
3. During the overtopping field experiment two pressure gauges were placed in the concrete in front of the object. First of all it would be interesting to see how the grass cover would behave without the concrete. This would help to confirm or contradict the erosion models. Also measurements with pressure gauges in zone B could help to see whether the dynamic pressures increase in this area, since the velocity did not increase according to ComFLOW (this could also be measured). Other measurements could be performed to confirm the pressure distribution found in ComFLOW around the corner of the object. This seems rather costly though, since quite some pressure meters would be necessary. Furthermore it would be interesting to investigate the difference between objects on the horizontal part (after the slope) and on the slope; is this just a difference in velocity or does the geometry also have an effect. Finally it has to be stated that most of these investigations could also be performed by ComFLOW. However, the model needs some improvements first so it calculates the correct velocities as is stated at number two.
4. During the field experiment with the tensile strength apparatus, the displacement of the grass cover was used as the starting point of the experiment. However, now the load on the grass cover for several wave volumes by dynamic pressures is determined, the grass could be loaded multiple times by these forces. This would give insight in how long a grass cover is able to withstand several wave volumes. Furthermore, the contribution of the side planes of a grass turf seemed overrated in theory. These experiments could contribute to the investigation whether the theory needs improvement.
5. The validation of the models is performed with an earlier field experiment at the Vechtdijk. This experiment had a slope with a tree close to the toe. Despite the results,

the effects of a round shape compared to a square shape should be investigated. Likewise if the location of the object on the dike only affects the impact velocity or also the erosion. Finally the grass quality is assumed as average, which should be confirmed by performing more experiments.

6. The cumulative pressure load model for zone A is based on the dynamic pressures of the overtopping waves. These values are not easy to measure and therefore not the easiest way to describe the model. If a relation could be determined between the dynamic pressure and the impact velocity of the wave; the erosion in front of the object could also be described by the easier applicable flow velocity. Another option is to determine a relation between the wave volume and the dynamic pressure to make the input of the model more convenient.
7. All the models use a strength parameter that depends on the dimensions of the eroding grass turf. These are assumed from previous experiments, but have a large variation. Therefore it seems relevant to study the sensibility of the models with respect to the turf dimensions.

Bibliography

- Bakker, J.J., Galema, A.A., Mom, R.J.C., Steendam, G.J. (2010). *SBW Golfverslag en Sterkte Grasbekleding: Factual Report*, Infram.
- Battjes, J.A. (2001). *Lecture Notes Ocean Waves*, Delft University of Technology.
- Bogaard, T. (2012). *Modelling the anisotropy of turbulence with the SWASH model*, MSc-Thesis, Delft University of Technology.
- Bosman, G. (2007). *Velocity and flow depth variations during wave overtopping*, MSc-Thesis, Delft University of Technology
- Chen, X. (2011). *Hydrodynamic loads on buildings caused by overtopping waves*, CoMEM.
- Davidse, M.P. (2009). *Wave impact on asphaltic concrete revetments*, MSc-Thesis, Delft University of Technology.
- Dessens, M. (2004). *The influence of flow acceleration on stone stability*, MSc-Thesis, Delft University of Technology.
- EurOtop (2007). *Wave Overtopping of Sea Defences and Related Structures - Assessment Manual*, UK: N.W.H. Allsop, T. Pullen, T. Bruce. NL: J.W. van der Meer. DE: H. Schuettrumpf, A. Kortenhaus.
- Galema, A.A., De Jong, R., Pruis, K.W., Wisse, A. (2006). *Golfverslag en sterkte binnentalud van dijken*, Noordelijke Hogeschool Leeuwarden.
- Garcia, S.H. (2007). *Reinforced grass on inner dike slopes*, MSc-Thesis, Delft University of Technology.
- Hoffmans, G.J.C.M. (2012). *The Influence of Turbulence on Soil Erosion*, Deltares Select Series.
- Hoffmans, G.J.C.M. (2013). *Turf-tensile experiments*, Deltares.
- Hoffmans, G.J.C.M., Akkerman, G.J., Verheij, H.J., Van Hoven, A., Van der Meer, J. (2009). *The erodibility of grassed inner dike slopes against wave overtopping*, Deltares, Royal Haskoning, Delft University of Technology, Van der Meer Consulting.
- Hoffmans, G.J.C.M., Van Hoven, A., Verheij, H.J. (2010). *Instability of Grass caused by Wave overtopping*, Deltares.
- Hoffmans, G.J.C.M., Verheij, H.J. (1997). *Scour Manual*, Balkema.
- Husrin, S. (2007). *Laboratory experiments on the erosion of clay*, FLOODsite.
- ILH (2013). *International Levee handbook (concept)*, CIRIA et al.
- ILIT (2006). *Investigation of the Performance of the New Orleans Flood Protection Systems in Hurricane Katrina*, Independent Levee Investigation Team.
- INFRAM (2012). *Graszodetrekproeven Millingen, Nijmegen en Zwolle*, Memo INFRAM, Deltares.

- Jonkman, S.N., Vardhanabhuti, B., Blommaart, P., De Bruin, B., Hardeman, B., Kaensap, K., Van der Meer, M., Schweckendiek, T., Vrijling, J.K. (2012). *Post-flood field investigation in the Lower Chao Phraya River Basin*, Expertise Netwerk Waterveiligheid
- Kleefsman, T. (2005). *Water Impact Loading on Offshore Structures: A Numerical Study*, PhD-Thesis, University of Groningen.
- Klein Breteler, M., Smith, G.M. (1996). *Grasdijken: Aanvullende analyse van de waterbeweging op het binnentalud*, Waterloopkundig Laboratorium.
- Kok, M., Vermeij, E., Kanning, W., Kolen, B., Thonus, B., Adams, A. (2007). *Twee jaar na Katrina*, HKV icm. TU Delft.
- Kolen, B., Slomp, R., Van Balen, W., Terpstra, T., Bottema, M., Nieuwenhuis, S. (2010). *Schade door overstroming*, HKV icm. Rijkswaterstaat.
- Le, H.T. (2012). *Wave Overtopping Simulator tests on a 1/15 slope protected by two local grass species*, Delft university of Technology.
- Le, H.T., Van der Meer, J., Nguyen, Q.L., Verhagen, H.J., Schiereck, G.J. (2009). *Wave Overtopping Resistance of Grassed Dike Slopes in Vietnam*, Delft University of Technology.
- Le, H.T., Van der Meer, J., Nguyen, Q.L., Verhagen, H.J., Schiereck, G.J. (2011). *Wave Overtopping Resistance of Grassed Dike Slopes in Vietnam*, Delft University of Technology.
- Luppes, R., Bunnik, T., Kleefsman, T., Loots, E., Veldman, A., Wellens, P., Wemmenhove, R. (2010). *Manual ComFLOW v3.0*, University of Groningen.
- Mous, B.C. (2010). *Wave impact on grass covered outer slopes*, MSc-Thesis, Delft University of Technology.
- Schiereck, G.J. (2001). *Introduction to Bed, Bank and Shore protection*,
- Schüttrumpf, H., Van Gent, M.R.A. (2003). *Wave overtopping at Seadikes*, Coastal structures 2003.
- Stanczak, G. (2007). *Laboratory tests on the erosion of clay revetment of sea dike with and without a grass cover induced by breaking wave impact*, FLOODsite.
- Steendam, G.J., Hoffmans, G.J.C.M., Bakker, J.J., Meer et al., J., Frissel, J., Paulissen, M., Verheij, H. (2012). *SBW Wave Overtopping and Grass Cover Strength: Model development*, Deltares.
- TAW (1998). *Technisch Rapport Erosiebestendigheid van Grasland als Dijkbekleding*, Technische Adviescommissie voor de Waterkeringen.
- TAW (2001). *Technisch Rapport Waterkerende Grondconstructies*, Technische Adviescommissie voor de Waterkeringen.
- TAW (2002). *Technisch Rapport Golfoploop en Golfoverslag bij Dijken*, Technische Adviescommissie voor de Waterkeringen.
- Thornton, C., Van der Meer, M., Scholl, B., Hughes, S., Abt, S. (2011). *Testing Levee Slope Resiliency at the new Colorado State University Wave Overtopping Test Facility*, Colorado State universit, Task Force Hope, USACE.

- Valk, A. (2009). *Wave overtopping: Impact of water jets on grassed inner slope transitions*, MSc-Thesis, Delft University of Technology.
- Van den Bos, W. (2006). *Erosiebestendigheid van grasbekleding tijdens golfoverslag*, MSc-Thesis, Delft University of Technology.
- Van den Bosch, I.C. (2010). *Numerical modeling of wave run-up on a dike*, MSc-Thesis, Technical University of Delft.
- Van der Meer, J., Bernardini, P., Snijders, W., Regeling, H.J. (2006). *The wave overtopping simulator*, ICCE 2006.
- Van der Meer, J., Hardeman, B., Steendam, G.J., Schüttrumpf, H., Verheij, H.J. (2010). *Flow Depths and Velocities at Crest and Inner Slope of a Dike, in Theory and with the Wave Overtopping Simulator*, Van der Meer Consulting, Rijkswaterstaat, Infram, University of Aachen, Delft University of Technology and Deltares.
- Van der Meer, J., Thornton, C., Hughes, S. (2011). *Design and operation of the US wave overtopping simulator*, Van der Meer Consulting, Colorado State University, USACE.
- Van der Meer, J., Verheij, H.J., Van Hoven, A. (2010a). *SBW Golfoverslag en Sterkte Grasbekleding: Evaluatie Vechtdijk*, Deltares.
- Van Hoven, A., Hoffmans, G.J.C.M., Verheij, H.J., Akkerman, G.J., Van der Meer, J. (2010). *SBW Golfoverslag en Sterkte Grasbekleding: Modelontwikkeling Vechtdijk*, Deltares.
- Van Hoven, A., Verheij, H.J., Van der Meer, J., Steendam, G.J. (2009). *SBW Golfoverslag en Sterkte Grasbekleding: Evaluatie Boonweg*, Deltares.
- Vavrina, L. (2006). *Erosion Processes on Dike Slopes*, MSc-Thesis, Delft University of Technology, University of Duisburg-Essen.
- Verheij, H., Klein Breteler, M. (2010a). *SBW Reststerkte, inventarisatiestudie: Reststerkte van gras bij erosie door golfoverslag* Deltares.
- Verheij, H.J., Hoffmans, G.J.C.M., Steendam, G.J., Van der Meer, J., Van Hoven, A. (2010b). *SBW Golfoverslag en Sterkte Grasbekleding: Evaluatie Afsluitdijk*, Deltares.
- Verheij, H.J., Van Hoven, A., Van der Meer, J., Steendam, G.J. (2009). *SBW Golfoverslag en Sterkte Grasbekleding: Modelontwikkeling Afsluitdijk*, Deltares.
- Verruijt, A. (2006). *Offshore Soil Mechanics*, Delft University of Technology.
- VTV (2006). *Voorschrift Toetsen Veiligheid: Guidelines safety assessment for primary sea defences*, Rijkswaterstaat.
- Weijers, J., Tonneijck, M. (2009). *Lecture Notes Flood Defences*, Delft University of Technology.
- Wenneker, I. (2010). *ComFLOW simulations of Wave Impacts on a Dike*, Deltares.
- Wu, T.H., McKinnell, W.P., Swanston, D.N. (1979). *Strength of tree roots and landslides on Prince of Wales Island, Alaska*, Canadian J. of Geotechnical Res.

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A Recent failure examples

Three recent flood events are discussed in this appendix: New Orleans 2005, France 2010 and Thailand 2011; followed by some applied improvements after the events.

A.1 New Orleans, 2005

Hurricane Katrina was one of the five deadliest hurricanes in the United States history. Not just directly, but also from the subsequent floods. These floods also caused property damage of around \$81 billion. Eventually 80 per cent of the city of New Orleans was flood, mostly because of catastrophically failing surge protection systems. After the disaster many investigations on what happened and what should be done have taken place. A few dike failures that are interesting for the topic of structural transitions are discussed here.

In Figure A-1 the water came from the upper left side and passed over the railroad and dike. It started eroding at the transition between the railroad ballast and the earthen dike. Finally the entire dike section eroded. Figure A-2 shows erosion due to overtopping of a concrete wall. A large flood control gate is shown in Figure A-3 and it remained perfectly in place. The connected earthen dike structure though, is completely eroded. Finally Figure A-4 shows a same kind of construction, only this one is eroded on the other side (Kok et al., 2007) (ILIT, 2006).

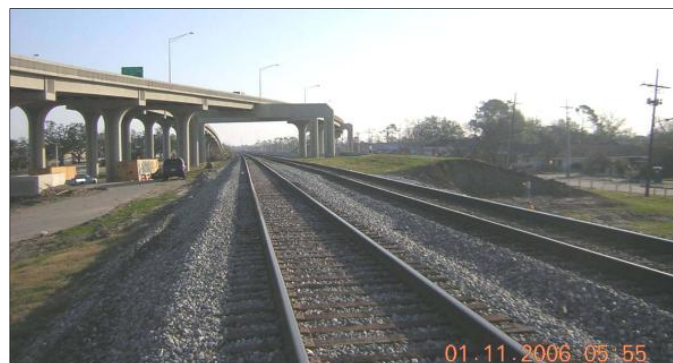


Figure A-1: The eroded earthen dike behind the railroad ballast (picture: Rune Storesund)



Figure A-2: Observed scour at a pump station structure transition with soil (picture: Rune Storesund)



Figure A-3: The dike transition suffered from severe erosion (picture: Les Harder)



Figure A-4: The transition suffered from extensive erosion due to overtopping (picture: Les Harder)

A.2 France, 2010

On February 2010 the storm Xynthia, passed over the northeast part of France. This caused a major power fail, a lot of property damage and some dike failures. Most of the dike damage was fixed easily, because there was no complete breach. Some other dikes were overtopped, but still kept their strength. Nevertheless, there was a lot of erosion, especially at transitions between soft and hard revetments. Figure A-5 shows a breach of approximately 10 [m] at the transition between a promenade and an earthen dike structure. (Kolen et al., 2010)



Figure A-5: Failure at the transition between an earthen structure and a promenade (Kolen et al., 2010)

A.3 Thailand, 2011

Large parts of Thailand have experienced severe flooding during the second half of the year 2011. The economic and societal damage is enormous. At several locations severe erosion of earthen dikes was observed. Especially dikes with asphalt roads on top showed damage, due to overflow. In addition, some floodgates showed breaches next to the location of the floodgate, like the ones that were observed in New Orleans. An example of this is shown in Figure A-6. (Jonkman et al., 2012)



Figure A-6: Failure next to the Pra Ngam floodgate

A.4 Solutions

The obvious solution to counteract the forces that occur around structures is to reinforce the top layer. This is exactly what they did in New Orleans after the floods of 2005. The surge barriers were surrounded by rip-rap on the dike bodies to work as a revetment for the outer slope, crest and inner slope, see Figure A-8. However, this does create another transition further away from the barrier between the rip-rap and adjacent grass cover. They also reinforced the soil around non-water retaining objects, see Figure A-7. Here rock-gabion transition zones were applied to prevent scour. These solutions are able to increase the strength, but are also expensive to apply for every object. (ILIT, 2006)



Figure A-7: Dike protection work of soil-structure interactions (picture: Rune Storesund)



Figure A-8: The transition has been reinforced with rip-rap. (picture: Rune Storesund)

B Former field experiments

Researching the processes that occur around structural transitions cannot be performed entirely from behind a desk. Therefore field experiments take place to gain more insight. As discussed before, in the Netherlands the so called SBW project included several field experiments in there research. Other countries are also showing activities in field experiments in the area of dike erosion, especially Vietnam and the United States. The main activities of these three countries are discussed in respectively paragraphs B.1, B.2 and B.3.

B.1 The Netherlands

As discussed in the introduction of this thesis, the project 'Sterkte en Belastingen Waterkeren' (SBW) is a project of Rijkswaterstaat that focusses on the failure mechanisms of dikes. The project 'Golfoverslag en Sterkte Grasbekleding' (English: Wave overtopping and Strength Grass covers) is a part of the larger SBW project. The goal is to gain insight into the failure mechanisms of a grass cover due to overtopping and to develop safety assessment and design guidance. The methodology of the project is based on a cyclic model (KOLB-principle), see Figure B-1. This cycle is completed for every experiment that is performed within the project. During these experiments a wave overtopping simulator is used to simulate a certain storm intensity that loads the inner slope.

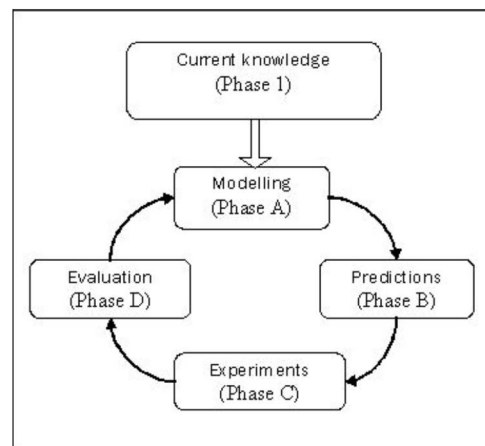


Figure B-1: KOLB-principle

The first experiment of the project was at the Boonweg in Friesland. Here four dike sections are tested, with as main difference the type of management and maintenance of each section. This experiment led to an update of an erosion model, because the bulging mechanism was observed for the first time, see Van Hoven et al. (2009).

In 2009 the next experiment took place at the Afsluitdijk. Three dam sections were tested, one empty section, one with pavement after the slope and one with a concrete stair in the slope. The results show that the grass cover on the crest and inner slope offered a lot of resistance against erosion. At the geometrical transition though, erosion did appear. Also the pavement eroded very fast after the first brick was taken out. The stair showed a lot of stream concentration,

which led to erosion at the sides and bottom of the stair. This went so fast the experiment had to be stopped early. This experiment is evaluated in Verheij et al. (2010b).

The importance of transitions was clear, but the actual knowledge was not sufficient. Therefore in 2010 the next experiment took place at the Vechtdijk, where the effect of a sloping road and tree was tested along with an empty slope test. At the sloping road or ramp the first stones washed away at a discharge of 50 [l/s/m], followed by a lot of underlying sand, see Figure B-2.



Figure B-2: End result at the first test slope

The tree had a similar result, in no-time the sand between the roots started to wash out at a discharge of 30 [l/s/m], see Figure B-3. To continue the experiment the tree had to be protected against the higher discharges. (Van der Meer et al., 2010a)



Figure B-3: Result after second session 30 [l/s/m]

B.2 Vietnam

Since 2009, destructive tests have been performed on three sea dikes in the North of Vietnam. Vetiver grass and Bermuda grass are the most common species on the dike slopes. The first test showed that the Vetiver grass could resist against erosion on an inner slope until discharges of 80 [l/s/m]. The Bermuda grass though, failed at a discharge of 70 [l/s/m]. (Le et al., 2011)

In 2011, destructive tests have been carried out on a 1/15 slope with the same two local grass species as cover. They also built two objects (square and round) on the dike slope to gain some insight into the effects on the erosion rate. In general the tests resulted in erosion at the crest and toe of the dike, but only superficial. The objects seemed to have very little effect, both showed a bit of erosion in front of it, where the square one initiated slightly more erosion. Thus it is concluded that in case of very gentle slopes, objects will most likely not result in failure. (Le, 2012)

B.3 United States

During Hurricane Katrina that struck New Orleans in 2005, the most common cause of dike failure was attributed to severe wave overtopping and erosion of the landward (or protected) side slope. In the absence of reliable design guidance for assessing dike slope resiliency, Task Force Hope of the New Orleans Corps of Engineers recognized the need to conduct full-scale tests of dike slopes to evaluate the performance of grass and various slope armouring alternatives. Colorado State University (CSU) was commissioned to design and build a unique testing facility that would be capable of simulating full-scale wave overtopping, using maximum average overtopping discharges between 200 and 300 l/s per m. CSU designed and constructed a large wave overtopping test facility based on the Dutch mobile overtopping simulator design.

Levee Slope Surface	Hrs	Max. Ave. Discharge		End Result
		l/s per m	ft ³ /s per ft	
Bare Clay	1.3	19	0.2	Severe Erosion
Bermuda Grass	24	370	4.0	No Damage
Bahia Grass	17	280	3.0	No Damage
Bermuda w/TRM	9	370	4.0	No Damage
Bermuda w/HSTRM	9	370	4.0	No Damage
Bermuda w/Ruts	9	370	4.0	Minor Erosion
Lime-Stabilized Clay	2	370	4.0	Severe Erosion
Articulated Concrete Block	3	370	4.0	Minor Erosion
Dormant Bermuda Grass	3	230	2.5	Erosion

Figure B-4: Summary of dike resiliency tests (Thornton et al., 2011)

In Figure B-4 the results of the 2011 tests are presented. It shows the bare clay slope failed rapidly, but well-maintained and healthy Bermuda grass did not fail under extraordinary levels of wave overtopping. The superior resiliency of the Bermuda grass was attributed to dense roots, ample thatching of the grass plants, and few imperfections. Healthy Bermuda grass with wheel ruts also survived at high average wave overtopping rates, but dormant Bermuda grass did only resist significant damage at smaller overtopping loads. (Thornton et al., 2011)

Additional research that would help advance the state of engineering analysis should include the testing of less than perfect grasses and soils. This is usually the case at most dike covers and also at structural transitions.

C Grass tensile strength

In 2012 and 2013, two tests are performed with a turf-tensile apparatus. The complete results are described in respectively paragraph C.1 and paragraph C.2.

C.1 Experiment one

The results of the grass tensile strength test performed by INFRAM (2012) in august of 2012 are shown in Table C-1. The numbering of the tests is stated in the following way:

- The first number shows the location (1 = Millingen a.d. Rijn, 2 = Nijmegen, 3 = Zwolle).
- The character shows whether it is a dry (droog) or wet (nat) test.
- The second number shows the amount of cut sides.
- The third number shows the number of the test.

Table C-1: Grass tensile strength test results (INFRAM, 2012)

Test	Location	Slope	Orientation	Result [Bar]	Tensile force [kN]	Avg. [kN]	Possible tensile strength without cut sides [kN]
1D2.1	Millingen a.d. Rijn	1:2.9	N	11.50	0.88	0.99	1.42
1D2.2	Millingen a.d. Rijn	1:2.9	N	14.25	1.09		
1D4.1	Millingen a.d. Rijn	1:2.9	N	6.50	0.50		
1D4.2	Millingen a.d. Rijn	1:2.9	N	8.00	0.61	0.56	0.43 = difference 2 and 4 cut sides
1N2.1	Millingen a.d. Rijn	1:2.7	N	12.00	0.92	0.90	1.37
1N2.2	Millingen a.d. Rijn	1:2.7	N	11.50	0.88		
1N4.1	Millingen a.d. Rijn	1:2.7	N	6.25	0.48		
1N4.2	Millingen a.d. Rijn	1:2.7	N	5.00	0.38	0.43	0.47 = difference 2 and 4 cut sides
2D2.1	Nijmegen	1:2.6	ZO	6.00	0.46	0.57	0.80
2D2.2	Nijmegen	1:2.6	ZO	9.00	0.69		
2D4.1	Nijmegen	1:2.6	ZO	--	--		
2D4.2	Nijmegen	1:2.6	ZO	4.50	0.34	0.34	0.23 = difference 2 and 4 cut sides
2N2.1	Nijmegen	1:2.5	ZO	6.00	0.46	0.43	0.36
2N2.2	Nijmegen	1:2.5	ZO	5.25	0.40		
2N4.1	Nijmegen	1:2.5	ZO	6.50	0.50		
2N4.2	Nijmegen	1:2.5	ZO	--	--	0.50	0.07 = difference 2 and 4 cut sides
3D2.1	Zwolle	1:3.6	Z	6.00	0.46	0.47	0.55
3D2.2	Zwolle	1:3.6	Z	6.25	0.48		
3D4.1	Zwolle	1:3.6	Z	5.50	0.42		
3D4.2	Zwolle	1:3.6	Z	4.75	0.36	0.39	0.08 = difference 2 and 4 cut sides
3N2.1	Zwolle	1:3.6	Z	4.75	0.36	0.49	0.68
3N2.2	Zwolle	1:3.6	Z	8.00	0.61		

3N4.1	Zwolle	1:3.6	Z	4.00	0.31		
3N4.2	Zwolle	1:3.6	Z	3.75	0.29	0.30	0.19 = difference 2 and 4 cut sides

Table C-2: Remainder of grass tensile strength test results (INFRAM, 2012)

Test	Location	Slope	Orientation	Thickness [cm]	torn %	turf [cm]	aggregate %	Area [cm ²]
1D2.1	Millingen a.d. Rijn	1:2.9	N	4	75%	6	25%	--
1D2.2	Millingen a.d. Rijn	1:2.9	N	--	--	--	--	--
1D4.1	Millingen a.d. Rijn	1:2.9	N	4.5	100%	--	--	15 x 15
1D4.2	Millingen a.d. Rijn	1:2.9	N	--	--	--	--	15 x 15
1N2.1	Millingen a.d. Rijn	1:2.7	N	4	--	6	--	15 x 28
1N2.2	Millingen a.d. Rijn	1:2.7	N	4	50%	5	50%	15 x 18
1N4.1	Millingen a.d. Rijn	1:2.7	N	3.5	80%	4.5	20%	15 x 15
1N4.2	Millingen a.d. Rijn	1:2.7	N	3	100%	--	--	15 x 15
2D2.1	Nijmegen	1:2.6	ZO	3	100%	--	--	--
2D2.2	Nijmegen	1:2.6	ZO	3	50%	4	50%	--
2D4.1	Nijmegen	1:2.6	ZO	--	--	--	--	--
2D4.2	Nijmegen	1:2.6	ZO	3	50%	4	50%	15 x 15
2N2.1	Nijmegen	1:2.5	ZO	4	50%	5	50%	15 x 18
2N2.2	Nijmegen	1:2.5	ZO	4.5	100%	--	--	15 x 18
2N4.1	Nijmegen	1:2.5	ZO	4	70%	5	30%	15 x 15
2N4.2	Nijmegen	1:2.5	ZO	--	--	--	--	15 x 15
3D2.1	Zwolle	1:3.6	Z	5	--	11	--	15 x 18
3D2.2	Zwolle	1:3.6	Z	3	75%	6	25%	15 x 27
3D4.1	Zwolle	1:3.6	Z	4	50%	9.5	50%	15 x 15
3D4.2	Zwolle	1:3.6	Z	3	50%	9.5	50%	15 x 15
3N2.1	Zwolle	1:3.6	Z	4	100%	--	--	15 x 15
3N2.2	Zwolle	1:3.6	Z	5	40%	8	60%	15 x 15
3N4.1	Zwolle	1:3.6	Z	2.5	50%	6	50%	15 x 15
3N4.2	Zwolle	1:3.6	Z	4	60%	6	40%	15 x 15

C.2 Experiment two

The results of the grass tensile strength test performed by Deltares in March of 2013 are shown in Table C-3 till Table C-9 and Figure C-1 till Figure C-5.

Table C-3: Overview of experimental results of the turf-tensile tests (Hoffmans, 2013)

Test [-]	No. of sides cut [-]	Slope orientation [-]	Max. stress [kN/m ²]	Thickness turf (mean) [cm]	Thickness turf (max.) [cm]	Deformation (max) [cm]	Turf area [cm ²]	Comment
1	2-side	North	24.5	-	-			horz. berm
2	2-side	North	28.9	4	7.5	> 5	25 x 15	on slope (upper)
3	4-side	North	22.1	5	8	-	15 x 15	halfway on slope
4	2-side	North	42.6	8	12	6		halfway on slope
5	2-side	North	40.9	5	11	6		halfway on slope
6	4-side	North	23.8	4		5	15 x 15	halfway on slope
7	2-side	North	18.7	3		5	15 x 15	moss
8	2-side	North	16.3	4	4	6	15 x 15	moles
9	2-side	South	38.5	6.5	9.5	6	23 x 15	halfway on slope
10	4-side	South	25.5	6	7.5	5	15 x 15	halfway on slope

Table C-4: Results of turf-tensile test 5 (Turf, 2 sides cut)

Step	Deformation [cm]	Max. pressure [bar]	Comment
1	1	8.5	
2	1	6.5	
3	1	6	
4	1	5.7	
5	1	6	
6	2	10	deformation step is increased to 2 cm
7	2	9	
8	3	12	turf tears
9	3	10	plastic deformation of 0.5 cm
10	3	10	
11	3	9	
12	3	8	
13	3	8	
14	3	8	
15	4	11	turf tears
16	4	9	plastic deformation of 0.8 cm
17	4	8.5	
18	4	8	
19	4	7.5	
20	4	8	
21	5	10	
22	5	7.5	
23	5	7.5	
24	6	7	end of test

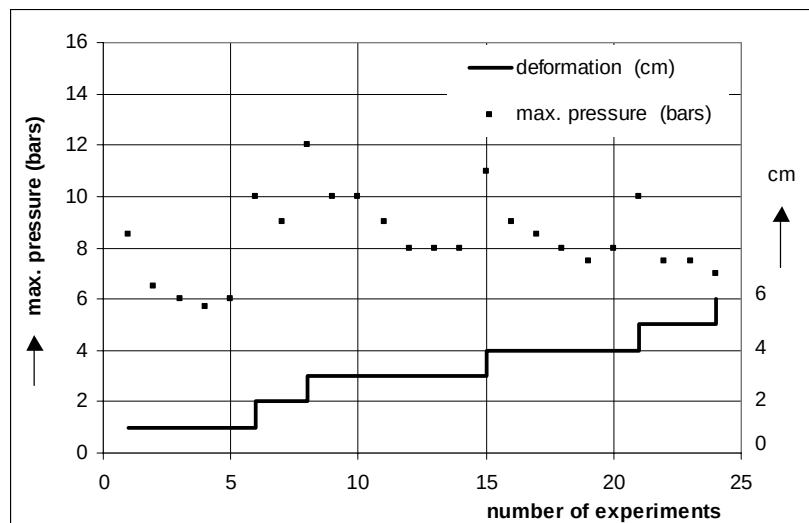


Figure C-1: Results of turf-tensile test 5 (Turf, 2 sides cut)

Table C-5: Results of turf-tensile test 6 (turf, 4 sides cut)

Step	Deformation [cm]	Max. pressure [bar]	Comment
1	0.5	5.5	?
2	0.5	5	
3	0.5	5	
4	0.5	3,7	
5	0.5	4.9	
6	1	5.7	
7	1	5	
8	1	5	
9	1	5.3	
10	1	5.4	
11	2	7	plastic deformation 0,5 cm
12	2	5.3	
13	2	5	
14	2	5.3	
15	2	4.8	
16	3	6.7	
17	3	5.5	
18	3	5.2	
19	3	5.75	
20	3	5.2	
21	4	6.3	(6.12) pressures are obtained from electronic apparatus plastic deformation of 1,0 cm turf tears
22	4	4.6	
23	4	3.9	
24	4	3.8	
25	4	4.1	
26	5		end of test

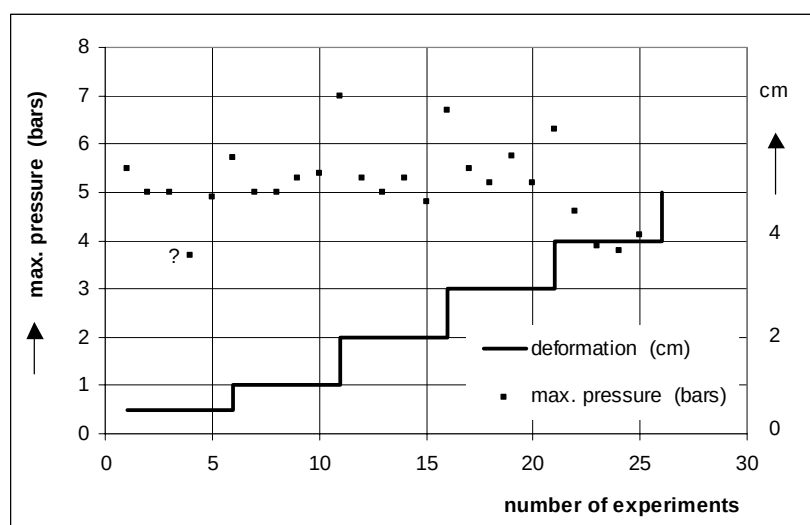


Figure C-2: Results of turf-tensile test 6 (turf, 4 sides cut)

Table C-6: Results of turf-tensile test 7 (moss, 2 sides cut)

Step	Deformation [cm]	Max. pressure [bar]	Comment
1	0.5	5.2	
2	0.5	4.4	
3	0.5	4.4	
4	0.5	3.8	
5	0.5	3.4	
6	1	5.5	
7	1	3.7	
8	1	3.9	
9	1	4.1	
10	1	4.3	
11	2	4.9	
12	2	4.1	
13	2	3.6	
14	2	3.7	
15	2	3.7	plastic deformation 0.5 cm
16	3	4.4	
17	3	3.8	
18	3	3.5	
19	3	3.3	
20	3	3.3	
21	4	3.9	
22	4	3.5	
23	4	3.3	plastic deformation 1.0 cm
24	4	3.2	
25	4	3.2	
26	5	3.6	end of test

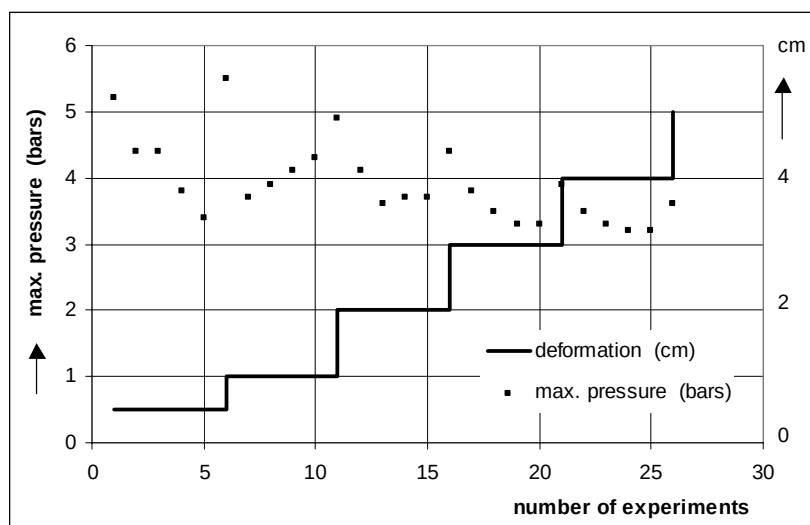


Figure C-3: Results of turf-tensile test 7 (moss, 2 sides cut)

Table C-7: Results of turf-tensile test 8 (moles, 2 sides cut)

Step	Deformation [cm]	Max. pressure [bar]	Comment
1	0.5	3.8	
2	0.5	3.1	
3	0.5	3.1	
4	0.5	3.7	?
5	0.5	3.6	
6	1	3	?
7	1	4.2	
8	1	3.7	
9	1	3.7	
10	1	3.7	
11	2	4.8	
12	2	4.2	
13	2	4.3	
14	2	3.8	
15	2	3.4	
16	3	4.5	
17	3	3.9	
18	3	3.5	plastic deformation of 0.5 cm
19	3	3.7	
20	3	3.8	
21	4	3.8	
22	6	3.6	end of test

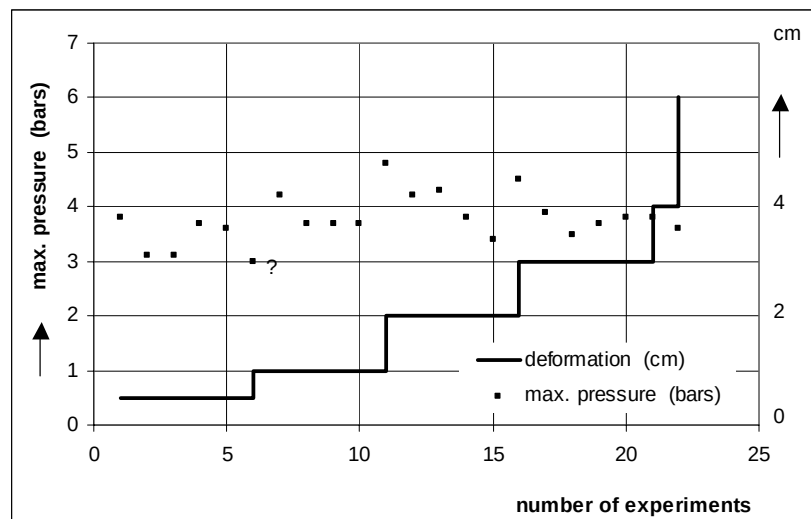


Figure C-4: Results of turf-tensile test 8 (moles, 2 sides cut)

Table C-8: Results of turf-tensile test 9 (turf, 2 sides cut)

Step	Deformation [cm]	Max. pressure [bar]	Comment
1	0.5	5.8	
2	0.5	3.6	
3	0.5	4.8	
4	0.5	5.8	?
5	0.5	4.9	
6	1	7.7	
7	1	6.9	
8	1	6.4	
9	1	6.2	
10	1	6.2	
11	2	9.7	plastic deformation 0.5 cm
12	2	9.1	
13	2	8.1	
14	2	7.6	
15	2	7.3	
16	3	11	
17	3	9.5	
18	3	9.4	plastic deformation 1 cm
19	3	8.8	
20	3	8.5	
21	4	11.3	turf tears
22	4	10.2	
23	4	9.2	
24	4	8.7	
25	4	8.5	
26	5	10.5	turf tears
27	5	8.2	
28	5	7.8	plastic deformation 1.5 cm
29	5	7.8	
30	5	7.3	
31	6	8.2	
32	6	6.4	end of test

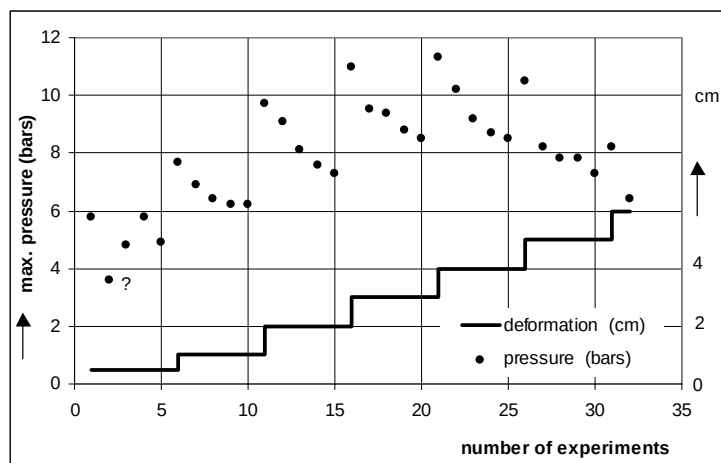


Figure C-5: Results of turf-tensile test 9 (turf, 2 sides cut)

Table C-9: Results of turf-tensile test 10 (turf, 4 sides cut)

Step	Deformation [cm]	Max. pressure [bar]	Comment
1	0.5	5.3	
2	0.5	5.4	
3	0.5	3.5	
4	0.5	5	
5	0.5	4.5	
6	1	6.5	
7	1	5.4	
8	1	4.8	
9	1	5.3	
10	1	5	
11	2	7.5	
12	2	6	
13	2	5.4	plastic deformation 0.5 cm
14	2	5	
15	2	5.3	
16	3	6.3	
17	3	4.2	plastic deformation 1 cm
18	3	4.2	
19	3	4.5	
20	3	4.2	
21	4	4.8	
22	4	4.2	
23	4	3.6	plastic deformation 1.5 cm
24	4	3.1	
25	4	3	
26	5	3.6	end of test

D ComFLOW analysis

This appendix gives some additional information on the results of the numerical ComFLOW model described in paragraph 4.1. The following paragraphs show pressures and velocities found in the ComFLOW model. To clarify the several investigated locations in the model, these are generalized to the cross-sections and points shown in Figure 4-4.

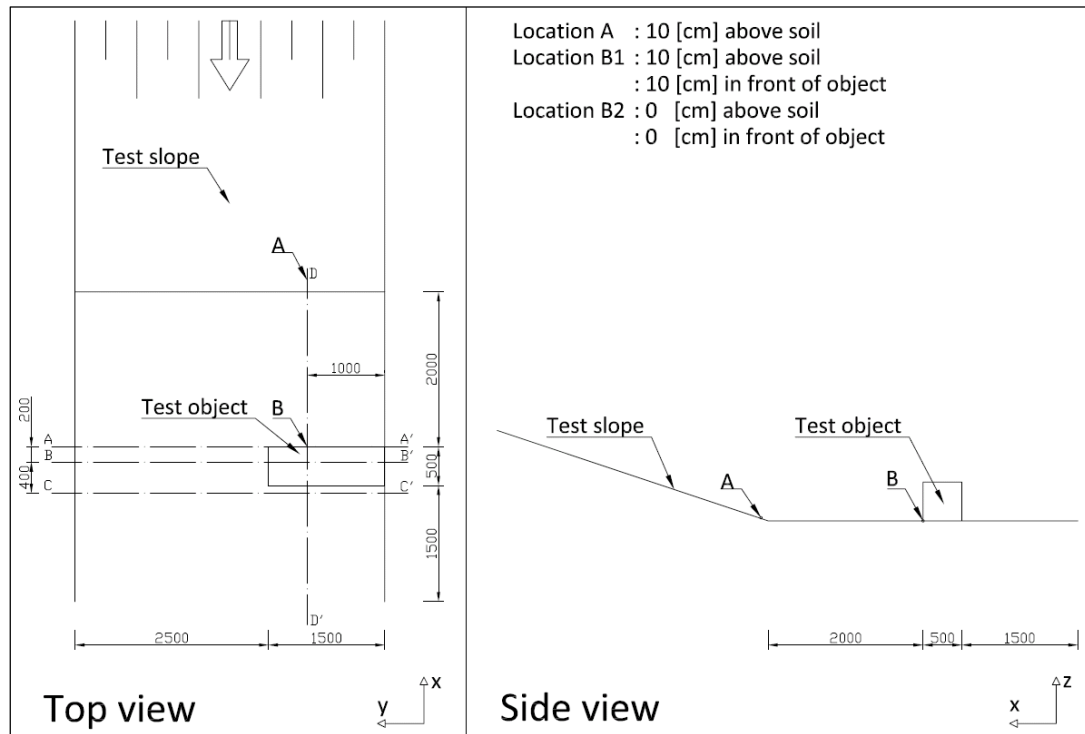


Figure D-1: Applied cross-sections for analysis in ComFLOW

D.1 Wave modelling

It was said before that the cumulative load erosion models shown in chapter 3 is a summation of the relevant waves overtopping a dike during a storm. The tested average discharges in the field experiment of Nijmegen are 1.0, 10, 50, 100 and 200 [l/m/s]. The waves used to simulate these discharges have volumes from about 20 [l/m] up to 4500 [l/m]. Several of these are modelled in ComFLOW to determine several flow aspects that load the grass cover just in front of an object. The wave volumes during a certain overtopping discharge that are used for the field experiment are shown in Table D-1.

Table D-1: Wave characteristics from field experiments that are tried to model in ComFLOW

1 [l/s/m]	Deltares			
No. of wave	Vol. [l/m]	Total Vol. [l/m]	Wave height [m]	Wave velocity [m/s]
68	50	3400	0.01	1.8
31	175	5425	0.03	2.7
18	375	6750	0.05	3.4
8	750	6000	0.08	4.1
1	1500	1500	0.14	5.1
10 [l/s/m]	Deltares			
No. of wave	Vol. [l/m]	Total Vol. [l/m]	Wave height [m]	Wave velocity [m/s]
261	50	13050	0.01	1.8
212	175	37100	0.03	2.7
150	375	56250	0.05	3.4
92	750	69000	0.08	4.1
31	1500	46500	0.14	5.1
3	2500	7500	0.20	5.9
50 [l/s/m]	Deltares			
No. of wave	Vol. [l/m]	Total Vol. [l/m]	Wave height [m]	Wave velocity [m/s]
834	375	312750	0.05	3.4
372	750	279000	0.08	4.1
234	1500	351000	0.14	5.1
57	2500	142500	0.20	5.9
18	3500	63000	0.26	6.7
6	4500	27000	0.31	7.1
3	5250	15750	0.35	7.4
75 [l/s/m]	Deltares			
No. of wave	Vol. [l/m]	Total Vol. [l/m]	Wave height [m]	Wave velocity [m/s]
699	375	262125	0.05	3.4
474	750	355500	0.08	4.1
342	1500	513000	0.14	5.1
108	2500	270000	0.20	5.9
39	3500	136500	0.26	6.7
6	4500	27000	0.31	7.1
18	5250	94500	0.35	7.4

D.2 Effect object width on pressure

As is stated before, the pressure is distributed quite evenly over the width of the structure, with a reduction of the occurring pressure near the corner; located at 1.5 [m] from the right side. This is shown again in Figure D-2 for a wave of 2500 [l/m], but also smaller and larger waves are included. It appears that for large waves of 4500 [l/m], a peak occurs near the corner of the object. This peak has a timeframe of only several milliseconds and is therefore seen as a sonic effect. The measured pressures near the side of the model are pressures that occur in timeframes of tens of seconds. The reason for this sonic peak near the corner should be investigated.

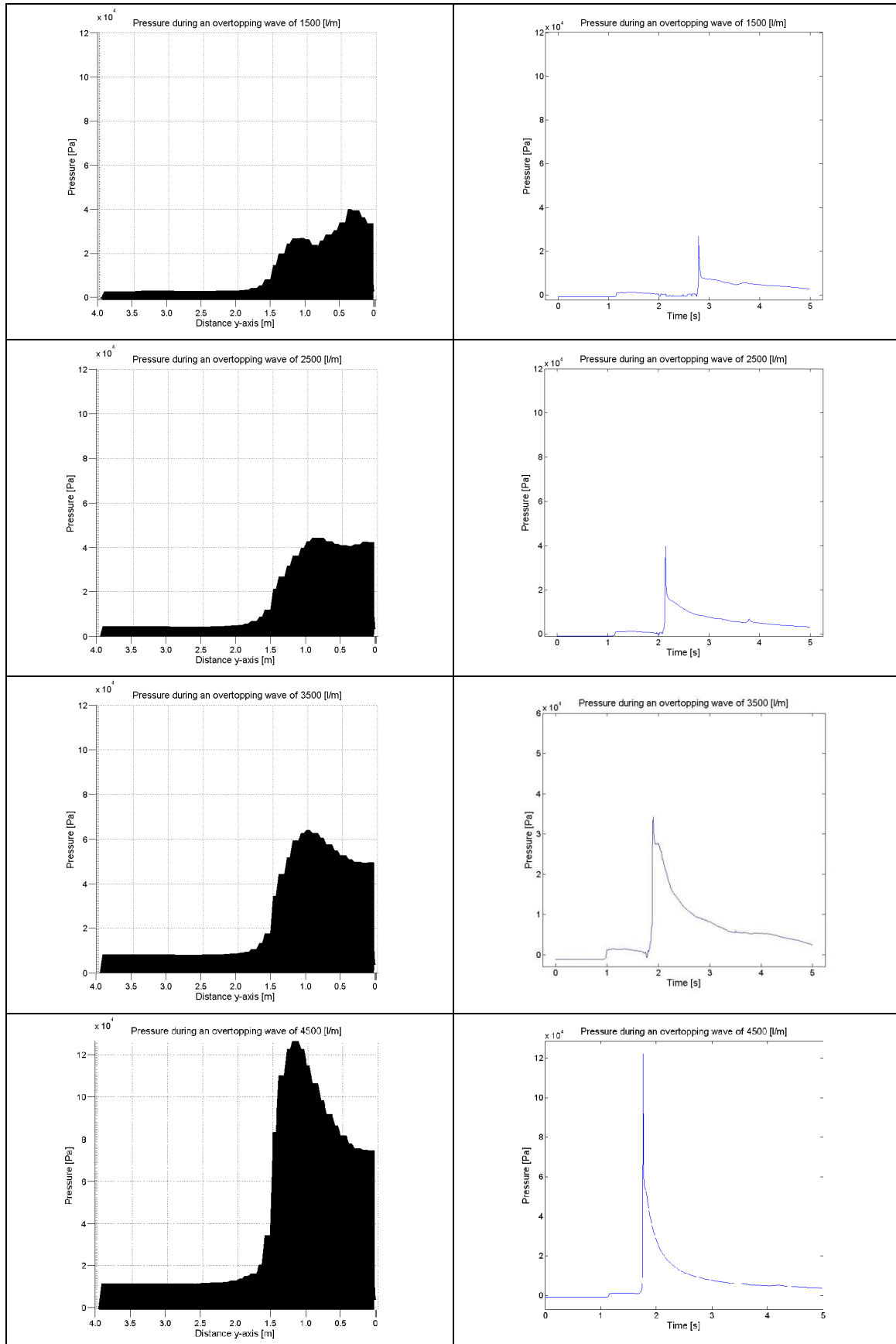


Figure D-2: Pressure distribution in front of the object (cross-section A-A')

D.3 Effect object on velocity

The potential erosion locations around objects are stated as in front of the object and around the corners. It is assumed the increase in flow velocity next to the object is of influence on the erosion at these locations. To verify this assumption the velocity is investigated in the model, ComFLOW. The results for several cross-sections given a wave of 2500 [m/s] are shown in 4.1.3. In Figure D-3 also the results for larger waves are shown to verify the results of chapter 4. Similar to the results of the 2500 [l/m] wave, these results also show hardly any difference in the local flow velocity at the sides of the object; right after the impact. It is possible to state that the velocity shows a marginal increase next to the object. Nevertheless, these local decelerations found in the ComFLOW model are quite small and therefore stay insignificant.

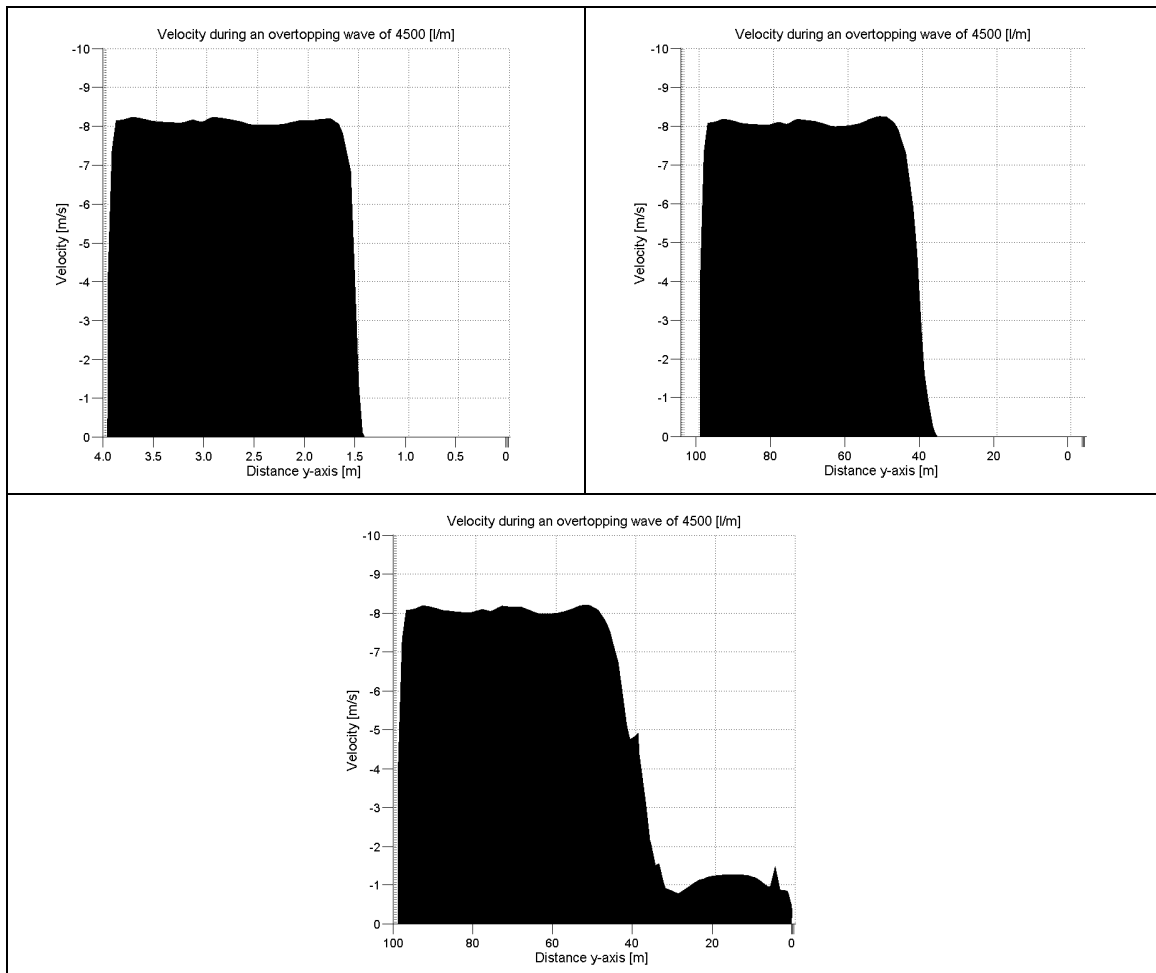


Figure D-3: Velocities in cross-sections A-A', B-B' and C-C'

E Field experiments Nijmegen

As stated before, overtopping field experiments occurred close to Nijmegen in January and February of 2013. The results of these tests are already described in paragraph 4.2, but this appendix shows several illustrations to clarify the occurred events.

	
Overview of the test strip (1)	Overview of the test strip (2)
	
Detail of the object	Detail of the object, including a wave
	
Top view of sensor location	Top view of sensor location



Figure E-1: Pictures overtopping field experiment



Figure E-2: Pictures turf-tensile field experiment

The visual results of the erosion on the slope of test strips one and two are described in respectively Table E-1 and Table E-2.

Table E-1: General results of test strip one

Discharge [l/m/s]	Result
1	No damage
10	Small holes up to 10 [cm], mostly due to the presence of moles.
50	Severe damage at the toe, mostly due to the presence of a semi-pavement. The holes on the slope increased up to 50 [cm].
100	After one hour the holes on the slope increased fast to 60 [cm] and the geotextile in the semi-pavement was torn in half.

Table E-2: General results of test strip two

Discharge [l/m/s]	Result
1	No damage
10	No damage
50	At the downstream corner of the object an erosion hole appeared. The reason for this hole is probably a combination of the bad vegetation and the faster and concentrated flow in this corner due to the sleeper slope towards the fish stairs. The dike slope itself showed hardly any damage.
100	An erosion hole appeared at the semi-pavement after approximately four hours and the underlying geotextile was exposed, this part was than protected. At the end of the session the first serious erosion appeared on the slope at about one metre before the toe.
200	Damage increased slowly on the slope and the test was stopped after the first session.

The main thing that can be concluded from the tests is the importance of the wave velocity. It seems that a gentler slope and thus a lower flow velocity significantly reduce the amount of erosion. This can also be seen at the second test strip. Here the erosion occurred downstream of the object (outside the official test strip) where the slope and thus also the velocity increases again.

